Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings

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

DISCLAIMER

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.

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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* as well as the *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 around 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.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

1.1 Purpose

This report, *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings* has been developed by the SAC Joint Venture under contract to the Federal Emergency Management Agency (FEMA) to provide structural engineers with recommended criteria for evaluation of the probable performance of existing steel moment-frame buildings in future earthquakes and to provide a basis for updating and revision of evaluation and rehabilitation guidelines and standards. 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 evaluation and upgrade criteria, hereinafter referred to as *Recommended Criteria*, is presented in the form of specific recommendations for 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 and guideline 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 seismic evaluation and upgrade criteria are intended as a resource document for organizations engaged in developing and updating guidelines and standards for seismic evaluation and upgrade of steel moment-frame buildings. 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, erectors, inspectors, building officials, 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 behaves 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 welded 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 industrial, commercial and institutional structures employing welded 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 welded 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 to the buildings, 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, welded steel moment-frame buildings damaged by the 1994 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.

Welded 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 welded 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.

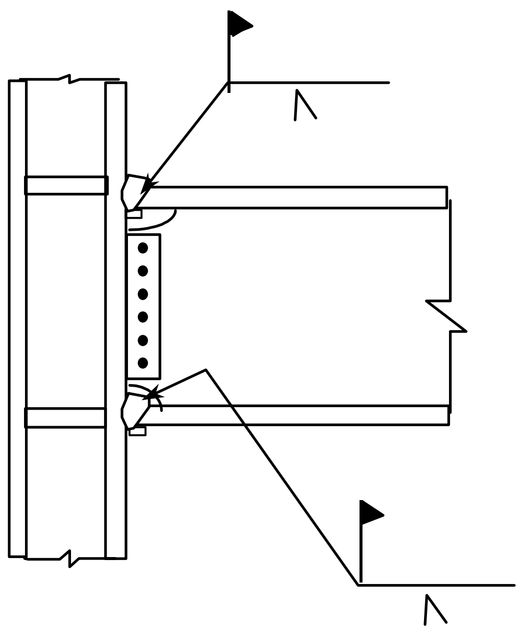


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

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.

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.

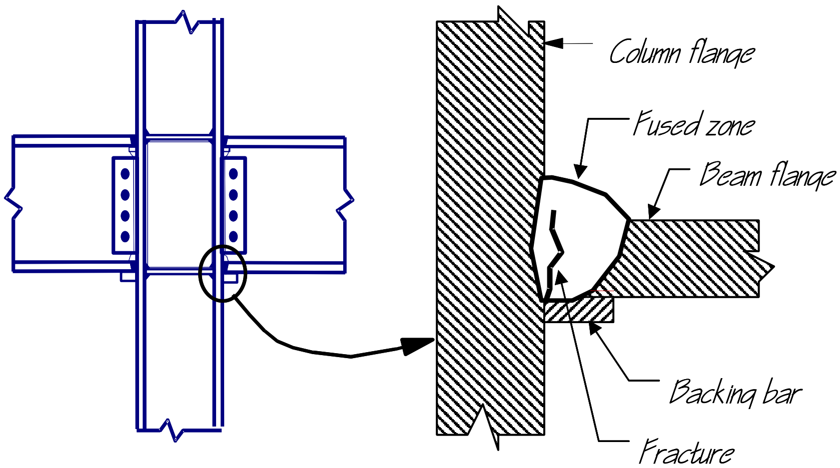


Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection

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

Once such fractures occur, the beam-column connection loses a significant portion of the flexural rigidity and strength needed to resist 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 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).

a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-4 Column Fractures

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 reliably 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 welded steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.

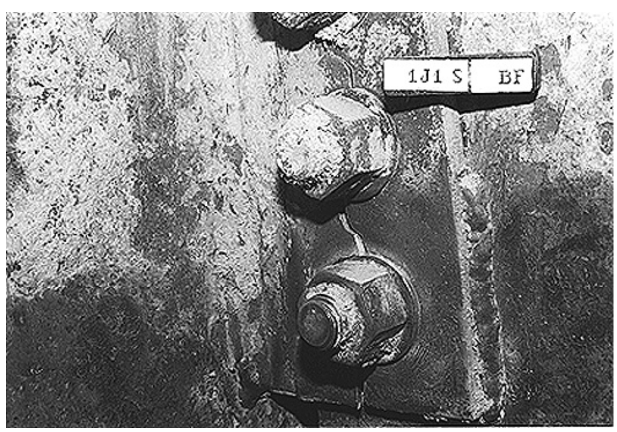


Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

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 that the damage 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 guidelines 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

These *Recommended Criteria* supersede the evaluation and upgrade recommendations for existing WSMF buildings contained in *FEMA-267*, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, and the *Interim Guidelines Advisories*, *Nos. 1* and *2 (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 welded steel moment-frame construction in order to permit more reliable seismic performance. Some users may wish to apply these *Recommended Criteria* to specific engineering projects, prior to their adoption by future codes and standards. Such users are cautioned to consider carefully the codes and standards actually enforced by the building department having jurisdiction for a specific project, and to adjust the *Recommended Criteria* accordingly. These users are also cautioned 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 them in practice.

1.5 Overview of These *Recommended Criteria*

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

- **Chapter 2: Evaluation Overview.** This chapter provides an historic perspective of the development of steel moment-frame design and construction practice in the United States. It also includes discussion of the performance of welded steel moment-frame construction in recent earthquakes and the causes for much of the damage observed in this construction. Guidelines for collection of basic data on the configuration, and the details and materials of construction of a building, needed to conduct an evaluation, are presented, as is a brief introduction into the types of evaluation that may be conducted.
- **Chapter 3: Performance Evaluation.** This chapter presents simplified analytical procedures for determining the probable structural performance of regular, welded, steel moment-frame buildings, given the site seismicity. These procedures allow the calculation of a level of confidence (say, 95%) that an existing structure will achieve a stipulated performance level (e.g., a Collapse Prevention level) for a specified earthquake hazard (e.g., a 2% probability of exceedence in 50 years). If the calculated level of confidence is unacceptably low, then the structure can be upgraded and re-evaluated for more acceptable performance, using these same procedures.
- **Chapter 4: Loss Estimation.** This chapter presents a simplified procedure for estimating the probable postearthquake repair costs for existing, welded, steel moment-frame buildings using basic information on the building's configuration and age, and the intensity of ground shaking at the site.
- **Chapter 5: Seismic Upgrade.** This chapter presents recommendations for two approaches to seismic upgrade of existing, welded, steel moment-frame buildings. The first approach, termed simplified upgrade, consists of modification of individual moment-resisting

connections to reduce their susceptibility to ground-shaking-induced brittle fracture. The second method is a detailed procedure in which the performance of the structure is first evaluated, using the procedures of Chapter 3, an upgrade approach is conceived and designed in a preliminary manner, and the performance of the upgraded structure is evaluated for acceptability. This process is repeated until a suitable level of confidence of acceptable performance is obtained. Upgrades in this second method may consist of connection upgrades, as in the simplified upgrade approach, but may also include modification of the structural system, such as introduction of braces, or energy dissipation devices.

- **Chapter 6: Connection Qualification.** This chapter presents modeling recommendations and performance data for different types of beam-column connections.
- **Appendix A: Detailed Procedures for Performance Evaluation**. This appendix provides recommendations for the implementation of the detailed analytical performance evaluation procedures upon which the simplified procedures of Chapter 3 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 3. Engineers may find the application of these more detailed procedures beneficial in demonstrating that building performance is better than indicated by Chapter 3. Use of these more detailed procedures is required for the performance evaluation of structures with certain irregularities, as indicated in Chapter 3.
- **Appendix B: Detailed Procedures for Loss Estimation**. This appendix provides procedures for developing building-specific, vulnerability (and loss) functions for steel moment-frame buildings. These vulnerability and loss functions are compatible with *HAZUS*, a nationally applicable computer program developed by FEMA that permits estimation of earthquake losses on a building-specific basis, or community or regional basis. These vulnerability and loss functions may also be used with other loss-modeling software and methodologies.
- **References, Bibliography, and Acronyms.**

2. Evaluation Overview

2.1 Scope

This section provides a discussion of the history of the development of steel moment-frame buildings and the general earthquake damage and vulnerabilities associated with such buildings. An overview of the evaluation procedures contained in these recommended criteria is presented along with corresponding sections regarding material property and condition assessment approaches.

2.2 Steel Moment-Frame Building Construction

2.2.1 Introduction

Steel frames have been used in building construction for more than one hundred years. In the early 20th century, typical steel frames were of riveted construction. Beam-column connections were of two common types illustrated in Figure 2-1, in which beams were connected to columns using either stiffened or unstiffened angles at the top and bottom beam flanges. Designers often assumed that these assemblies acted as “pinned” connections for gravity loads and that the stiffened connections would act as “fixed” connections for lateral loads. Although some hot-rolled shapes were available, these were typically limited to beam applications. Columns and girders were often fabricated out of plate and angle sections. Frames were typically designed for lateral wind loading, employing approximate methods of frame analysis, such as the portal method or cantilever method.

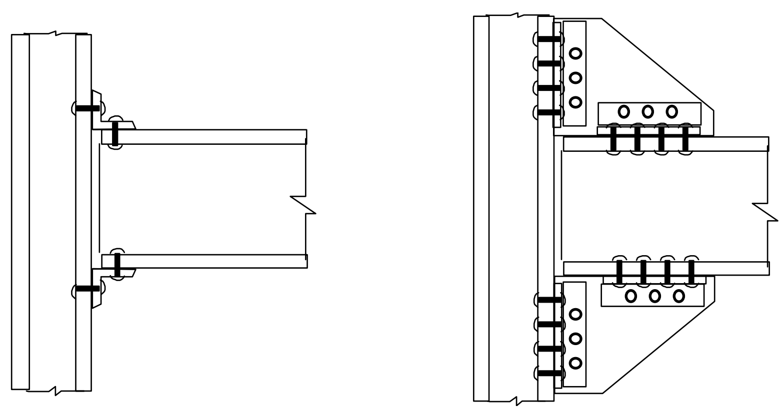


Figure 2-1 Typical Early Beam-Column Connections

Most early steel frame buildings had exterior walls of unreinforced masonry. The exterior building frame was typically embedded in these walls providing for significant interaction between the steel and masonry elements. Although these buildings were usually designed neglecting the effects of the masonry in lateral load resistance, in actuality there is significant interaction between the masonry walls and steel frames and the masonry provides much of the lateral resistance of such buildings.

Infilled masonry construction remained common until the early 1940s. At about that time, reinforced, cast-in-place concrete walls began to replace the masonry used in earlier buildings. These reinforced concrete walls were typically designed to provide the lateral resistance for the structure, and the steel frame was often designed only to carry gravity loading, though some buildings with a "dual" system of concrete walls and steel moment frames also were constructed during this period. Steel moment frames without infill walls came into wider use when curtain wall systems became popular, in the late 1940s and early 1950s. This was the time when moment resistance and stiffness of the connections became a critical issue. The earliest steel moment frames employed riveted or bolted connections similar to those used in the earliest infill masonry buildings. However, as design procedures became more sophisticated and the building codes began to require design for larger seismic forces, designers started to design fully restrained connections intended to develop the full flexural strength of the beams. Connections were usually complex and expensive, consisting, for example, of plates, stiffened angles, and T-sections that were riveted or bolted.

During the Second World War, structural welding was introduced in the ship-building industry as a means of speeding ship construction. It is interesting to note that these early attempts at welded construction were not entirely successful and were plagued by unanticipated fracture problems. Several Liberty Ships, a class of cargo vessel, some of which were among the first to employ welded hull construction, experienced massive fracture damage and a few actually fractured in two and sank. These problems were eventually traced to sharp corners at openings in the hull and superstructure as well as to inadequate notch toughness in the materials of construction. By the 1950s, however, these problems were largely mitigated by improved design and construction practice and welded construction had completely replaced the earlier bolted and riveted construction techniques formerly prevalent in this industry.

In the late 1950s, structural welding began to spread to the building industry. This trend, together with the need to design strong and stiff, but economical, connections, accelerated a design shift from riveted or bolted, partially restrained connections to designs employing welded, fully restrained connections. Many different types of welded connections were used, the earlier ones consisting mostly of shop-welded, field-bolted cover plates connecting the beam flanges to the columns. In the late 1950s the field-welded direct connection between beam flanges and column flanges started to see some use. Experimental research performed in the mid to late 1950s, primarily at Lehigh University, provided criteria for welding and for continuity plate requirements to minimize web crippling and column flange distortions. Additional experimental research performed in the mid 1960s to early 1970s at the University of California at Berkeley provided evidence that certain types of butt-welded beam-flange-to-column-flange connections could behave satisfactorily under cyclic loading. These data lead to widespread adoption of the bolted-web, welded-flange, beam-column connection shown in Figure 2-2, by engineers designing for earthquake resistance.

2.2.2 Welded Steel Moment-Frame (WSMF) Construction

Today, WSMF construction is commonly used throughout the United States and the world, particularly for mid-rise and high-rise construction. Prior to the 1994 Northridge earthquake, this type of construction was considered one of the most seismic-resistant structural systems, due to the fact that severe damage to such structures had rarely been reported in past earthquakes and there was no record of earthquake-induced collapse of such buildings, constructed in accordance with contemporary US practice. However, the widespread reports of structural damage to such structures following the Northridge earthquake called for re-examination of this premise.

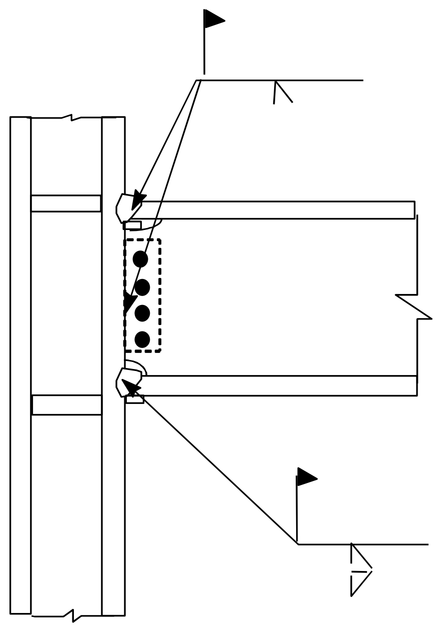


Figure 2-2 Typical Bolted Web, Welded Flange Connection

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, but not brittle fractures. Based on this presumed behavior, building codes permit design of steel moment-frame structures for lateral forces that are approximately 1/8 those which would be required for the structure to remain fully elastic. Supplemental provisions within the building code, intended to control the amount of interstory drift sustained by these flexible frame buildings, typically result in structures which are substantially stronger than this minimum requirement and in zones of moderate seismicity, substantial overstrength may be present to accommodate wind and gravity load design conditions. In zones of high seismicity, most such structures designed to minimum code criteria will not start to exhibit plastic behavior until ground motions are experienced that are 1/3 to 1/2 the severity anticipated as a design basis. This design approach has been developed based on historical precedent, the observation of steel building performance in past earthquakes, limited research that has included laboratory testing of beam-column models (albeit with mixed results), and nonlinear analytical studies.

2.2.3 Damage to Welded Steel Moment-Frame (WSMF) Construction in the 1994 Northridge, California, Earthquake

Following the apparent widespread discovery of steel frame damage in the 1994 Northridge earthquake, the City of Los Angeles enacted an ordinance requiring mandatory inspections of approximately 240 buildings located in the zones of heaviest ground shaking within the City. This ordinance required that a report be filed for each building, indicating that inspections had been performed in accordance with FEMA 267, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, or other suitable approach, and that repairs be made. The resulting database of reported information provides a good overview of the types of damage sustained by buildings in the Northridge earthquake, though some damaged buildings, located in the zones of the most severe ground shaking, were outside the City of Los Angeles and were not included under the ordinance.

Review of statistics obtained from a data base of the damage reported under this ordinance program indicates that the damage was less severe than had originally been perceived. Reports for approximately one third of the buildings affected by the ordinance indicated that no damage was found in the structures. Reports for another one third of the buildings indicated only that there were rejectable defects at the roots of some beam-flange-to-column-flange welds. At the time these inspections were made, there was some uncertainty as to whether such conditions were actually damage or poor quality construction, which had not been detected during the original performance of construction quality assurance, but these conditions were routinely reported as damage. More recent investigations strongly suggest that these weld root flaws are not earthquake damage, but defects from the original construction. Only one third of the total reports prepared under the Los Angeles City ordinance indicated damage other than weld root defects. Of the buildings with reported damage other than weld root defects, two-thirds had less than 10% of their connections fractured. Only 11% of all the buildings included in the ordinance had more than 10% of their connections damaged, while relatively few buildings (13% of the total) accounted for 90% of all damage other than defects at the weld roots.

The distribution of damage in these buildings points to some important potential findings. The concentration of severe damage in a relatively small percentage of the total buildings inspected would seem to indicate that in order to sustain severe damage, a steel moment-frame building must either experience very strong response to the earthquake ground motion, or, as a result of design configuration or construction quality, or both, be particularly susceptible to damage. It would seem that most steel moment-frame buildings are not particularly susceptible to severe damage under ground shaking of Modified Mercalli Intensity VII or less.

Although initial reports following the 1994 Northridge earthquake indicated that more than 100 buildings had sustained severe damage, in many cases this reported damage was limited to discontinuities and defects at the root of the complete joint penetration (CJP) welds between the beam bottom flange and the column flange. As previously noted, there is strong evidence to suggest that most such conditions are not damage at all, but rather, pre-existing construction defects that were not detected during the original construction quality assurance program. Subsequent research in other buildings and cities suggests that the presence of such defects is widespread and generally present in the population of welded steel moment-frame (WSMF) buildings constructed in the United States prior to the Northridge earthquake.

Notwithstanding the above comments, a number of buildings did experience brittle fracture damage in their beam-column connections. The amount of damage sustained by buildings was generally related to the severity of ground shaking experienced at the building site as well as the severity of response of the structure to the ground shaking, although this second factor was not necessarily measured during the earthquake. However, the presence of construction defects in the welded joints was also a significant factor in the initiation of fracture damage. Joints with severe defects at the weld roots were more susceptible to fracture initiation than joints without such defects. Since the distribution of joints with defects in an existing structure is somewhat random, this tends to minimize the effectiveness of structural analysis in predicting the exact locations where damage is likely to occur under ground shaking. However, probabilistic methods based on structural analysis can be successful in indicating the general likelihood of damage, given certain levels of ground shaking. Therefore, the evaluation and design criteria contained in these *Recommended Criteria* are based on such probabilistic approaches.

Commentary: Detailed information on the types of damage discovered in various WSMF buildings following past earthquakes may be found in a companion report, FEMA-355E - State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.

2.2.4 Damage to Welded Steel Moment-Frame (WSMF) Construction in Other Earthquakes

Following the discovery of unanticipated damage to WSMF construction in the 1994 Northridge earthquake, engineers and building officials became concerned that similar, but as yet undetected damage, had occurred in WSMF buildings that had been affected by other earthquakes, such as the 1989 Loma Prieta earthquake in the San Francisco Bay Area. A concerted effort was undertaken by this project to determine the amount and extent of earthquake damage resulting from this and other recent earthquakes. Specifically, available WSMF damage information was gathered from the 1989 Loma Prieta, 1992 Landers, and 1992 Big Bear events. Unfortunately, since no mandatory inspection programs of WSMF buildings were enacted following these other earthquakes, the available data is not complete. It was, however, possible to confirm that six buildings in the San Francisco Bay Area sustained connection fractures in the Loma Prieta earthquake and one building in Big Bear, California sustained connection fractures as a result of the 1992 events. This confirms that the damage experienced in the 1994 Northridge earthquake was not a result either of unique ground shaking characteristics produced by that earthquake or of design and construction practices unique to the Los Angeles region. Further details of these investigations may be found in *FEMA-355E*.

One year to the day following the Northridge earthquake, on January 17, 1995, a magnitude 6.9 earthquake occurred near Kobe, Japan. Kobe is a large city with a population of about 1.5 million and had many WSMF structures in its building stock. These structures ranged from relatively small and low-rise buildings constructed in the 1950s and 1960s to modern high-rise structures constructed within the preceding 10 years. Design and construction practice in Japan is significantly different from common practice in the United States. Many of the smaller Japanese steel moment-frame (WSMF) structures employ cold-formed, tubular steel columns, with the beams, rather than columns, running continuously through the moment-resisting connections. In a detailed study of the damage sustained by 630 modern steel buildings in the heavily shaken area, the Building Research Institute of Japan determined that approximately one third experienced no significant damage, one third relatively minor damage, and the remaining third severe damage, including partial or total collapse of approximately half of the buildings in this remaining third (*FEMA-355E*). Just as in the United States, the Japanese believed that this damage was serious enough to warrant investment in a large program of research and development to determine the cause of the poor performance of WSMF buildings and to develop new techniques for design and construction of more reliable WSMF buildings.

2.2.5 Post-Northridge Earthquake Construction Practice

Investigation of the damage that occurred in the 1994 Northridge earthquake revealed a number of factors believed to have contributed to the poor performance of WSMF structures. These included the following:

- It was common practice to use large framing members even in relatively small buildings. Initial testing of WSMF connections, conducted in the 1960s and 1970s, utilized assemblies that employed small-sized elements, typically W18 beams and light W12 and W14 column sections. Typical buildings damaged by the Northridge earthquake employed W30 or larger beams connected to heavy W14 columns. It appears that size plays a significant role in the behavior of WSMF connections and that details that behave well for connections of small sections do not necessarily behave as well for larger sections.
- Typical detailing practice prior to the Northridge earthquake relied on the development of large inelastic behavior within the beam-column connections. This was the case even though one of the basic rules of detailing structures for superior seismic performance is to design connections of elements such that the connection is stronger than the elements themselves, so that any inelastic behavior occurs within the element and not the connection. There are several reasons for this rule. The strength and ductility of any connection is highly dependent on the quality of the workmanship employed. Connections, being relatively limited in size, must undergo extreme local yielding if they are to provide significant global ductility. The basic fabrication process for connections, employing cutting, welding, and bolting, tends to induce a complex series of effects on both the residual stress state and metallurgy of the connected parts that is often difficult to predict. Despite these common axioms of earthquake-resistant design the connections were called on for large inelastic behavior.
- Welding procedures commonly employed in the erection of WSMF buildings resulted in deposition of low-notch-toughness weld metal in the critical beam-flange-to-column-flange joints. This weld metal is subject to the initiation and development of unstable brittle fractures when subjected to high stress and strain demands and used in situations with significant geometric stress risers, or notches.

- Welding practice in many of the damaged structures was found to be sub-standard, despite the fact that quality assurance measures had been specified in the construction documents and that construction inspectors had signed documents indicating that mandatory inspections had been performed. Damaged welds commonly displayed inadequate fusion at the root of the welds as well as substantial slag inclusions and porosity. These defects resulted in ready crack initiators that enabled brittle fractures to initiate in the low-toughness weld metals.
- Detailing practice for welded joints inherently resulted in the presence of fracture-initiators. This includes failure to remove weld backing and runoff tabs from completed joints. These joint accessories often contain or obscure the presence of substandard welds. In addition, they introduce geometric conditions that are notch-like and can serve as fracture initiators.
- The presence of low-notch-toughness metal in the fillet region of some structural shapes can contribute to early fractures. The metallurgy of the material in the fillet or "k-area" region of a rolled shape often has lower notch toughness properties than material in other locations of the section due to a number of shape production factors including a relatively prolonged cooling period for this area, as well as significant cold working during shape straightening. While not normally a problem, the combined presence of weld access holes through this region at the beam-column connection and large induced stresses from buckling and yielding of the beam flanges under inelastic frame action can result in initiation of fractures in this region. These problems are made more severe by improperly cut weld access holes, which can result in sharp notches and crack initiation points. This was not a common problem in the Northridge earthquake because most connections that experienced damage did so because of other, more significant vulnerabilities. However, some of the damage that occurred to Japanese structures in the Kobe earthquake was apparently the result of these problems.
- In the 1980s, some engineers came to believe that shear yielding of the panel zones in a beam-column connection, as opposed to flexural hinging of the beam, was a more benign and desirable way to accommodate frame inelastic behavior. In response to this, in the mid-1980s the building code was modified to include provisions that allowed the design of frames with weak panel zones. Contrary to the belief that panel-zone yielding is beneficial and desirable, excessive yielding actually produces large secondary stresses at the beam-flange-to-column-flange joint, which can exacerbate the initiation of fractures.
- The yield strength of structural shape material had become highly variable. In the 1980s and 1990s, the steel production industry in the United States underwent a major realignment with new mills coming on-line and replacing older mills. Although there had always been significant variation in the mechanical properties of structural steel material, the introduction of material produced by these newer mills resulted in significant additional variation. The newer mills used scrap-based steel production, which tends to produce higher-strength material than did the older mills. In fact, much of the A36 material produced by these newer mills also met the strength requirements for the higher strength A572, Grade 50 specification. Many designers had traditionally specified A572 material for columns and A36 material for beams, in order to obtain structures economically with weak beams and strong columns. The introduction of higher strength A36 material into the market effectively negated the intent of this specification practice and often resulted in frame assemblies in which the beams were stronger than the columns or panel zones were weaker than intended,

relative to the beam strength. These combined effects resulted in greater strength demands on welded joints.

- The typical steel moment-frame beam-column connection inherently incorporated a number of stress concentrations. Although design calculations of connection capacity assume that stresses are uniformly distributed across beam flanges and that flexural stresses are carried primarily by the flanges while shear stresses are carried primarily by the web, in reality, the flange also carries significant local bending and shear stress and stresses are not uniformly distributed within the flange elements. The result of this is that large stress and strain demands occur at various locations, including the center of the weld root of the welded beam-flange-to-column-flange joint. This exacerbates the tendency of the weld defects, which are common in this region, to initiate brittle fractures in the low-notch-toughness metal. This effect is further exacerbated by the fact that the material at the center of the beam-flange-to-column-flange joint is under high tri-axial restraint. Under these conditions the material cannot yield, but rather will respond to stress in an elastic manner until the ultimate tensile strength is exceeded, at which time it initiates fracture. This problem is most severe when heavy sections are used, as the thicker material provides greater restraint.

Following the discovery of the susceptibility of typical pre-Northridge connections to fracture damage, an emergency change to the *Uniform Building Code* was adopted by the International Conference of Building Officials, removing the prequalified status of the typical bolted-web, welded-flange moment connection previously prescribed by the code and substituting in its place requirements that each connection design be qualified by a program of prototype laboratory testing. In 1994, the University of Texas at Austin engaged in a limited program of connection testing, using funding provided by the American Institute of Steel Construction and a private institution. That testing indicated that connections reinforced with cover plates to encourage the formation of plastic behavior within the span of the beam, away from the face of the columns, could provide acceptable behavior. This detail is illustrated in Figure 2-3. During the period 1994-1996 this became the most commonly specified connection type.

In the earliest connections of this type, welding was performed with electrodes that deposited material without rated notch toughness and with a wide variety of cover plate configurations. In August, 1995, *FEMA-267* was published, providing a standardized methodology for design of these connections, and the design and fabrication of these connections became more consistent. *FEMA-267* required the use of weld filler metals with rated notch toughness, and also included information on other types of connections that were believed capable of providing acceptable performance, including haunched connections, reduced-beam-section connections, vertical rib plate connections, side plate connections and slotted web connections. The recommendations contained in *FEMA-267* were based on preliminary research and were of an interim nature. While it is expected that frames constructed with connections designed using the *FEMA-267* guidelines are more resistant to connection fractures than earlier frames, it should not be assumed that they are completely free of potential for such damage.

Subsequent to the publication of *FEMA-267*, numerous other connection types have been developed and tested. For the upgrade of existing buildings, solutions utilizing connection modifications are discussed in Chapter 5 of these *Recommended Criteria* and supporting information is presented in Chapter 6, Connection Qualification.

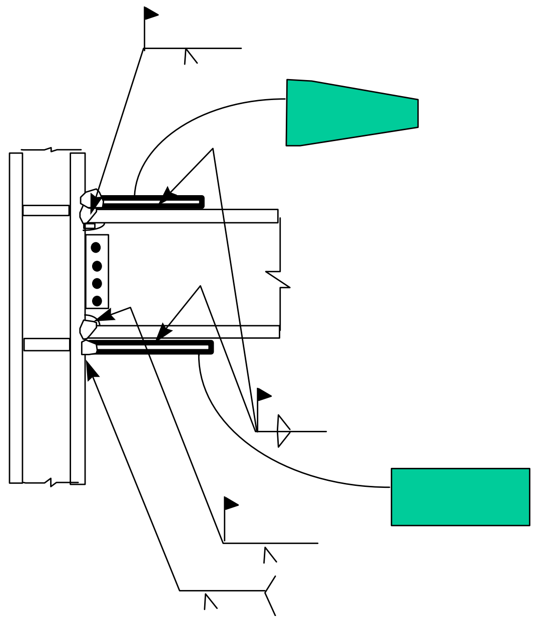


Figure 2-3 Typical Cover Plate Connection

2.3 Typical Pre-Northridge Connection Damage

Following the 1994 Northridge earthquake, damage to elements of welded steel moment frames (WSMF) was generally categorized according to a system published in *FEMA-267*. Under this system, damage is categorized as belonging to the weld (W), girder (G), column (C), panel zone (P), or shear tab (S) categories. Damage at a connection may be confined to one category or may include multiple types. The damaged WSMF may also exhibit global effects, such as permanent interstory drifts. The components of a typical pre-Northridge connection are shown in Figure 2-4.

Observation of damage sustained by buildings in the Northridge earthquake indicates that in many cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained elastic. Typically, but not always, fractures initiated at the complete joint penetration (CJP) weld between the beam bottom flange and column flange as shown in Figure 1-2. Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

In some cases, the fracture progressed completely through the thickness of the weld, and if fire protective finishes were removed, the fracture was 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 through-thickness failure 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.

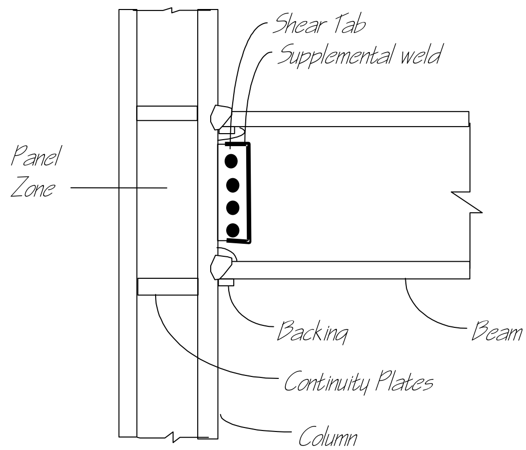


Figure 2-4 Components of Moment Connection

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.

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining flange connection and the web bolts. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of 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 extreme damage to architectural elements. The following sections detail typical damage types, using the system for categorizing damage recommended in *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings* for postearthquake damage assessment.

2.3.1 Girder Damage

Girder damage may consist of yielding, buckling or fracturing of the flanges of girders at or near the girder-column connection. Eight separate types are defined in Table 2-1. Figure 2-5 illustrates these various types of damage.

Minor yielding of girder flanges (type G2) is the least significant type of girder damage. It is often difficult to detect and may be exhibited only by local flaking of mill scale and the formation of characteristic visible lines in the material, running across the flange. If a finish, or fireproofing has been removed by scraping, the detection of this type of damage is difficult.

Girder flange yielding, without local buckling or fracture, results in negligible degradation of structural strength.

Table 2-1 Types of Girder Damage

Type	Description
G1	Buckled flange (top or bottom)
G2	Yielded flange (top or bottom)
G3	Flange fracture in heat affected zone (HAZ) (top or bottom)
G4	Flange fracture outside heat affected zone (HAZ) (top or bottom)
G5	Flange fracture top and bottom (not used)
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsional buckling of section

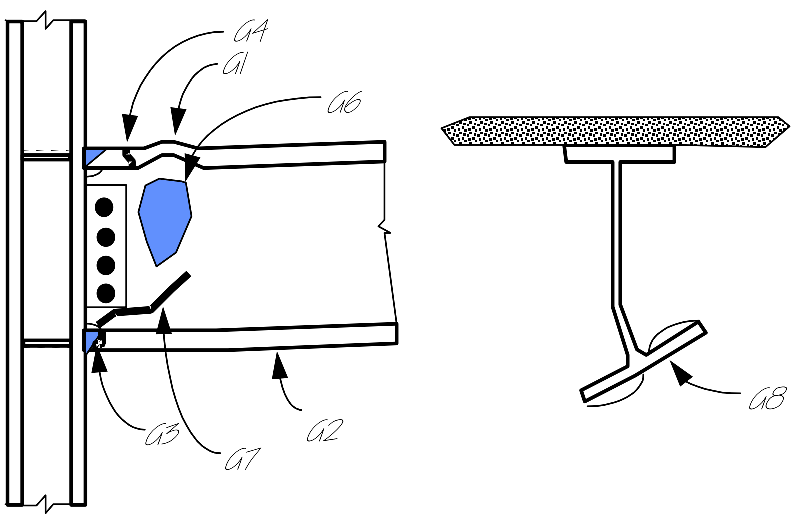


Figure 2-5 Types of Girder Damage

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength, particularly when accompanied by girder web buckling (type G-6). For compact sections, this strength loss occurs gradually, and increases with the number of inelastic cycles and the extent of the inelastic excursion. Following the initial onset of buckling, additional buckling will often occur at lower load levels and result in further reductions in strength, from the levels of previous cycles. The localized secondary stresses which occur in the girder flanges due to the buckling can result in initiation of flange fracture damage (G4) if the frame is subjected to a large number of cycles. Such fractures typically progress slowly, over repeated cycles, and grow in a ductile manner. Once this type of damage initiates, the girder flange will begin to lose tensile capacity under continued or reversed loading, although it may retain some capacity in compression.

In structures with low-toughness welds, girder flange cracking within the heat-affected zone (type G3) can occur as an extension of brittle fractures that initiate in the weld root. This is particularly likely to occur at connections in which improper welding procedures were followed, resulting in a brittle heat-affected zone. However, these fractures can also occur in connections with tough welded joints (made following appropriate procedures), as a result of low-cycle fatigue, exacerbated by the high stress concentrations that occur at the toe of the weld access hole, in unreinforced beam-column connections. Like the visually similar type G4 damage, which can also result from low-cycle fatigue conditions at the toe of the weld access hole, it results in a complete loss of flange tensile capacity, and consequently significant reduction in the contribution to frame lateral strength and stiffness from the connection.

In the 1994 Northridge earthquake, girder damage was most commonly detected at the bottom flanges, although some instances of top flange failure were also reported. There are several reasons for this. First the composite action induced by the presence of a floor slab at the girder top flange tends to shift the neutral axis of the beam towards the top flange. This results in larger tensile deformation demands on the bottom flange than on the top. In addition, the presence of the slab tends to reduce the chance of local buckling of the top flange. The bottom flange, however, being less restrained can experience buckling relatively easily.

There are a number of other factors that could lead to a greater incidence of bottom flange fractures. The location of the weld root and backing are among the most important of these. At the bottom flange joint, the backing is located at the extreme tension fiber, while at the top flange it is located at a point of lesser stress and strain demand for three reasons: (1) it is located on the inside face of the flange, (2) the local bending introduced in the flanges as a result of panel zone shear deformations, and (3) because of the presence of the floor slab. Therefore, any notch effects created by root defects and backing are more severe at the bottom flange. Another important factor is that welders can typically make the complete joint penetration groove weld at the girder top flange without obstruction, while the bottom flange weld must be made with the restriction induced by the girder web. Also the welder typically has better access to the top flange joint. Thus, top flange welds tend to be of higher quality, and have fewer stress risers, which can initiate fracture. Finally, studies have shown that inspection of the top flange weld is more likely to detect defects accurately than inspection at the bottom flange, contributing to the better quality likely to occur in top flange welds.

2.3.2 Column Flange Damage

Seven types of column flange damage are defined in Table 2-2 and illustrated in Figure 2-6. Column damage typically results in degradation of a structure's gravity-load-carrying strength as well as lateral-load resistance.

Table 2-2 Types of Column Damage

Type	Description
C1	Incipient flange crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside heat-affected zone
C4	Full or partial flange crack in heat-affected zone
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column splice failure

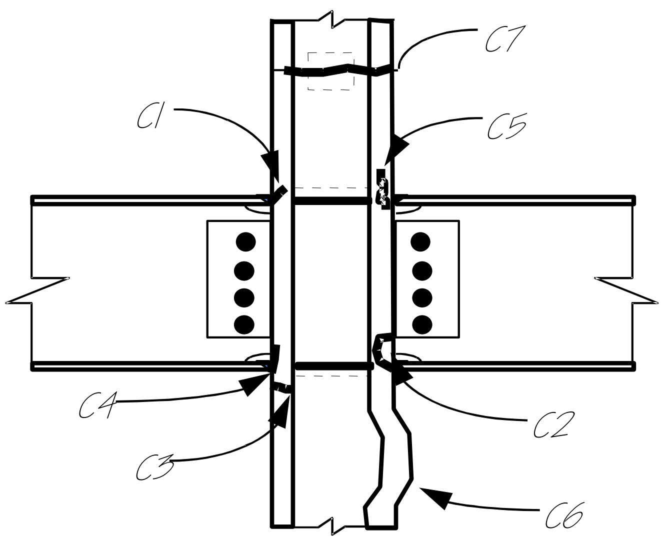


Figure 2-6 Types of Column Damage

Column flange damage includes types C1 through C7. Type C1 damage consists of a small crack within the column flange thickness, typically at the location of the adjoining girder flange. C1 damage does not go through the thickness of the column flange and can be detected only by nondestructive testing. Type C2 damage is an extension of type C1, in which a curved failure surface extends from an initiation point, usually at the root of the girder-to-column-flange weld, and extends longitudinally along the column flange. In some cases this curved failure surface may emerge on the same face of the column flange as the one where it initiated. When this occurs, a characteristic nugget or divot can be withdrawn from the flange. Types C3 and C4 fractures extend through the thickness of the column flange and may extend into the panel zone. Type C5 damage is characterized by a step-shaped failure surface within the thickness of the column flange and aligned parallel to it. This damage is often detectable only with the use of nondestructive testing.

Type C1 damage does not result in an immediate large strength loss in the column; however, such small fractures can easily progress into more serious types of damage if subjected to additional large tensile loading by aftershocks or future earthquakes. Type C2 damage results in both a loss of effective attachment of the girder flange to the column for tensile demands and a significant reduction in available column flange area for resistance of axial and flexural demands. Type C3 and C4 damage result in a loss of column flange tensile capacity and, under additional loading, can progress into other types of damage.

Type C5, lamellar tearing damage, may occur as a result of non-metallic inclusions within the column flange, particularly in older steels, when, prior to rolling, segregation of alloy inclusions was not controlled as well as in modern steels. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands has been known for a number of years and has sometimes been identified as a contributing mechanism for type C2 column flange through-thickness failures. No lamellar tearing failures were identified after the Northridge earthquake.

Type C6 damage consists of local buckling of the column flange, adjacent to the beam-column connection. While such damage was not actually observed in buildings following the 1994 Northridge earthquake, it can be anticipated at locations where plastic hinges form in the columns. Buckling of beam flanges has been observed in the laboratory at interstory drift demands in excess of 0.02 radians. Column sections are usually more compact than beams and therefore are less prone to local buckling. Type C6 damage may occur, however, in buildings with strong-beam-weak-column systems and at the bases of columns in any building when large interstory drifts have occurred.

Type C7 damage, fracturing of welded column splices, also was not observed following the Northridge earthquake. However, the partial penetration groove welds commonly used in these splices are susceptible to fracture when subjected to large tensile loads. Large tensile loads can occur on a column splice as a result of global overturning effects, or as a result of large flexural demands in the column.

2.3.3 Weld Damage, Defects, and Discontinuities

Three types of weld damage are defined in Table 2-3 and illustrated in Figure 2-7. All apply to the complete joint penetration welds between the girder flanges and the column flanges.

Type W2 fractures extend completely through the thickness of the weld metal and can be detected by either Magnetic Particle Testing (MT) or Visual Inspection (VI) techniques. Type W3 and W4 fractures occur at the zone of fusion between the weld filler metal and base material of the girder and column flanges, respectively. All three types of damage result in a loss of tensile capacity of the girder-flange-to-column-flange joint.

Table 2-3 Types of Weld Damage, Defects and Discontinuities

Type	Description
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface

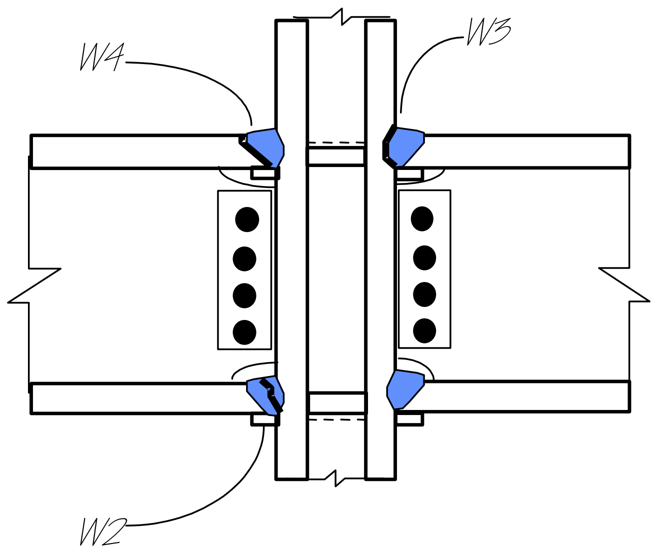


Figure 2-7 Types of Weld Damage

In addition to the W2, W3, and W4 types of damage indicated in Table 2-3 and Figure 2-7, the damage classification system presented in *FEMA-267* included conditions at the root of the complete joint penetration weld that did not propagate through the weld nor into the surrounding base metal, and could be detected only by removal of the weld backing or through the use of nondestructive testing. These conditions were termed types W1a, W1b, and W5.

As defined in *FEMA-267*, type W5 consisted of small discontinuities at the root of the weld, which, if discovered as part of a construction quality control program for new construction would not be rejectable under the *AWS D1.1* provisions. *FEMA-267* recognized that W5 conditions were likely to be the result of acceptable flaws introduced during the initial building construction, but included this classification so that it could be reported in the event that it was detected in the course of the ultrasonic testing that *FEMA-267* required. There was no requirement to repair such conditions.

Type W1a and W1b conditions, as contained in *FEMA-267* consisted of discontinuities, defects and cracks at the root of the weld that would be rejectable under the *AWS D1.1* provisions. W1a and W1b were distinguished from each other only by the size of the condition. Neither condition could be detected by visual inspection unless weld backing was removed, which, in the case of W1a conditions, would also result in removal of the original flaw or defect. At the time *FEMA-267* was published, there was considerable controversy as to whether or not the various types of W1 conditions were actually damage or just previously undetected flaws introduced during the original construction. Research conducted since publication of *FEMA-267* strongly supports the position that most, if not all, W1 damage consists of pre-existing defects, rather than earthquake damage.

2.3.4 Shear Tab Damage

Six types of damage to girder-web-to-column-flange shear tabs are defined in Table 2-4 and illustrated in Figure 2-8. Severe damage to shear tabs is often an indication that other damage has occurred to the connection, i.e., to the column, girder, panel zone, or weld.

Table 2-4 Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S2	Fracture of supplemental weld
S3	Fracture through tab at bolts or severe distortion
S4	Yielding or buckling of tab
S5	Loose, damaged or missing bolts
S6	Full length fracture of weld to column

Shear tab damage should always be considered significant, as failure of a shear tab connection can lead to loss of gravity-load-carrying capacity for the girder, and potentially partial collapse of the supported floor. Severe shear tab damage typically does not occur unless other significant damage has occurred at the connection. If the girder flange joints and adjacent base metal are sound, they prevent significant differential rotations from occurring between the column and girder. This protects the shear tab from damage, unless excessively large shear demands are experienced. If these excessive shear demands do occur, than failure of the shear tab is likely to trigger distress in the welded joints of the girder flanges.

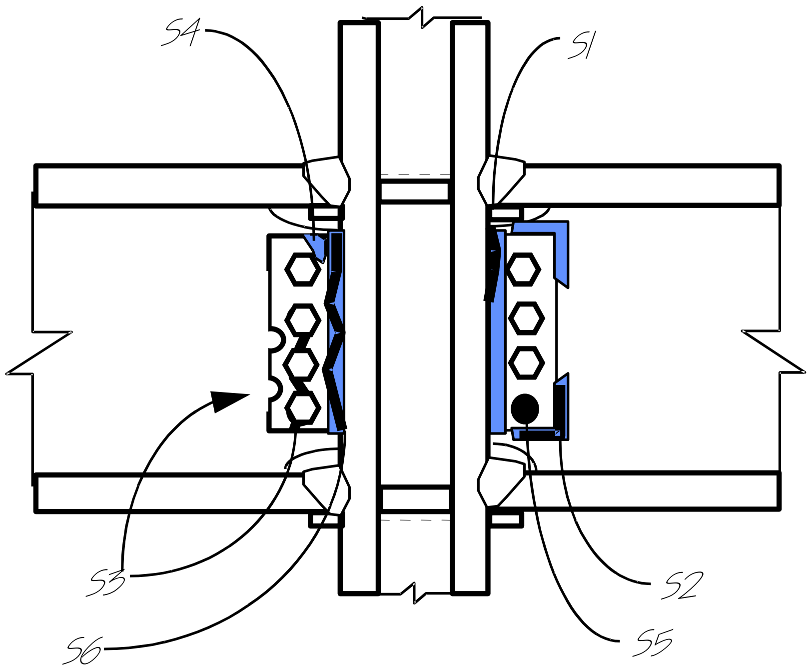


Figure 2-8 Types of Shear Tab Damage

2.3.5 Panel Zone Damage

Nine types of damage to the column web panel zone and adjacent elements are defined in Table 2-5 and illustrated in Figure 2-9. This class of damage can be among the most difficult to detect since elements of the panel zone may be obscured by beams framing into the weak axis of the column.

Fractures in the welds of continuity plates to columns (type P2), or damage consisting of fracturing, yielding, or buckling of the continuity plates themselves (type P1) may be of relatively little consequence to the structure, so long as the fracture does not extend into the column material itself. Fracture of doubler plate welds (type P4) is more significant in that this results in a loss of effectiveness of the doubler plate and the fractures may propagate into the column material.

Although shear yielding of the panel zone (type P3) is not by itself undesirable, under large deformations such shear yielding can result in kinking of the column flanges and can induce large secondary stresses in the girder-flange-to-column-flange connection. In testing conducted at the University of California at Berkeley, excessive deformation of the column panel zone was identified as a contributing cause to the initiation of type W2 fractures at the top girder flange. It is reasonable to expect that such damage could also be initiated in real buildings, under certain circumstances.

Fractures extending into the column web panel zone (types P5, P6 and P7) have the potential, under additional loading, to grow and become type P9, a complete disconnection of the upper half of the column within the panel zone from the lower half, and are therefore potentially as severe as column splice failures. When such damage has occurred, the column has lost all tensile capacity and its ability to transfer shear is severely limited. Such damage results in a total loss of reliable seismic capacity. It appears that such damage is most likely to occur in connections that are subject to column tensile loads, or in connections in which beam yield strength exceeds the yield strength of the column material.

Table 2-5 Types of Panel Zone Damage

Type	Description
P1	Fracture, buckle or yield of continuity plate
P2	Fracture in continuity plate welds
P3	Yielding or ductile deformation of web
P4	Fracture of doubler plate welds
P5	Partial depth fracture in doubler plate
P6	Partial depth fracture in web
P7	Full or near full depth fracture in web or doubler
P8	Web buckling
P9	Severed column

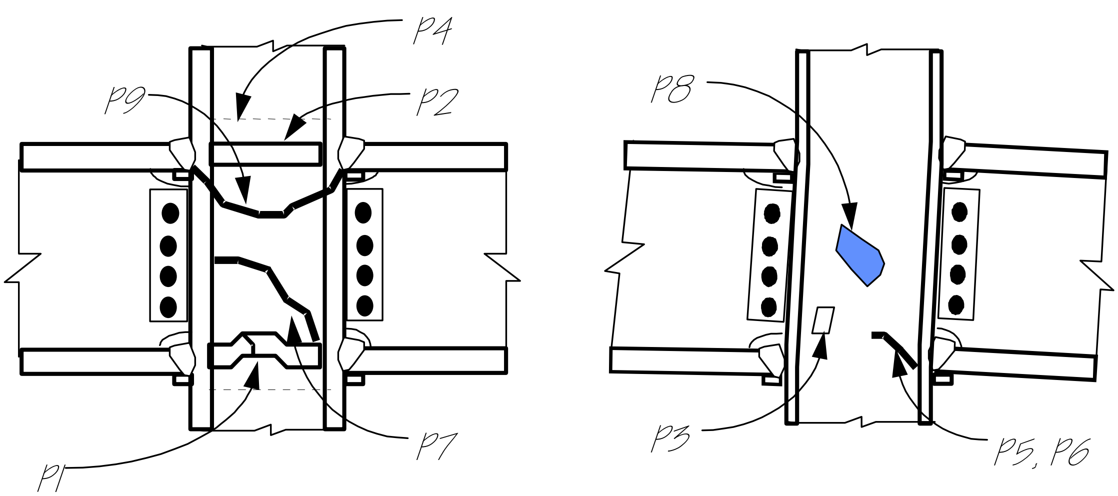


Figure 2-9 Types of Panel Zone Damage

Panel-zone web buckling (type P8) may result in rapid loss of shear stiffness of the panel zone with potential total loss of reliable seismic capacity. Such buckling is unlikely to occur in connections that are stiffened by the presence of a vertical shear tab for support of a beam framing into the column's minor axis.

2.3.6 Other Damage

In addition to the types of damage discussed in the previous sections, other types of structural damage may also be found in steel moment-frame buildings. Other framing elements which may experience damage include: (1) column base plates, beams, columns, and their connections that were not intended in the original design to participate in lateral force resistance, and (2) floor and roof diaphragms. In addition, large permanent interstory drifts may develop in the structures. Based on observations of structures affected by the 1994 Northridge earthquake, such damage is unlikely unless extensive damage has also occurred to the lateral-force-resisting system.

2.4 Evaluation Procedures

This document provides recommendations for performing several types of evaluation of the probable performance of existing steel moment-frame buildings in future earthquakes, as outlined below:

- **Performance Evaluation.** The purpose of performance evaluation is to permit estimation of a level of confidence that a structure will be able to achieve a desired performance objective (i.e., have less than a given probability of experiencing damage in excess of one or more defined limit states). In these *Recommended Criteria*, building damage is characterized in terms of two performance levels. Section 3.2.2 provides definitions of these performance levels. Once a performance objective for a building has been selected, a performance evaluation can be performed in accordance with Section 3.3 to determine a level of confidence with regard to the structure's ability to meet this performance objective. The level of confidence that can be attained with regard to the ability of a building to meet a desired performance objective is dependent on the amount of information that is available with regard to the building's configuration and construction, and the rigor of the analytical methods used in the evaluation. The performance evaluation procedures contained in Section 3.3 include simple methods for the quantification of uncertainty and confidence with regard to performance prediction of regular, well-behaved structures. More detailed methods, that permit more certain evaluation of performance capability, and which must be used for evaluation of irregular buildings are contained in Appendix A. Procedures and information regarding material properties and condition assessments to be utilized in support of the performance evaluation are presented in Section 2.5.

  Commentary: In recent years, a series of standardized building performance evaluation methodologies, including ATC-14, FEMA-154, FEMA-178 and most recently FEMA-310, have been developed. These methodologies were developed to provide the engineering community with consistent yet economical methods of determining the probable performance of different types of buildings when subjected to specific earthquake ground shaking levels. Evaluations performed in accordance with these methodologies generally consist of responding to a series of evaluation statements, intended to identify the presence of certain common *vulnerabilities, such as soft stories, weak stories, and discontinuous lateral-force-resisting systems that have been frequently observed to result in poor building performance in the past. These methodologies also commonly employ a series of analytical evaluations that include approximate evaluations of building strength and stiffness.*

While these methodologies provide good screening criteria to identify those buildings that have obvious vulnerabilities, and also serve to identify those buildings that have outstanding seismic performance characteristics, the approximate analytical procedures employed in these methods inherently incorporate so much uncertainty as to make them relatively ineffective for quantifying building performance.

Nevertheless, it is recommended that FEMA-310 be performed as a first step in the analytical evaluation of a building's probable seismic performance. Such an evaluation will provide the engineer with a basic understanding of potential critical flaws in the building configuration and provide a basis for a more detailed analytical evaluation of the building's performance, under the procedures of these Recommended Criteria.

- **Loss Evaluation**. The purpose of a loss evaluation is to determine the probable repair costs for a structure (or class of structures), if it is subjected to an earthquake hazard of defined intensity. In most loss-estimation methodologies, repair costs are expressed as a percentage of the building replacement cost. Loss-estimation evaluations sometimes include estimates of potential interruption of building occupancy as well as repair cost. Two approaches to loss estimation are provided herein: a rapid loss-estimation methodology and a detailed loss-estimation method. Rapid loss estimation, described in Chapter 4, can be quickly performed using basic data on the building's construction characteristics and specification of the intensity of ground shaking for which the loss evaluation is being performed. Detailed loss estimation requires an analytical evaluation of the building and estimation of the ground shaking response accelerations at which different damage states are likely to be exceeded. Appendix B provides information on detailed loss-estimation methods that are compatible with *HAZUS*, FEMA's nationally applicable earthquake-loss-estimation model.

Commentary: The rapid loss evaluation methodology is an approach similar to that taken in ATC-13 (ATC, 1985), in which the probability of experiencing a certain loss is related to the intensity of ground shaking experienced at the site, measured by the Modified Mercalli Intensity (MMI). Such methodologies were originally developed to estimate the probable distribution of losses for broad classes or populations of buildings. These methodologies are generally based on either actuarial statistics of the actual losses experienced by populations of buildings in past earthquakes, or on statistics related to expert opinion on the probable performance of actual buildings, or both. The methods have no direct way to account for individual building structural performance characteristics such as strength, stiffness, redundancy, or regularity, and as a result, inherently incorporate a great deal of uncertainty when applied to estimation of the loss for a specific building structure. However, in recent years, the application of these *methodologies to single building loss estimation, though technically incorrect, has become common. This application is not recommended.*

The detailed loss-estimation methodology presented in Appendix B provides for the direct consideration of structural characteristics, important to building performance, in the loss-evaluation process. In this methodology, structural analyses of the building structure are performed to characterize the probable response of the building to ground motion. Statistical data are then used to relate building response to damage and loss, at defined levels of uncertainty. The detailed loss-estimation methodology is recommended for applications in which it is desired to estimate the probable losses for a single building, as opposed to populations of buildings. It is particularly recommended as a design verification methodology for those cases when it is desired to upgrade a building to protect against future economic loss.

2.5 Material Properties and Condition Assessments

In order to perform a meaningful evaluation of either type, it is necessary to understand the structure's basic configuration, its condition, and certain basic material properties. The extent of the necessary knowledge depends on the type of evaluation and the level of certainty desired for the conclusions drawn from the evaluation. Original construction documents, including the drawings and specifications will provide sufficient data for the evaluation of most steel moment-frame buildings, so long as the building was actually constructed in accordance with these documents. As a minimum, the evaluation should include at least one visit to the building site to determine its overall condition and to confirm that available record documents are reasonably representative of the actual construction. If no construction documents are available, then extensive field surveys may be required to define the structure's configuration, including the locations of frames, the sizes of framing elements and connection details, as well as the materials of construction.

2.5.1 Material Properties

The primary material properties required to perform analytical evaluations of a steel moment-frame building include the following:

- yield strength, ultimate tensile strength and modulus of elasticity of steel for the columns in the moment frames,
- yield strength, ultimate tensile strength and modulus of elasticity of steel for the beams in the moment frames,
- ultimate tensile strength and notch toughness of the weld metal in the moment-resisting connections, and
- yield and ultimate tensile strength of bolts in the moment-resisting connections.

Although structural steel is an engineered material, there can be significant variability in the properties of the steel in a building, even if all of the members and connection elements conform to the same specifications and grades of material. Exhaustive programs of material testing to quantify the physical and chemical properties of individual beams, columns, bolts, and welds are not justified and should typically not be performed. It is only necessary to characterize the properties of material in a structure on the basis of the likely statistical distributions of the properties noted above, with mean values and coefficients of variation. Knowledge of the material specification and grade that a structural element conforms to, and its approximate age will be sufficient to define these properties for nearly all evaluations. For rapid loss-estimation evaluations, it will not be necessary to determine material properties.

In general, analytical evaluations of global building behavior are performed using expected or mean values of the material properties (based on the likely distribution of these properties) for the different grades of material present in the structure. Expected values are denoted in these procedures with the subscript “e”. Thus, the expected yield and ultimate tensile strength of steel are denoted, respectively, F_{ye} and F_{ue}. Some calculations of individual connection capacities are performed using lower-bound values of strength. Where lower-bound strength values are required, the yield and tensile strength are denoted as F_y and F_u, respectively. Lower-bound strengths are defined as the mean minus two standard deviations, based on statistical data for the particular specification and grade.

If original construction documents, including drawings and specifications, are available, and indicate in an unambiguous manner the materials of construction to be employed, it will typically not be necessary to perform materials testing in a steel moment-frame building. When material properties are not clearly indicated on the drawings and specifications, or the drawings and specifications are not available, the material grades indicated in Table 2-6 may be presumed. Alternatively, a limited program of material sample removal and testing may be conducted to confirm the likely grades of these materials.

If sampling is performed, it should take place in regions of reduced stress, such as flange tips at ends of simply supported beams, flange edges in the mid-span region of members of moment-resisting frames, and external plate edges, to minimize the effects of the reduced area. If a bolt is removed for testing, a comparable bolt should be reinstalled in its place. If coupons are removed from beams or columns, the material should either be replaced with the addition of reinforcing plate, or the area of removal should be dressed to provide smooth contours of the cutout area, without square corners or notches. Removal of a welded connection sample must be followed by repair of the connection. When sampling is performed to confirm the grades of material present in a structure, mechanical properties should be determined in the laboratory using industry standard procedures in accordance with *ASTM A-370*.

For the purpose of analytical evaluation of steel moment-frame buildings, the expected and lower bound strength of structural materials shall be taken from Table 2-7, based on the age, material specification, and grade of material.

Commentary: In general, great accuracy in the determination of the material properties of structural steel elements in steel moment-frame (WSMF) buildings is neither justified nor necessary, in order to perform reasonably reliable evaluations of building performance. The two most important parameters are the yield strengths of the beams and columns and the notch toughness of the weld metal.

Weld Filler Metal

Welding was first introduced into the building construction industry in the early 1950s. Prior to that time, most structural connection was made either by riveting or bolting. Early structural welding typically used the shielded metal arc welding (SMAW) process and "stick" filler metals with an ultimate tensile strength of 60 ksi. Although a variety of weld filler metals were available, the most commonly employed filler metal in the 1950s and early 1960s conformed to the E6012 designation. In the 1960s, as higher strength steels came on the market, there was a gradual shift to the E7024 weld filler metal, which was capable of depositing metal with a 70 ksi ultimate tensile strength. Neither of these filler metals had specific rating for notch toughness, although some welds placed with these filler metals may have considerable toughness. In the mid-1960s, contractors began to switch to the semi-automatic, flux cored arc welding (FCAW) process, which permitted more rapid deposition of weld metal and therefore, more economical construction of welded structures.

Welds in most steel moment-frame buildings constructed in the period 1964-1994 were made with the FCAW process, employing either E70T-4 or E70T-7 weld filler metal. This material generally has low notch toughness at service temperatures. Precise determination of the notch toughness of individual welds is not required in order to predict the probable poor performance of moment-resisting connections made with these materials and the typical detailing of the time. However, if weld metal with significant notch toughness (40 ft-lbs at service temperature) has been used, even connections of the type typically constructed prior to the 1994 Northridge earthquake can provide limited ductility. It is rarely possible to determine the type of weld filler metal used in a building without extraction and testing of samples. Construction drawings and specifications typically do not specify the type of weld filler metal to be employed and even when they do, contractors may make substitutions for specified materials. Welding Procedure Specifications (WPS) for a project, if available, would define the type of weld filler metal employed, but these documents are rarely available for an existing building. Given the near universal use of the FCAW process with E70T-4 or E70T-7 weld filler metal during the period 1964-1994, sampling of weld metal for buildings constructed in this period is not recommended. For buildings constructed prior to 1970, sampling and testing of weld filler metal may indicate the presence of weld with superior notch toughness, which would provide a higher level of confidence that the building would be capable of meeting desired performance objectives. Buildings constructed prior to 1964 may conservatively *be assumed to be constructed using weld filler metal with low toughness, or samples may be extracted.*

Most buildings constructed after 1996 employ weld filler metals with adequate notch toughness to provide ductile connection behavior. Sampling and testing of weld metals for buildings constructed in this period are not therefore, deemed necessary. During the period 1994-96, many different types of weld filler metal were employed in buildings. Sampling and testing of weld filler metal in buildings of this period may be advisable.

When it is deemed advisable to verify the strength and notch toughness of weld filler metals, it is recommended that at least one weld metal sample be obtained and tested for each construction type (e.g., column-splice joint, or beam-flange-to-column-flange joint). Samples should consist of both local base and weld metal, such that composite strength of the connection can be assessed. If ductility is required at or near the weld, the design professional may conservatively assume that no ductility is available in the weld, in lieu of testing.

Beams and Columns

The actual strength of beam and column elements in a steel moment-frame structure is only moderately important for the performance evaluation of such structures. The primary parameter used in these Recommended Criteria to evaluate building performance is the interstory drift induced in the building by earthquake ground shaking. Building drift is relatively insensitive to the actual yield strength of the beams and columns. However, building interstory drift can be sensitive to the relative yield strengths of beams and columns. In particular, large interstory drifts can occur in buildings with weak columns and strong beams, as such conditions permit the development of a single story mechanism in which most of the building deformation is accommodated within the single story. During the 1970s and 1980s, it was common practice in some regions for engineers to specify beams of A36 material and columns of A572, Grade 50 material in order to develop economical designs with a strong-column-weak-beam configuration. If the properties of materials employed in a steel moment-frame building are unknown, it may be conservatively assumed that the beams and columns are of the same specification and grade of material, in accordance with the default values indicated in Tables 2-6 and 2-7. However, if it can be determined that different grades of material were actually used for beams and columns, it may be possible to determine a higher level of confidence with regard to the ability of a building to meet desired performance objectives. In such cases, it may be appropriate to perform a materials sampling and testing program to confirm the material specifications for beams and columns.

When it is decided to conduct a materials testing program to confirm the specification and grade of material used in beams and columns, it is suggested that at least two strength tensile coupons should be removed from each element type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent of each grade has been established.

Bolts

Bolt specifications may be determined by reference to markings on the heads of the bolts. Where head markings are obscured, or not present, the default specifications indicated in Table 2-6 may be assumed. If a more accurate determination of bolt material is desired, a representative sample of bolts should be extracted from the building and subjected to laboratory testing to confirm the material grade.

Table 2-6 Default Material Specifications for WSMF Buildings

Element Type	Age of Construction	Default Specification
Beams and Columns	1950-1960	ASTM A7, A373
	1961-1990	ASTM A36
	1990-1998	ASTM A572, Grade 50
	1999 and later	ASTM A992
Bolts	1950-1964	ASTM A307
	1964-1999	ASTM A325
Weld Filler Metal	1950-1964	E6012, E7024 (1)
	1964-1994	E70T4 or E70T7 (2)
	1994-1999	See note 3

Notes:

1 – Prior to about 1964, field structural welding was typically performed with the Shielded Metal Arc Welding (SMAW) process using either E6012 or E7024 filler metal. Neither of these electrode classifications are rated for specific notch toughness, though some material placed using these consumables may provide as much as 40 ft-lbs or greater notch toughness at typical service temperatures. It should be noted that due to other inherent characteristics of the moment resisting connection detailing prevalent prior to the 1994 Northridge earthquake, the presence of tough filler metal does not necessarily provide for reliable ductile connection behavior.

2 – During the period 1964-1994, the Flux Cored Arc Welding (FCAW) process rapidly replaced the SMAW process for field welding in building structures. Weld filler metals typically employed for this application

conformed either to the E70T4 or E70T7 designations. Neither of these weld filler metals are rated for specific notch toughness, and both have similar mechanical properties.

3 – Following the 1994 Northridge earthquake, a wide range of weld filler metals were incorporated in WSMF construction. Most of these filler metals had minimum ultimate tensile strengths of 70ksi and minimum rated notch toughness of 20 ft-lbs at –20^{o}F. However, due to the variability of practice, particularly in the period 1994-1996, limited sampling of weld metal in structures in this era is recommended to confirm these properties.

Table 2-7 Lower Bound and Expected Material Properties for Structural Steel Shapes of Various Grades

		Yield Strength (ksi)		Tensile Strength (ksi)	
Material Specification	**Year of Construction**	**Lower Bound**	**Expected**	**Lower Bound**	**Expected**
ASTM, A7, A373	pre-1960	30	35	60	70
ASTM, A36 Group 1	1961-1990	41	51	60	70
Group 2		39	47	58	67
Group 3		36	46	58	68
Group 4		34	44	60	71
Group 5		39	47	68	80
ASTM A242, A440, A441	1960-1970				
Group 1		45	54	70	80
Group 2		41	50	67	78
Group 3		38	45	63	75
Group 4		38	45	63	75
Group 5		38	45	63	75
ASTM, A572 Group 1	1970 – 1997	47	58	62	75
Group 2		48	58	64	75
Group 3		50	57	67	77
Group 4		49	57	70	81
Group 5		50	55	79	84
A36 and Dual Grade 50	1990 – 1997				
Group 1		48	55	66	73
Group 2		48	58	67	75
Group 3		52	57	72	76
Group 4		50	54	71	76

Notes:
1. Lower bound values are mean - two standard deviations, from statistical data.
2. Expected values are mean values from statistical data.
3. For wide-flange shapes, produced prior to 1997, indicated values are representative of material extracted from the web of the section.
4. For structural plate, expected strength may be taken as 125% of the minimum specified value. Lower-bound strength should be taken as the minimum specified value.

2.5.2 Component Properties

Behavior of components, including beams and columns, is dictated by such properties as area, width-to-thickness and slenderness ratios, lateral torsional buckling resistance, and connection details. Component properties of interest are:

- original cross-sectional shape and physical dimensions,
- size and thickness of additional connected materials, including cover plates, bracing, and stiffeners,
- existing cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections,
- as-built configuration of intermediate, splice, end, and base-plate connections,
- current physical condition of base metal and connector materials, including presence of deformation.

When performing detailed evaluations and loss estimates it is necessary to conduct a structural analysis of the building's response to ground motion. Each of these properties is needed to characterize building performance in the seismic analysis. The starting point for establishing component properties should be the construction documents. Preliminary review of these documents should be performed to identify primary vertical- and lateral-load-carrying elements and systems, and their critical components and connections. In the absence of a complete set of building drawings, the design professional must obtain the necessary information on section and connection properties through a program of field investigation.

2.5.3 Condition Assessment

A condition assessment of the existing building and site conditions should be performed as part of the seismic evaluation process, regardless of the type of evaluation being performed. The goals of this assessment are:

- To examine the physical condition of primary and secondary components and the presence of any degradation.
- To verify or determine the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems.
- To review other conditions such as neighboring buildings and the presence of nonstructural components that may significantly influence building performance.

The physical condition of existing components and elements, and their connections, must be examined for presence of degradation. Degradation may influence environmental effects (e.g., corrosion, fire damage, chemical attack) or past or current loading effects (e.g., overload, damage from past earthquakes, fatigue, fracture). The condition assessment should also examine for configuration problems observed in recent earthquakes, including effects of discontinuous components, improper welding, and poor fit-up.

Component orientation, plumbness, and physical dimensions should be confirmed during an assessment. Connections in steel components, elements, and systems require special consideration and evaluation. The load path for the system must be determined, and each connection in the load path must be evaluated. This includes diaphragm-to-component and component-to-component connections.

The condition assessment also affords an opportunity to review other conditions that may influence steel elements and systems and overall building performance. Of particular importance is the identification of other elements and components that may contribute to or impair the performance of the steel system in question, including infills, neighboring buildings, and equipment attachments. Limitations posed by existing coverings, wall and ceiling space, infills, and other conditions should also be defined such that prudent rehabilitation measures may be planned.

Commentary: In order to perform reliable performance assessments of buildings, it is important to have knowledge of the existing condition of the building and its components. However, the framing in most welded steel moment-frame (WSMF) buildings construction is protected from deterioration by fireproofing and other building finishes, and therefore, most WSMF buildings will remain in good condition throughout their service lives. Unless a WSMF building has been subjected to an extreme loading event, such as a fire, extreme windstorm, or strong earthquake, or the structure exhibits signs of deterioration, such as rust stains, or lack of plumb, exhaustive condition surveys of WSMF structures are not generally justified, except as required to confirm that the construction conforms to the available construction documents.

2.5.3.1 Scope and Procedures

The scope of a condition assessment should include all primary structural elements and components involved in gravity-load and lateral-load resistance.

If coverings or other obstructions exist, indirect visual inspection through use of drilled holes and a fiberscope may be utilized. If this method is not appropriate, then local removal of covering materials may be necessary. The following guidelines should be used:

- If detailed design drawings exist, exposure of at least one different primary connection should occur for each connection type. If no deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of additional coverings from primary connections of that type must be done until the design professional has adequate knowledge to continue with the evaluation and rehabilitation.
- In the absence of construction drawings, the design professional should establish inspection protocols that will provide adequate knowledge of the building needed for reliable evaluation.

Physical condition of components and connectors may also dictate the use of certain destructive and nondestructive test methods. If steel elements are covered by well-bonded fireproofing materials or encased in durable concrete, it is likely that their condition will be suitable. However, local removal effort is dictated by the component and element design. It may be necessary to expose more connections because of varying designs and the critical nature of the connections.

2.5.3.2 Quantifying Results

The results of the condition assessment should be used in the preparation of building system analytical models for the evaluation of seismic performance. To aid in this effort, the results should be quantified and reduced with the following specific topics addressed:

- component section properties and dimensions,
- connection configuration and presence of any eccentricities,
- type and location of column splices, and
- interaction of nonstructural components and their involvement in lateral-load resistance.

All deviations noted between available construction records and as-built conditions should be accounted for and considered in the structural analysis.

3. Performance Evaluation

3.1 Scope

This chapter provides simplified criteria for evaluating the probable seismic performance of existing welded steel moment-frame buildings. These procedures may be used to quantify the ability of a building to achieve desired performance objectives, either before or after the construction of structural upgrades. It includes definition 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*, and procedures for estimating a level of confidence that a building will 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 recommendations 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 these Recommended Criteria. 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.

3.2 Performance Definitions

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 seismic upgrade design may be intended to provide a 95% level of confidence that a structure provide Collapse Prevention or better performance for earthquake hazards with a 2% probability of exceedance in 50 years, or a 50% confidence level that a structure provide Immediate Occupancy or better performance, for earthquake hazards with a 50% probability of exceedance in 50 years. The user may determine the level of confidence associated with achieving any desired performance objective.

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 can not 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 very 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 3.2.1.2 for additional discussion of the probabilistic approach adopted by these Recommended Criteria.

3.2.1 Hazards

3.2.1.1 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.

3.2.1.2 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 provided with the *FEMA-302 NEHRP Recommended Provisions* and the *FEMA-273 NEHRP Rehabilitation Guidelines* have been prepared based on hazard curves that have been developed by the United States Geological 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 for 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 ground shaking will vary depending on regional and site seismicity. In areas of low *seismicity the hazard 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 *Recommended Criteria* 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 to Section 3.6. 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 Seismic Rehabilitation of Buildings and references noted therein.

Although Section 3.2 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 shaking with a 2% probability of exceedance in 50 years" should more correctly be stated as being "less than a 2% chance in 50 years of damage more severe than the collapse prevention level, given the mean definition of seismicity."

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 associate 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.

3.2.1.3 Other Hazards

In order to predict reliably the probable performance of a structure, 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 which 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 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 this publication.

3.2.2 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 3-1 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 3.2.2.2.

Table 3-1 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-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-I buildings when subjected to the Maximum Considered Earthquake	Same level of performance	Much less damage and lower risk.
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 WMSF 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.

3.2.2.1 Nonstructural Performance Levels

These *Recommended Criteria* only addresses 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 these *Recommended Criteria*. *FEMA-273* provides a more complete set of recommendations with regard to evaluating the performance of nonstructural components.

3.2.2.2 Structural Performance Levels

Two discrete structural performance levels, Collapse Prevention and Immediate Occupancy are defined in these *Recommended Criteria*. Table 3-2 relates these structural performance levels to the limiting damage states for framing elements of steel moment-frame structures. Acceptance criteria, which relate to the permissible interstory drifts and earthquake-induced forces for the various elements of steel moment-frame structures, 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 3-2 Structural Performance Levels

		Structural Performance Levels	
Elements	**Type**	**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	Connection Type 1[1]	Some 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
	Connection Type 2[1]	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
Drift	Interstory	Large permanent	Less than 1% permanent

Notes: 1 Connection types are defined in Section 3.6.2.1, Table 3-9.

3.2.2.2.1 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.

3.2.2.2.2 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 (1), strain a number of elements to a *point at which the elements degrade in stiffness and strength, or (2), as a result of P-Δ 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.

3.3 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 hazard 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, and
- axial forces on individual columns.

These structural response parameters are related to the amount of damage experienced by individual structural components as well as to the structure as a whole. For each performance level, these *Recommended Criteria* specify acceptance criteria (median estimates of capacity) for all the design parameters indicated above. Acceptability of structural performance is evaluated considering both local performance (at the 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 incorporating the variation inherent in structural response, such that a confidence level can be established with regard to the ability of a structure to provide specific performance at selected, low, probabilities of exceedance.

Once an analysis is performed, predicted demands are adjusted by two factors, an analytical uncertainty factor γ_a, which corrects the analytically predicted demands for bias and uncertainty inherent in the analytical technique, and a demand variability factor, γ, 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 ϕ to account for uncertainties and variation inherent in structural capacity prediction. Procedures are given to calculate the level of confidence provided by a seismic evaluation or upgrade design, to achieve a specific performance objective, based on the ratio of factored demand to factored capacity. If the predicted level of confidence is inadequate, then either more detailed investigations and 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 behavior and modification of the demand and resistance factors, or the structure should be upgraded such that a sufficient level of confidence can be attained given the level of understanding. If it is deemed appropriate to upgrade a structure 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 upgrade design solution is found. Procedures for estimating confidence are contained in Section 3.6.

Commentary: These procedures adopt a demand and resistance factor design (DRFD) model for performance evaluation. This approach is similar to the Load and Resistance Factor design 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 these procedures 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 with regard to a specific performance objective.

The factored interstory drift demand, calculated from the analysis represents a median estimate of the probable maximum interstory drift demand, at the desired probability of exceedance. 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 ϕ, that adjust the estimated capacity of the structure to median values. Appendix A provides procedures for determination of ϕ 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 λ is calculated from the ratio of the factored demands and capacities as indicated in Section 3.6. The value of λ 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 demand and capacities.

3.4 Analysis

In order to evaluate the performance of a welded 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 3.5.

3.4.1 Alternative Procedures

Four alternative analytical procedures are available for use in performance evaluation of welded 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 structures 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 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 (the appropriate level of confidence of achieving the desired performance) using the basic approach outlined in Section 3.6.

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 building. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for buildings that exhibit significant nonlinear behavior, particularly if the buildings 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 building in response to ground motion. However, there are considerable uncertainties associated with the values of the response parameters predicted by this technique.*

3.4.2 Procedure Selection

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

3.4.3 Linear Static Procedure

3.4.3.1 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 3.6.

Table 3-3 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 Conditions	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 than those defined in *FEMA-302*.

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

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 building 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, (so named in FEMA-273) 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. However, 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.

The performance of welded steel moment-frame buildings is most closely related to the total inelastic deformation demands on the various seismic 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 welded steel moment-frame buildings 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 LSP is based on the assumption that the distribution of deformations predicted by an elastic analysis where all members remain linear elastic during all loadings, is similar to the distribution of deformations that will occur in actual nonlinear response. This assumption is inaccurate and can become more so for buildings 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 buildings and buildings with relatively long fundamental periods of vibration.

3.4.3.2 Period Determination

The fundamental period for each of the two orthogonal directions of building response shall be calculated 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:

$$T = C_t h_n^{0.8} \tag{3-1}$$

where

T = fundamental period (in seconds) in the direction under consideration,
C_t = 0.028 for steel moment frames,
h_n = height (in feet) of 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:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \tag{3-2}$$

where Δ_w and Δ_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 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+ 1 σ) that the actual period will exceed the calculated value. The formula has intentionally been selected to underestimate 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 3.4.3.3.1). 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.

3.4.3.3 Determination of Actions and Deformations

3.4.3.3.1 Pseudo Lateral Load

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

$$V = C_1 C_2 C_3 S_a W \tag{3-3}$$

where:

C_1 = 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 acceleration response segment of the spectrum to the constant spectral velocity response 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-frame structures the value of C_2 shall be taken as 1.0.

C_3 = modification factor to represent increased dynamic displacements due to *P-Δ* effects and stiffness degradation. C_3 may be taken from Table 3-4 or shall be calculated from the equation:

$$C_3 = 1 + \frac{5(\theta_i - 0.1)}{T} \geq 1.0 \tag{3-4}$$

where:

θ_i = the coefficient determined in accordance with Section 3.2.5.1 of *FEMA-273*.

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 the *NEHRP Recommended Provisions*,
- the total weight of permanent equipment and furnishings.

Commentary: The pseudo lateral force, when distributed over the height of the linearly-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 (3-3) may be significantly larger than the actual strength of the structure to resist this force. The acceptance evaluation procedures in Section 3.6 are developed to take this into account.

The values of the C_3 coefficient contained in Table 3-4 are conservative for most structures, and will generally result in calculation of an unduly low level of confidence. Use of Equation 3-4 to calculate C_3 is one way to improve calculated confidence without extensive additional effort, and is recommended.

Table 3-4 Modification Factors C_3 for Linear Static Procedure

Performance Level	C_3
Immediate Occupancy	1.0
Collapse Prevention	
Type 1[1] FR connections	1.2
Type 2[2] FR connections	1.4

Notes:
1. Type 1 connections are capable of resisting median total drift angle demands of 0.04 radians without fracture or strength degradation.
2. Type 2 connections are capable of resisting median total drift angle demands of 0.01 radians without fracture or strength degradation. Generally, welded unreinforced connections, employing weld metal with low notch toughness, typical of older steel moment-frame buildings should be considered to be of this type.

3.4.3.3.2 Vertical Distribution of Seismic Forces

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

3.4.3.3.3 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.

3.4.3.3.4 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.

3.4.3.3.5 Determination of Interstory Drift

Interstory drifts shall be calculated using lateral loads calculated in accordance with Section 3.4.3.3.1 and stiffness obtained from Section 3.5. Factored interstory drift demands $\gamma_a \gamma \delta_i$ at each story i, shall be determined by applying the appropriate analysis uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6.2.

3.4.3.3.6 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 γ_a and demand variability factor γ 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:

$$P'_c = \pm\left[2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_L - 2\left(\sum_{i=x}^{n}\frac{M_{pe}}{L}\right)_R\right] \tag{3-5}$$

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 moment-connected 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 moment-connected 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.

3.4.4 Linear Dynamic Procedure

3.4.4.1 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 3.4.3.3 of these *Recommended Criteria*.

Estimates of interstory drift and column axial demands shall be evaluated using the applicable procedures of Section 3.6. Calculated displacements and column axial demands are factored by the applicable analytical uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6 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 inelastic response of components and elements, but are generally not used to evaluate performance.

Commentary: The linear dynamic procedure is similar in approach to the linear static procedure, described in Section 3.4.3. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, it 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. Under the linear dynamic procedure, inertial seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly 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, linearly-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.

3.4.4.2 Analysis

3.4.4.2.1 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 3.4.4.3, 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.

3.4.4.2.2 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.

3.4.4.3 Determination of Actions and Deformations

3.4.4.3.1 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 3.4.3 and by the applicable analytical uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6.2.

3.4.4.3.2 Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces, as given in Section 3.4.3.3.6, by the applicable analysis uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6.3.

3.4.5 Nonlinear Static Procedure

3.4.5.1 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 increased lateral forces or displacements until either a target displacement is exceeded or 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 3.3.3.3A of *FEMA-273* with modifications, as 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 3.6. Calculated interstory drifts, column forces, and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

Commentary: The nonlinear static analysis approach inherently assumes that behavior is dominated by the first mode response of the structure. For this reason, this approach should 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 United States sites, tend to have most of their energy at very short periods, particularly on firm soil and rock sites. For sites subject to such shaking, nonlinear static analyses may be valid only for very short and rigid structures. The limitations on the use of NSP, based on period, contained in Table 3-3, are based on recent work that indicates that higher mode response does not tend to become significant in structures responding to ground shaking with 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 acceleration response to constant velocity response.

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 criteria have adapted standard approaches used in linear analysis for this purpose.

3.4.5.2 Analysis Considerations

3.4.5.2.1 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 δ_t given by Equation 3-11 of *FEMA-273*. Performance evaluation shall be based on those column forces and interstory drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement δ_t corresponding to the hazard level (probability of exceedance) appropriate to the performance objective being evaluated.

Gravity loads shall be applied to appropriate elements and components of the mathematical model during the 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.

3.4.5.2.2 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.

3.4.5.2.3 Lateral Load Patterns

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

3.4.5.2.4 Period Determination

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

3.4.5.2.5 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 3.4.5.3.4 of these *Recommended Criteria*.

3.4.5.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) 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*.

3.4.5.2.7 Connection Modeling

Existing, fully restrained, unimproved welded moment-resisting connections should be modeled as indicated in Section 6.2.1.2 of these *Recommended Criteria*. Simple shear tab connections with slabs present should be modeled as indicated in Section 6.2.2.1.2. Improved or upgraded fully restrained moment-resisting connections should be modeled as for unimproved connections except that the quantity θ_{SD} should be as indicated in Chapter 6 for the applicable connection type.

3.4.5.3 Determination of Actions and Deformations

3.4.5.3.1 Target Displacement

The target displacement, δ_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 performance evaluation in accordance with Section 3.6.

3.4.5.3.2 Diaphragms

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

3.4.5.3.3 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 analytical uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6.2.

3.4.5.3.4 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 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 30% of the loading applied in direction A and 100% in direction B.

3.4.5.3.5 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 analytical uncertainty factor γ_a and demand variability factor, γ, from Section 3.6.3.

3.4.6 Nonlinear Dynamic Procedure

3.4.6.1 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 3.6. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values for the applicable performance level.

3.4.6.2 Analysis Assumptions

3.4.6.2.1 General

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

3.4.6.2.2 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*.

3.4.6.3 Determination of Actions and Deformations

3.4.6.3.1 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.

3.4.6.3.2 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum of the interstory drifts calculated in accordance with Section 3.4.6.3.1 by the applicable analytical uncertainty factor γ_a and demand variability factor γ obtained from Section 3.6.2.

3.4.6.3.3 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 3.4.6.3.1 by the applicable analytical uncertainty factor γ_a, and demand variability factor γ from Section 3.6.3 or 3.6.4.

3.5 Mathematical Modeling

3.5.1 Basic Assumptions

In general, a steel moment-frame structure 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 are 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 behavior is not required for linear analysis procedures. 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.

> *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.*

3.5.2 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 6 for typical connections.

3.5.2.1 Elements Modeled

Only the beams and columns forming the lateral-force-resisting system need be modeled, although 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. Refer to Chapter 6 for procedures for modeling common gravity-load beam-column connections.

> *Commentary: Typically, engineers modeling steel moment-frame buildings 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 recommendations 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.

While it is permissible to conduct performance evaluations using models that neglect non-participating 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.

3.5.2.2 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 flexibility 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 tend to estimate conservatively the interstory drift demand on the structure. While it is permissible to conduct performance evaluations using models that neglect the effect of the panel zone on beam and column stiffness, models that include more realistic estimation of this effect 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 idealize the panel zone as a scissors-type model that accounts explicitly for the shear stiffness of the panel zone, and to complex multi-element models that accounts 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.

3.5.3 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:

1. 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

2. 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 δ_{max} from this effect at any point on any floor diaphragm exceeds the average displacement δ_{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 indirectly account for the uncertainty related to these torsional effects in the calculation of demand and resistance factors.

3.5.4 Foundation Modeling

In general, foundations may 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.

3.5.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic-force-resisting system. Connections between the edge beams of 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.

3.5.6 *P-Δ* Effects

P-Δ effects, caused by gravity loads acting on the displaced configuration of the building, 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 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 Ψ_i should be calculated for each direction of response, as follows:

$$\Psi_i = \frac{P_i \delta_i}{V_{yi} h_i} \tag{3–5}$$

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,*

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 the distance between the centerline of floor framing at each of the levels above and below, the distance between the top of floor slabs at each of the levels above and below, or similar common points of reference, and

δ_i = lateral interstory drift in story *i*, from the analysis in the direction under consideration, at its center of rigidity, using the same units as for measuring h_i.

In any story in which Ψ_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity Ψ_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by *P-Δ* effects. When

Ψ_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-Δ* effects, is conducted in accordance with the guidelines of Appendix A.

For nonlinear 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.

Commentary: The values of interstory drift capacity for the Collapse Prevention performance level, provided in Section 3.6, and the corresponding resistance factors, were computed considering P-Δ effects (FEMA-355F). For a given structure, it is believed that if the value of Ψ is less than 0.3 the effects of P-Δ have been adequately considered by these general procedures. For values of Ψ greater than this limit the statistics on frame interstory drift capacities contained in Section 3.6 are inappropriate. For such frames explicit determination of interstory drift capacities, considering P-Δ effects using the detailed Performance Evaluation procedures outlined in Appendix A is required.

The plastic story-shear quantity, V_{yi} should be determined by methods of plastic analysis. In a story in which: (1) all beam-column connections meet the strong-column-weak-beam criterion, (2) the same number of moment-resisting bays is present at the top and bottom of the frame, and (3) the strength of moment-connected girders at the top and bottom of the frame is similar, V_{yi} may be approximately calculated from the equation:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG\,j}}{h_i} \tag{3-6}$$

where:

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

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

In any story in which none of the columns meets the strong-column-weak-beam criterion, the plastic story-shear quantity, V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{n} M_{pC_k}}{h_i} \qquad (3\text{-}7)$$

where:

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.

3.5.7 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 3.4 for the various analytical procedures.

3.5.8 Vertical Ground Motion

The effects of vertical excitation on horizontal cantilevers may be considered either 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 vertical components 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 the 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.

3.6 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 3-5 must be independently evaluated, using the procedures of Section 3.6.1 and the parameters and acceptance criteria of Sections 3.6.2, 3.6.3, and 3.6.4, for each performance objective evaluated. The controlling parameter is that which results in the calculation of the lowest confidence for building performance.

Table 3-5 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 behavior (for Collapse Prevention and Immediate Occupancy). Refer to Section 3.6.2
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 3.6.3
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 3.6.4

3.6.1 Factored-Demand-to-Capacity Ratio

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

$$\lambda = \frac{\gamma \gamma_a D}{\phi C} \tag{3-8}$$

where:

C = capacity of the structure, as indicated in Sections 3.6.2, 3.6.3, and 3.6.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

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

γ = 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 3.6.2, 3.6.3, and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

γ_a = an analytical 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 3.6.2, 3.6.3 and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

ϕ = 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 3.6.2, 3.6.3 and 3.6.4 for interstory drift demand, column compressive demand and column splice tensile demand, respectively.

λ = a confidence index parameter from which a level of confidence can be obtained. See Table 3-6.

Factored-demand-to-capacity ratio λ shall be calculated using equation 3-8 for each of the performance parameters indicated in Table 3-5, which also references the appropriate section of these *Recommended Criteria* where the various parameters, γ_a, γ, ϕ required to perform this evaluation may be found. These referenced sections also define an uncertainty parameter β_{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 type of moment-resisting connections present (type 1 or type 2), the type of analytical procedure employed and the performance level being evaluated. Table 3-6 indicates the level of confidence associated with various values of the factored-demand-to-capacity ratio λ calculated using Equation 3-8, for various values of the uncertainty parameter β_{UT}. Linear interpolation between the values given in Table 3-6 may be used for values of factored-demand-to-capacity ratio λ and uncertainty β_{UT} intermediate to those tabulated.

Table 3-6 Factored-Demand-to-Capacity Ratios λ for Specific Confidence Levels and Uncertainty β_{UT} factors

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

Table 3-7 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.

Table 3-7 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%

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 predict accurately 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 is likely to represent reality.

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 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, β_U. The coefficient, β_{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. We assume that demand and capacity are lognormally distributed relative to these uncertainty parameters. This allows confidence to be calculated as a function of the number of standard deviations that factored-

demand-to-capacity-ratio λ lies above or below a mean value. Table 3-6 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.

3.6.2 Performance Limited by Interstory Drift Angle

3.6.2.1 Factored Interstory Drift Angle Demand

Factored interstory drift demand should be computed as the quantity $\gamma\gamma_a D$ where the demand D, is the largest interstory drift in any story computed from structural analysis, γ_a is the coefficient obtained from Table 3-8, and γ is the coefficient obtained from Table 3-9.

Table 3-8 Interstory Drift Angle Analysis Uncertainty Factors, γ_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]
Type 1 Connections								
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
Type 2 Connections								
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

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-Δ 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 different rows of 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 should be neglected in determining local connection performance. This portion of the interstory drift must usually be determined manually as most computer programs do not calculate this quantity separately.*

Table 3-9 Interstory Drift Angle Demand Variability Factors γ

Building Height	Immediate Occupancy (I.O.)	Collapse Prevention (C.P.)
Type 1 Connections[1]		
Low Rise (3 stories or less)	1.5	1.3
Mid Rise (4-12 stories)	1.4	1.2
High Rise (more than 12 stories)	1.4	1.5
Type 2 Connections[2]		
Low Rise (3 stories or less)	1.4	1.4
Mid Rise (4-12 stories)	1.3	1.5
High Rise (more than 12 stories)	1.6	1.8

Notes:

1- Type 1 connections are capable of resisting median total drift angle demands of 0.04 radians without fracture or strength degradation.
2- Type 2 connections are capable of resisting median total drift angle demands of 0.01 radians without fracture or strength degradation. Generally, welded unreinforced connections, employing weld metal with low notch toughness, typical of older welded steel moment-frame buildings should be considered to be this type.

3.6.2.2 Factored Interstory Drift Angle Capacity

Interstory drift capacity may be limited either by the global response of the structure, or by the local behavior of beam-column connections. Section 3.6.2.2.1 provides values for global interstory drift capacity for regular, well-configured structures as well as associated uncertainties, β_{UT}. Global interstory drift capacities for irregular structures must be determined using the detailed procedures of Appendix A. Section 3.6.2.2.2 provides procedures for evaluating local interstory drift angle capacity, as limited by connection behavior.

3.6.2.2.1 Global Interstory Drift Angle

Factored interstory drift angle capacity, ϕC, as limited by global response of the building, shall be based on the product of the resistance factor ϕ and capacity C, which are obtained from Table 3-10, for either Type 1 or Type 2 connections. Type 1 connections are capable of resisting median total interstory drift angle demands of 0.04 radians without fracturing or strength degradation. Type 2 connections are capable of resisting median total interstory drift angle demands of 0.01 radian without fracturing or strength degradation. Welded unreinforced moment-resisting connections with weld metal with low notch toughness should be considered Type 2. Table 3-11 provides values of the uncertainty coefficient β_{UT} to be used with global interstory drift evaluation.

Table 3-10 Global Interstory Drift Angle Capacity *C* and Resistance Factors ϕ for Regular Buildings

Building Height	Performance Level			
	Immediate Occupancy		Collapse Prevention	
	Interstory Drift Angle Capacity *C*	Resistance Factor ϕ	Interstory Drift Angle Capacity *C*	Resistance Factor ϕ
Type 1 Connections				
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
Type 2 Connections				
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

3.6.2.2.2 Local Interstory Drift Angle

Factored interstory drift angle ϕC limited by local connection response, shall be based on the capacity C of the connection and resistance factor ϕ obtained from Chapter 6 of these *Recommended Criteria*. For Immediate Occupancy performance, capacity C shall be taken as the quantity θ_{IO} and for Collapse Prevention performance, the quantity θ_U indicated in Chapter 6 for the connection types present in the building. For connection types not include in Chapter 6, the capacity and resistance factors should be obtained from laboratory testing and the procedures of Appendix A. Table 3-12 provides values of the uncertainty coefficient β_{UT} for use in evaluating performance as limited by local connection behavior.

Table 3-11 Uncertainty Coefficient β_{UT} for Global Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Type 1 Connections		
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
Type 2 Connections		
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 β_{UT} should be increased by 0.05 for LSP analysis
2- Value of β_{UT} may be reduced by 0.05 for NDP analysis

Table 3-12 Uncertainty Coefficient β_{UT} for Local Interstory Drift Evaluation

Building Height	Performance Level	
	Immediate Occupancy	Collapse Prevention
Type 1 Connections		
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
Type 2 Connections		
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 β_{UT} should be increased by 0.05 for LSP analysis
2- Value of β_{UT} may be reduced by 0.05 for NDP analysis

3.6.3 Performance Limited by Column Compressive Capacity

3.6.3.1 Column Compressive Demand

Factored column compressive demand shall be determined for each column as the quantity $\gamma\gamma_a D$, where:

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

Linear: The axial demands may be taken as those predicted by a linear static or linear dynamic analysis, conducted in accordance with Section 3.4.3 or 3.4.4.

Plastic: The axial seismic demands may be taken from plastic analysis, as indicated by Equation 3-5 in Section 3.4.3.3.6.

Nonlinear Static: The axial demands may be taken from the computed forces from a nonlinear static analysis, at the target displacement, in accordance with Section 3.4.5.

Nonlinear Dynamic: The axial demands may be taken from the computed design forces from a nonlinear dynamic analysis, in accordance with Section 3.4.6.

γ_a = analytical uncertainty factor, taken from Table 3-13.

γ = demand variability demand factor, taken as having a value of 1.05.

The uncertainty coefficient β_{UT} shall be taken as indicated in Table 3-13 based on the procedure used to calculate column compressive demand *D*.

Table 3-13 Analysis Uncertainty Factor γ_a and Total Uncertainty Coefficient β_{UT} for Evaluation of Column Compressive Demands

Analytical Procedure	Analysis Uncertainty Factor γ_a	Total Uncertainty Coefficient β_{UT}
Linear Static or Dynamic Analysis	1.15	0.35
Plastic Analysis (Section 3.4.3.3.6)	1.0	0.15
Nonlinear Static Analysis	1.05	0.20
Nonlinear Dynamic Analysis	$e^{1.4\beta^2}$	$\sqrt{0.0225+\beta^2}$

Note: β 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 γ 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 γ_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. 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, which is typical of conditions in coastal California.*

3.6.3.2 Column Compressive Capacity

Factored compressive capacity of each individual column to resist compressive axial loads shall be determined as the product of the resistance factor, ϕ, and the nominal axial strength C of the column, which shall be determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, 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 ϕ shall be assigned a value of 0.95.

3.6.4 Column Splice Capacity

The capacity of column tensile splices, other than splices consisting of complete joint penetration (CJP) butt 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 Sections 6.4.2.4 and 6.4.2.5 of these *Recommended Criteria* need not be evaluated.

3.6.4.1 Column Splice Tensile Demand

Factored column splice tensile demand shall be determined for each column as the quantity $\gamma\gamma_a D$ 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 γ shall be taken as having a value of 1.05 and the analysis uncertainty factor γ_a shall be taken as indicated in Table 3-13. The total uncertainty coefficient β_{UT} shall also be taken as indicated in Table 3-13.

3.6.4.2 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, ϕ, and the nominal tensile strength of the splice, C, as determined in accordance with the *AISC Load and Resistance Factor Design Specification*. Specifically, Chapter J of the *AISC Specification* shall be used to calculate the nominal tensile strength of the splice connection. For the purposes of this evaluation, ϕ shall be assigned a value of 0.9.

4. Loss Estimation

4.1 Scope

This chapter provides data that may be used to perform estimates of probable repair costs for steel moment-frame buildings based on actuarial data obtained from the 1994 Northridge earthquake. These data may be used to estimate the cost of repair for these buildings, within levels of confidence, given limited data on the building characteristics and an estimate of ground motion intensity. A more detailed approach that incorporates the information obtained from a structural analysis of the building is contained in Appendix B of these *Recommended Criteria*.

When an earthquake damages a building, there are a number of potential sources of economic loss. One source of losses is the cost associated with repairing the damage and restoring the building to service. Such losses are known as direct loss. Other sources of economic loss can result from an inability to occupy space in the damaged structure until it is repaired, the need to rent space for temporary or alternative quarters, relocation costs, litigation, devaluation of property values and a general decline in the economic welfare of the affected region. These losses are generally termed indirect losses. These *Recommended Criteria* provide methods only for estimating direct losses due to earthquake ground shaking.

The direct losses that can be estimated using the methods of these *Recommended Criteria* typically represent only a small portion of the total losses caused by earthquakes. The other indirect losses are a function of a number of complex factors that relate to the economic and social makeup of the affected region, and the decision making process performed by individual owners and tenants and go far beyond the considerations of damage sustained by individual buildings. Therefore, although such losses are very important, they are considered to be beyond the scope of these *Recommended Criteria*.

Although the tools presented herein can be applied to building specific loss estimates, they were originally developed with the intent of application to broad populations of buildings. The estimation of losses that may occur to a specific building in future earthquakes of unknown source, magnitude and distance is fraught with great uncertainty. Users are cautioned that actual performance of specific buildings in response to specific earthquake demands can be substantially different from what would be suggested by the statistically based methods presented in this chapter.

4.2 Loss Estimation Methods

Two alternative methods are provided in these *Recommended Criteria* to estimate probable repair costs for buildings due to future earthquake ground motion. The Rapid Loss Estimation Method, contained in this chapter, provides estimates of losses as a function of basic information about the building and estimates of seismic demands. The Detailed Loss Estimation Method found in Appendix B, directly utilizes engineering data obtained from a detailed structural analysis of the specific building. This Detailed Loss Estimation Method is compatible with FEMA's *HAZUS* (NIBS, 1997a,b) loss estimation software and can be used to generate building-specific vulnerability information for use with that system and other similar loss estimation models.

Commentary: The most common bases for producing loss estimates may be classified as historical experience, expert opinion, and engineering. All of these methods include significant uncertainty with regard to predicted damage and repair costs.

Historical experience-based estimates are developed based on statistics on the actual damage and costs incurred for given classes of structures subjected to estimated or recorded seismic demands. When such data are available, it is possible to determine the distribution of losses over the population contained in the database, including a median (best estimate of the loss for any structure in the class) and a measure of variation. This permits the loss for a structure similar to those in the database to be estimated within a range of confidence. Significant sources of uncertainty include the lack of database completeness, differences between the structure being evaluated and the general population in the database, and the seismic demand range captured in the database. No database is comprehensive.

The most commonly used loss estimation methodology is based on expert opinion of probable repair costs. ATC-13 (ATC, 1985) and other similar studies have developed damage functions by obtaining opinions from structural engineers and other experts on typical levels of damage for various classes of structures when subjected to different intensities of ground motion. Statistical data from such opinion surveys can then be used to derive loss estimates for other buildings. This approach also has much uncertainty and little to no direct tie to actual losses experienced in past events, other than as perceived by the experts. The 1994 Northridge earthquake illustrates the uncertainty inherent in expert opinion, in that the brittle fractures that occurred in steel moment-frame buildings had not previously been anticipated.

The Rapid Loss Estimation Method presented here uses both historical experience and expert opinion. A database of steel moment-frame building damage caused by the 1994 Northridge earthquake represented the historical experience. This was augmented by expert opinion where actuarial data were sparse or unreliable.

The Detailed Loss Estimation method uses engineering calculations to estimate the types of damage likely to be experienced by the structure. Probable repair costs are then determined based on this damage. Such an approach has not been widely used in the past. However, through a contract with the National Institute of Building Sciences, FEMA has recently prepared a general loss estimation methodology, known as HAZUS, that employs a generalized version of this approach. In the HAZUS methodology, building damage functions are based on a standard capacity (pushover) curve for model building type and fragility *curves that describe the probability of discrete states of damage. Separately, building loss functions convert damage to different types of loss including casualties, economic losses and loss of function. Damage state probabilities are a function of the spectral demand on the structure, determined by the intersection of building capacity and earthquake demand spectra. Uncertainty in building capacity, damage states, and ground shaking is included in the fragility functions that convert spectral demand into damage state probabilities. This approach is appealing in that it allows the direct use of the details of an individual building's construction that are important to its earthquake performance, including strength, stiffness, and configuration, in the loss estimation process. This approach has been adopted for the Detailed Loss Estimation Method found in Appendix B of these Recommended Criteria.*

4.2.1 Use of Loss Estimation Methods

Results from either the Rapid Loss Estimation or Detailed Loss Estimation methods may be used to estimate building damage and loss. These data can assist in making economic decisions regarding the building, e.g., benefit-cost studies to determine if structural upgrade is warranted. Estimates made using the rapid loss estimation method should be considered as representative only of *average* buildings. They should therefore be used with caution since the unique structural characteristics of any individual building will affect its vulnerability. While the detailed loss estimation method directly takes into account the structural characteristics of a building, it also uses general data for other aspects of the loss estimation process including the cost of repairing damage of given types, and the replacement value of the building. Hence, estimates performed by either of these techniques may require some adjustment by the user to better reflect the particular situation.

Commentary: When applying the rapid loss estimation method to a specific building, consideration should be given to such factors as the strength and stiffness of the lateral force resisting system, inherent redundancy, physical condition, quality of construction, and conformance with building code provisions. Buildings having substantial deficiencies would be expected to be significantly more vulnerable. Similarly, buildings that have superior earthquake resisting characteristics, relative to code requirements, would be expected to be less vulnerable. The detailed loss estimation method provides a direct method for evaluating these factors. In the rapid method, this can only be accounted for qualitatively, using the judgement of the evaluator.

In addition to these construction characteristics, known to affect building performance in earthquakes, a very significant factor that affects the costs associated with earthquake damage relates to building occupancy. It is much more difficult, and costly, to repair damage in buildings in critical occupancies, such as hospitals and semiconductor manufacturing clean rooms, than it is in buildings in standard office or residential occupancies. This is both because the finishes and utilities that must be disturbed to conduct structural repairs are more *complex and expensive, and also, because general working conditions are more restrictive. These factors are not directly accounted for by either of the methods.*

4.2.2 Scope of Loss Estimation Methods

The Rapid Loss Estimation and Detailed Loss Estimation methods may be used to estimate direct economic loss related to repair of building damage resulting from the effects of ground shaking. These direct losses include costs associated with inspection to determine the extent of damage, design and professional services fees, demolition and replacement costs for finished surfaces and utilities that must be removed and replaced to allow access for inspection and repair, and actual repair construction costs. The methodologies permit estimation of costs related to structural repair and to repair of non-structural building features including architectural finishes, mechanical and electrical equipment. The methodologies do not include losses related to contents including office equipment, inventory, and similar tenant property.

Ground shaking is the primary, but not the only source of earthquake induced damage, and therefore loss that occurs in earthquakes. Other hazards that can result in such losses include liquefaction, landsliding, earthquake induced fire and flood. While these hazards typically damage only a small percentage of the total inventory of buildings affected by an earthquake, they can be far more damaging to those properties that are affected than is ground shaking. Regardless, estimates of loss due to these effects are not included in these methodologies.

In addition to direct economic loss resulting from ground shaking, there are also many other types of loss that result from the effects of earthquakes. This includes life loss and injury, as well as large economic losses due to interruption of business. Estimation of these losses is also beyond the scope of the methodologies presented here.

4.3 Rapid Loss Estimation Method

4.3.1 Introduction

This section presents loss estimation functions that relate seismic demand, resulting primarily from ground shaking, to expected loss. The functions are presented in several formats so that users can adjust the various loss components to better reflect special knowledge about specific buildings. The functions were developed using 1994 Northridge earthquake damage data and are, therefore, expected to be representative of steel moment-frame buildings typical of California construction prior to 1994.

In this methodology, losses are quantified in three ways.

1. Damaged Moment Connections, expressed as a percentage of the total number of moment connections in the building.
2. Connection Restoration Cost, expressed as a percentage of the building replacement value.
3. Nonstructural Repair Cost, expressed as a percentage of the building replacement value. These other repair costs include costs related to restoration of non-structural elements,

including fascia, ceilings, and utilities. It does not include costs related to contents such as computer systems or stored inventories.

Commentary: The predictive models for building losses contained in this methodology are based on statistical data available from buildings affected by the 1994 Northridge earthquake. The damage surveys and database used in the development of the method dealt with the numbers and types of connection damage, and to a lesser extent, with repair costs and nonstructural damage. Hence, the primary parameter available and used in the statistical analysis was the quantity of damaged moment connections in affected buildings. Reported structural repair costs varied widely (some also included costs associated with defective welds as opposed to damaged connections), making it impossible to derive a reliable direct relationship between seismic demand and connection restoration cost. Instead, connection restoration costs were computed for each surveyed building as the estimated total number of damaged connections times average unit costs for connection repair. For other damage, including nonstructural repair costs and other structural repair costs, only very qualitative descriptions were reported. Therefore, these other repair costs could not readily be ascertained from the Northridge data. The unit costs used in the loss functions are provided so that users can adjust loss estimates to better reflect particular situations and so that should additional data become available in the future, the methodology can be extended in a consistent manner.

The only structural repair costs directly included in the loss functions presented in this methodology are costs related to repair of damaged moment resisting connections. Costs related to other structural repairs such as correcting permanent interstory drifts are not directly accounted for by these functions. However, Section 4.3.4 provides qualitative information that may allow the user to develop estimates of the potential additional costs that could be incurred in such repairs.

4.3.2 Seismic Demand Characterization

Direct damage repair costs are functions of seismic demand resulting primarily from ground shaking. The method presented here characterizes seismic demand in three alternative ways.

1. Modified Mercalli Intensity (MMI) at the building site. MMI is typically derived for a site, following an earthquake, based on observation of damage and other earthquake effects at the site. Several investigators have developed correlations between observed MMI and estimated ground shaking acceleration, velocity and displacement. The MMI values used in these *Recommended Criteria* were derived from estimated peak ground accelerations and velocities during the 1994 Northridge earthquake.
2. Peak Ground Acceleration (PGA) at the building site. This is the geometric mean (square root of the product) of the estimated peak values in each of the building's two principal directions.

3. Building Pseudo-Drift Ratio (S_d/H). This is defined as the spectral displacement S_d divided by the building height H from grade level to main roof. The spectral displacement is the geometric mean of the values in each of the building's two principal directions. The spectral displacement is that at the building fundamental period from a site-specific 5% damped response spectrum. Consistent units are used so that S_d/H is dimensionless.

Commentary: Seismic demands are intended to be those caused primarily by ground shaking. The Rapid Loss Estimation Method is not intended to cover losses governed by other hazards such as ground failure, inundation, and fires following earthquakes.

The damage patterns produced by the 1994 Northridge earthquake exhibited considerable scatter. Some buildings reported no connection damage whereas others in relatively close proximity had many damaged connections. The reasons for this are unclear; however, this random damage pattern has frequently been observed in other earthquakes. The scatter may be attributed to a number of factors including large uncertainties in the ground motion estimates for each site, the effects of individual building configuration and construction quality, and the relative thoroughness and accuracy of damage reporting for different buildings. Statistical data analysis using numerous different seismic demand measures (e.g., MMI, PGA, Peak Ground Velocity (PGV), and Peak Ground Displacement (PGD)) as damage predictors did not identify any single parameter as being clearly superior for prediction of percentage of damaged connections (FEMA-355E). Since no one measure of ground shaking intensity seemed to provide a best fit with the available Northridge data, the three measures of ground motion intensity presented in these Recommended Criteria were selected based on considerations of the probable needs of users.

MMI was chosen primarily because of its historical use in earlier loss studies and the fact that it continues to be used by many practitioners today. MMI is a highly subjective parameter intended to be determined after an earthquake, based on observed patterns of damage in different areas. It is of course problematic to use such an approach to characterize distributions of MMI for a future earthquake, that has not yet occurred. A number of researchers have attempted to develop correlation functions that relate observed MMI to less subjective measures of ground shaking including peak ground acceleration and peak ground velocity, which can then be predicted for future earthquakes using various attenuation relationships. These predictive models for MMI inherently incorporate significant variability and uncertainty. Nevertheless, most practitioners who use MMI based approaches to predict losses in future earthquakes, first use one of these predictive models for MMI upon which to index their loss estimates.

Consistent with this approach, the MMI values used in the loss functions presented here are those inferred from peak ground accelerations and velocities recorded during the 1994 Northridge earthquake, using a predictive model by

Wald et al. (1998). They are not based on actual damage observations. For a given site, there may be considerable difference between the observed MMI and the predicted MMI values.

PGA was chosen because it is an unambiguous, commonly recorded and reported, earthquake intensity parameter. One of its shortcomings as a loss predictor, however, is that PGA is not reflective of the spectral content of ground shaking. Steel moment-frame buildings are typically long-period structures and theoretically their response should more closely be related to peak response velocity or displacement than to peak ground acceleration. However, these quantities are often unavailable for an individual building site, and did not provide significantly better correlation with the available data.

Engineering study of the behavior of steel moment-frame buildings indicates that interstory drift is a reasonable parameter for predicting the amount of damage experienced by a structure. Therefore, S_d/H *was chosen as a ground motion intensity index for these Recommended Criteria because it is closely related to average interstory drift demands produced in steel moment-frame buildings. Also, it includes information about the seismic intensity at the site, and the dynamic characteristics of the ground shaking experienced as well as the particular building's dynamic response properties. Unfortunately, statistical analysis did not show this to be a better damage predictor than PGA. It is believed that the uncertainty in the survey data masks its predictive power. Nevertheless, its inclusion here is intended to promote the use of such engineering parameters in future loss studies.*

4.3.3 Connection Damage Loss Functions

Figures 4-1, 4-2 and 4-3 present functions that may be used to estimate Connection Damage Ratio (CDR) as a function of Modified Mercalli Intensity (MMI), Peak Ground Acceleration (PGA), and Pseudo Interstory Drift Ratio (PIDR), respectively. In these figures, connection damage is expressed as the percentage of moment connections within the total number of connections in the building's lateral-force-resisting system in all building directions, that are damaged as discussed in Section 2.3. A connection is defined as the attachment of one beam to one column. A connection is considered to be either damaged or undamaged (i.e., the relative severity of damage is not considered). A connection may be damaged at the beam bottom flange location, top flange, or both. Damage may also include the beam web connection and the column panel zone. No attempt is made to distinguish between these various types of damage. Defects at the roots of the CJP welds between beam and column flanges, which were often categorized as damage in buildings affected by the 1994 Northridge earthquake are not considered as damage herein.

Median and 90th percentile loss functions are presented. A set of typical buildings subjected to the same seismic demand will exhibit losses over a range. The median loss has the property of having the same numbers of buildings with smaller losses as there are with larger losses. The

90th percentile loss has the property that 9 out of 10 buildings have losses equal to or lesser in magnitude.

Commentary: Connection damage was the key parameter that was statistically evaluated from the 1994 Northridge damage surveys. Connection restoration costs (Section 4.3.4) are derived from the connection damage by use of unit repair costs. The figures show plots of the actual recorded damage for buildings contained in the data set as well as smooth curves that approximately represent the median and 90th percentile statistics. The curves were based in part on expert judgement that the extent of damage is dependent on seismic demand, even though the actual damage data indicates a weak correlation between damage and intensity. About ½ of the buildings in the database experienced no damaged connections, and hence many data points are clustered about the horizontal axis in the figures.

The building damage surveys used in the development of the functions presented are predominately from buildings covered by the City of Los Angeles Ordinance No. 170406 requiring the identification, inspection and repair of commercial steel moment-frame buildings subsequent to the 1994 Northridge earthquake. The database contained 185 buildings. Implicit in the use of this data for loss estimation is the assumption that this sample is representative of data for a major metropolitan area. Comparison of the aggregate building characteristics (e.g., height and gross area) against census tract data for the greater Los Angeles region suggests that the sample is indeed representative of the Los Angeles steel moment-frame building population. Whether the sample is representative of other metropolitan areas has not been studied. In addition, the sample does have certain qualities that are noteworthy. First, residential buildings were excluded from the Ordinance and hence are not in the sample. Second, most of the seismic demands were in a somewhat limited range (i.e., PGA from about 0.25g to 0.45g). Hence, data for PGAs that lie outside this range were sparse, and expert opinion was instrumental in defining the loss functions there.

Statistical analysis of the data found that building attributes such as height or redundancy (floor area per connection) were not significant parameters affecting the percentage of damaged connections. No adjustment factors for these characteristics were included herein, nor are they recommended, to adjust the estimates made using this data.

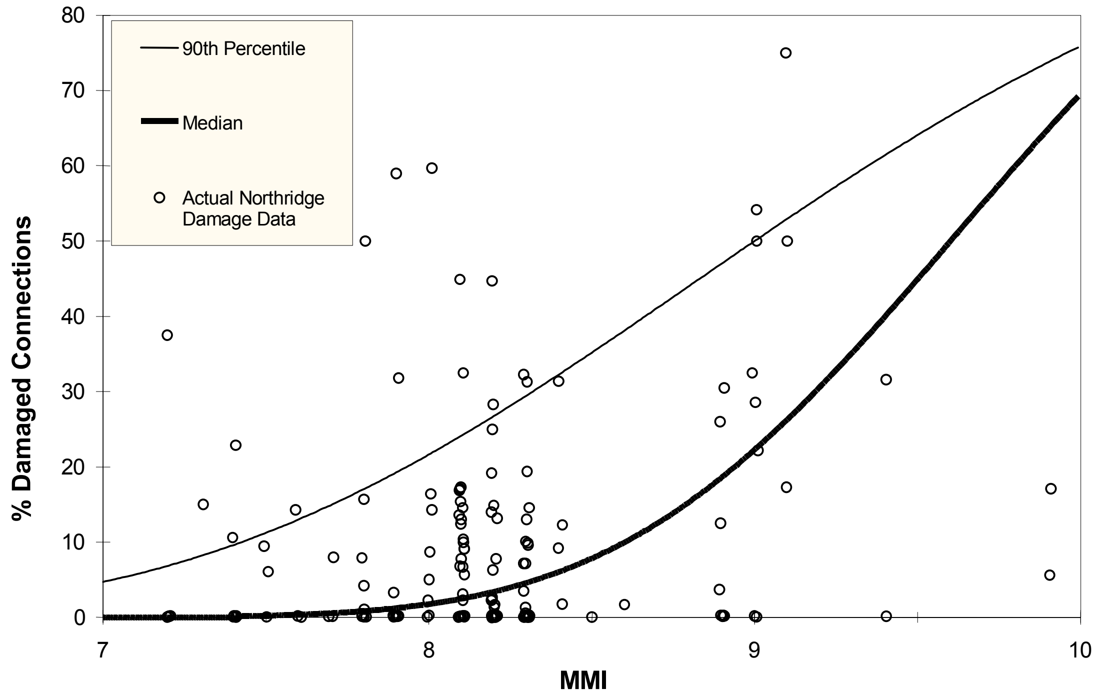


Figure 4-1 Connection Damage Ratio vs Modified Mercalli Intensity (MMI)

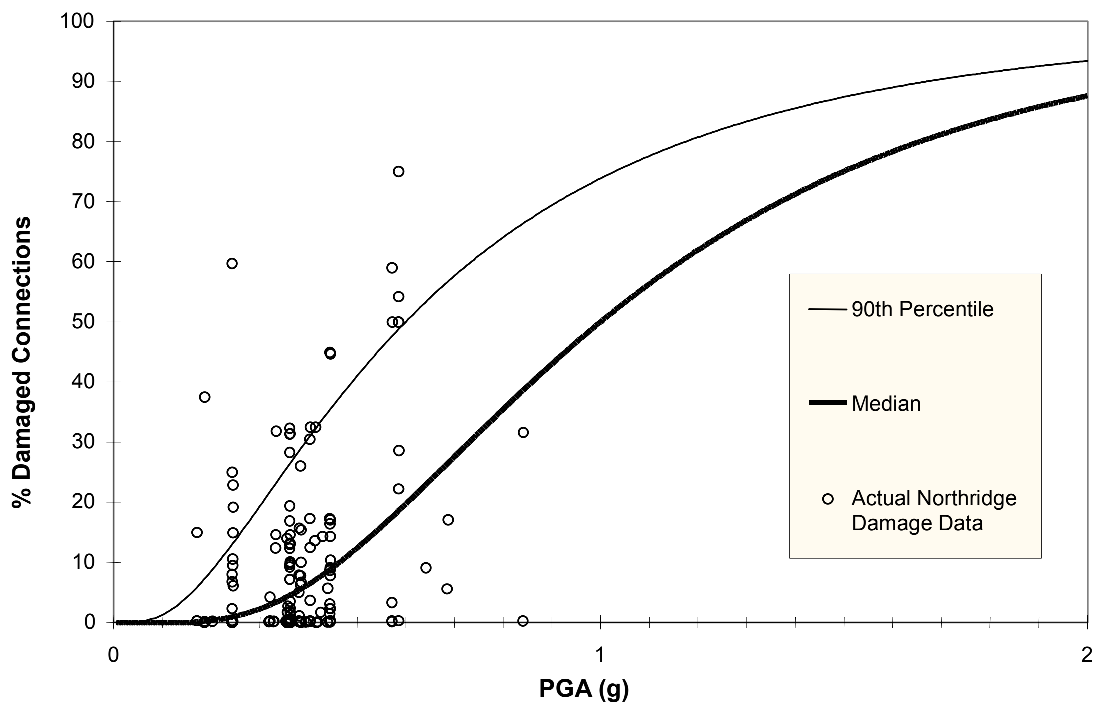


Figure 4-2 Connection Damage Ratio vs Peak Ground Acceleration (PGA)

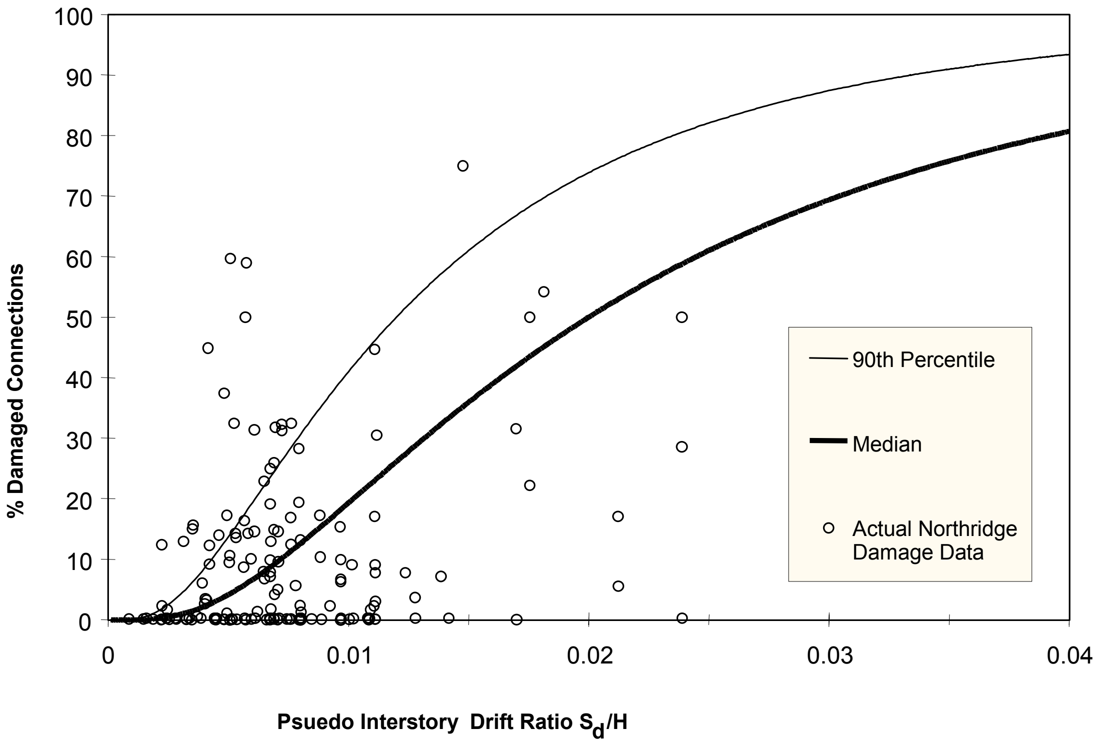


Figure 4-3 Connection Damage Ratio vs Building Pseudo Interstory Drift Ratio

4.3.4 Connection Restoration Cost Functions

Figures 4-4, 4-5, and 4-6 present connection restoration cost, expressed as a percentage of building replacement value, as a function of MMI, PGA and PIDR, respectively. In the development of the curves presented in the figures, the average unit cost for connection restoration has been taken as $20,000, including costs associated with selective demolition and restoration of finishes and utilities to provide access for repair. The building replacement value is taken as $125 per sq. ft times the gross building area.

Commentary: In the development of a typical steel moment-frame building, the cost of structural construction is approximately 25% of the total building development cost. Thus repair costs on the order of 20% or more approach the original cost of constructing the structure. The costs indicated in Figures 4-4, 4-5 and 4-6 do not include costs associated with repair of damage to elements other than moment-resisting connections, for example, column splices, and non-participating framing. However, in the 1994 Northridge earthquake, costs of these other repairs were not significant. In addition, the above costs do not consider the effect of large permanent lateral displacements that can occur in damaged frames. Several buildings damaged by the Northridge earthquake experienced permanent interstory drifts. Generally, when the permanent drift did not exceed a level that was visibly disturbing or interfered with operation of elevators, the buildings were not re-plumbed. Re-plumbing buildings that have experienced large permanent drifts can be costly, and in many cases may be

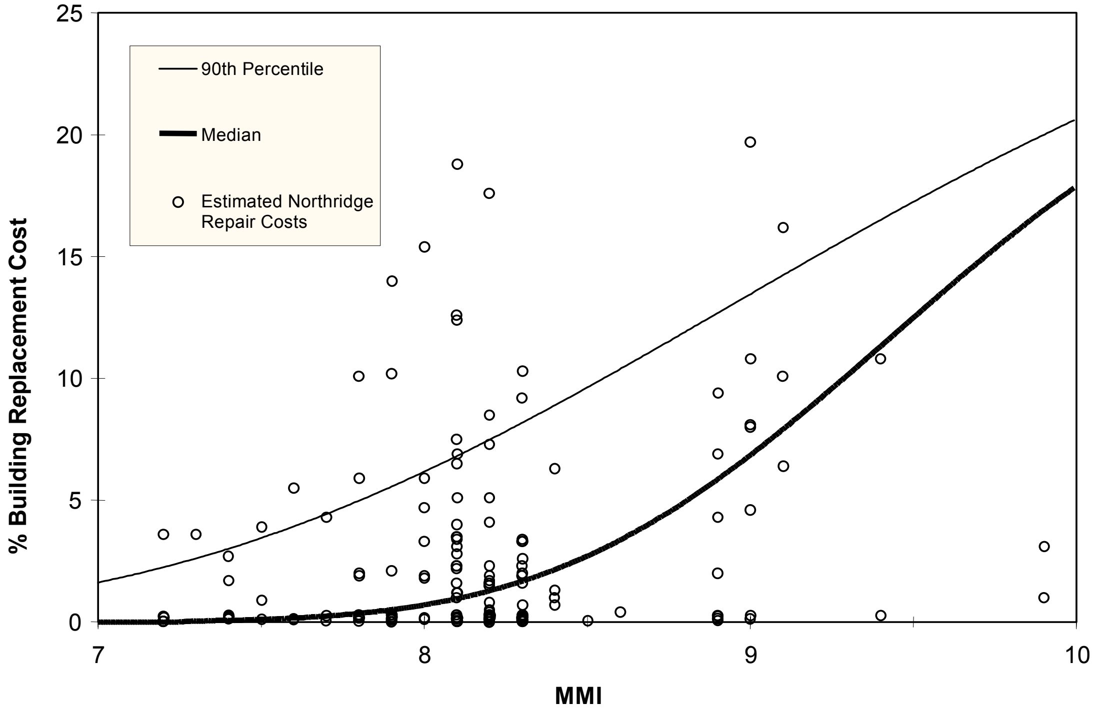


Figure 4-4 Connection Restoration Cost vs Modified Mercalli Intensity (MMI)

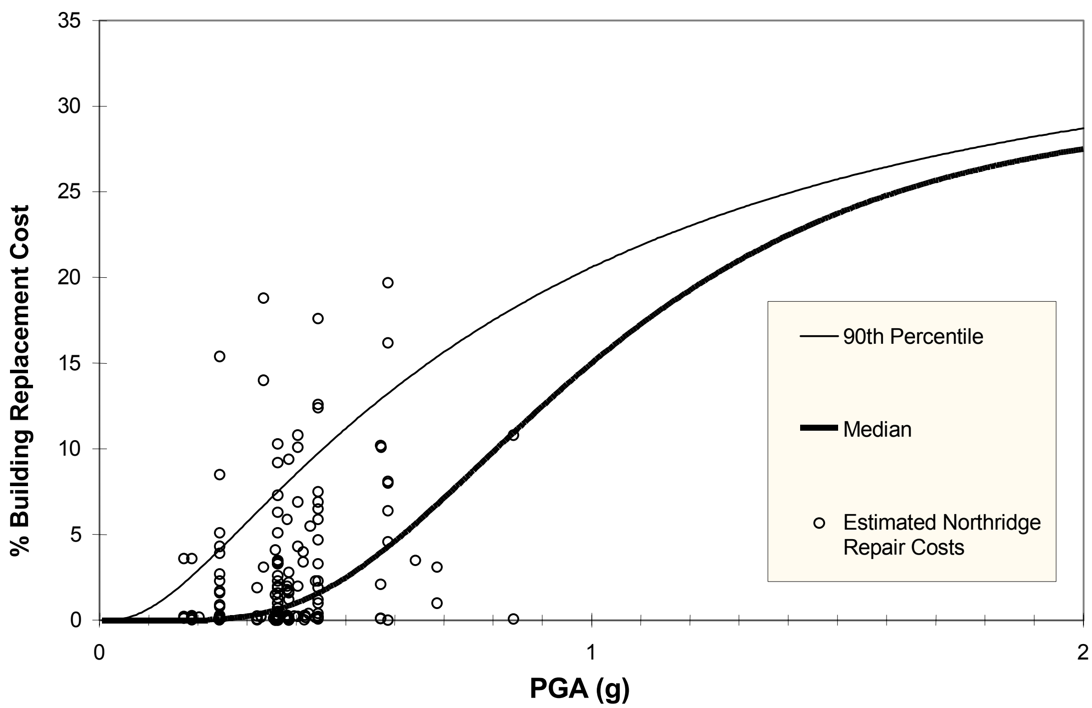


Figure 4-5 Connection Restoration Cost vs Peak Ground Acceleration (PGA)

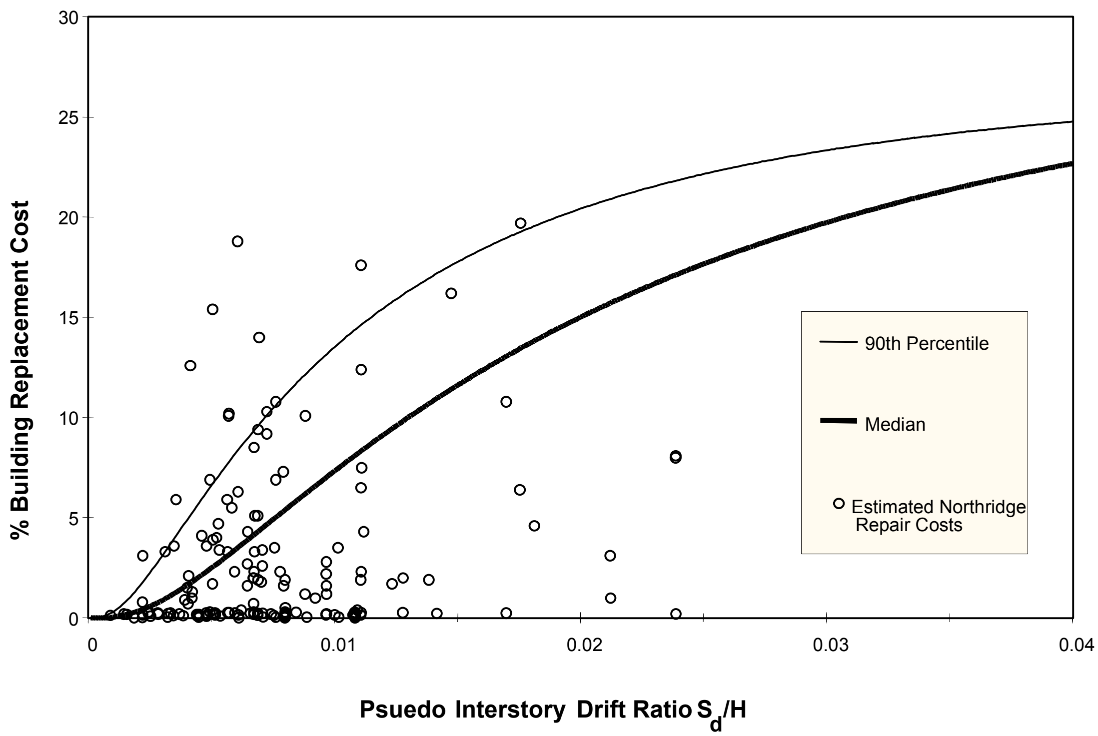


Figure 4-6 Connection Restoration Cost vs Building Pseudo Interstory Drift Ratio

impractical to accomplish. Thus if a building has experienced large permanent interstory drift, the effective cost of structural repair can be larger than indicated by these loss functions.

As a general rule, permanent interstory drift may be on the order of 1/3 to 1/2 of peak interstory drift. The AISC Standard Practice requires that erection of buildings produce a plumb within .005. Permanent interstory drifts of perhaps .01 may be tolerable in buildings, while drifts larger than this would probably require either straightening or loss of use of the building. These considerations have not been accounted for in the above loss functions.

4.3.5 Nonstructural Repair Cost Functions

Figures 4-7, 4-8 and 4-9 present nonstructural repair cost, expressed as a percentage of the building replacement value, as a function of MMI, PGA and PIDR, respectively. The costs are based on HAZUS unit costs and damage states and have been modified by expert opinion founded on 1994 Northridge earthquake experience and by engineering judgement. The unit costs are taken as Los Angeles commercial office types (professional, technical, and business services). Complete repair costs for acceleration-sensitive and drift-sensitive nonstructural building components are taken as $42 and $28 per sq. ft, respectively. These unit costs may serve as the basis for adjusting the loss functions for particular building situations.

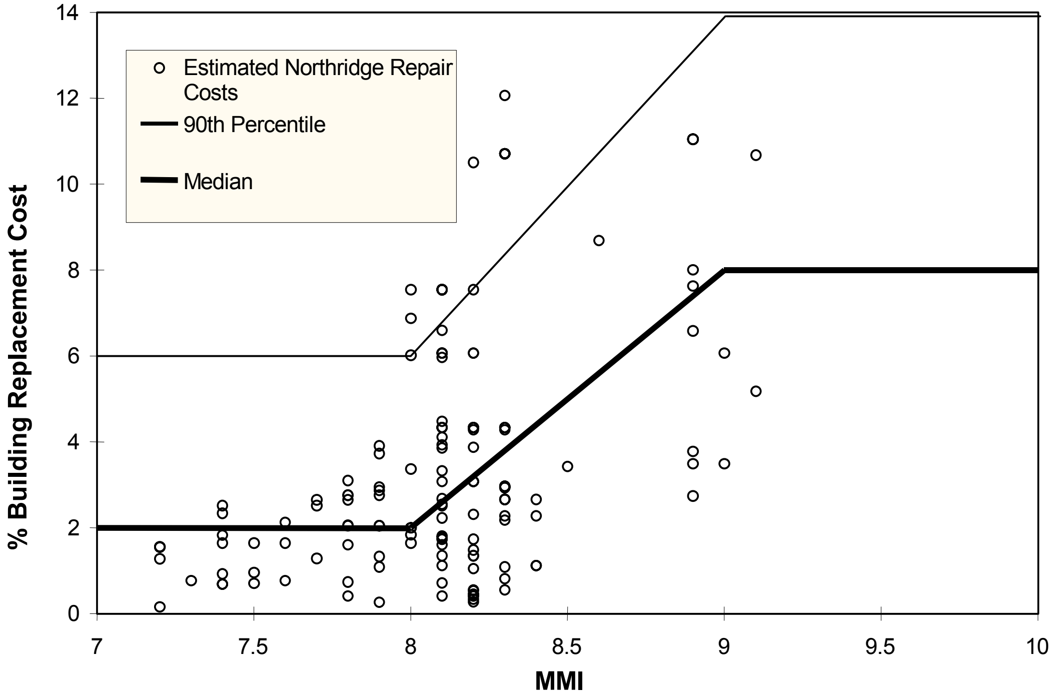


Figure 4-7 Nonstructural Repair Cost vs Modified Mercalli Intensity (MMI)

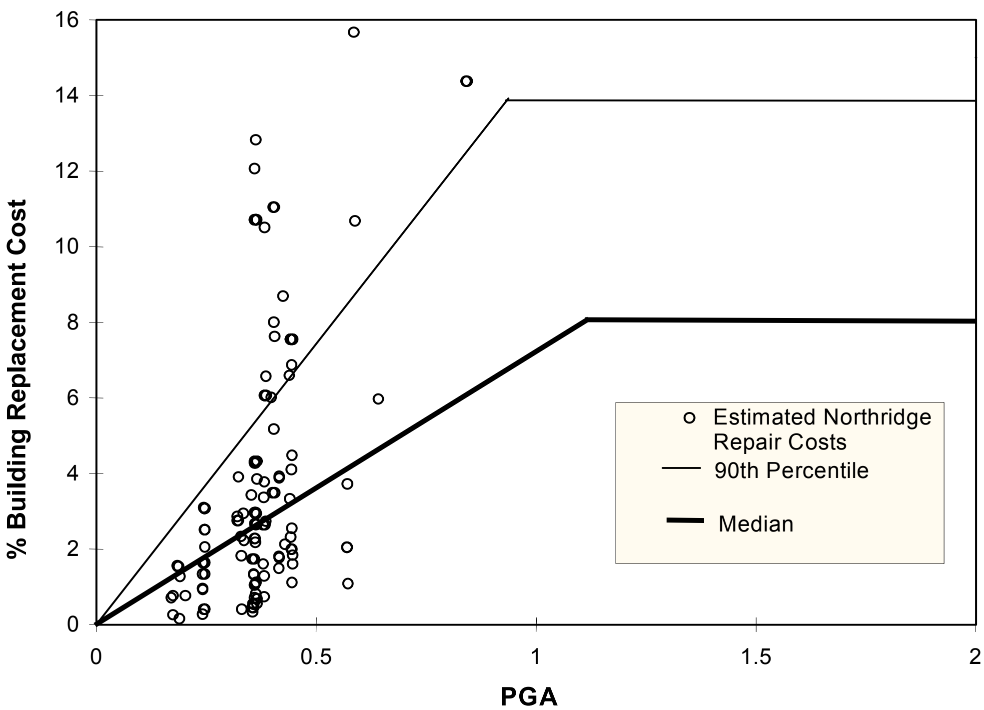


Figure 4-8 Non-Structural Repair Cost vs Peak Ground Acceleration (PGA)

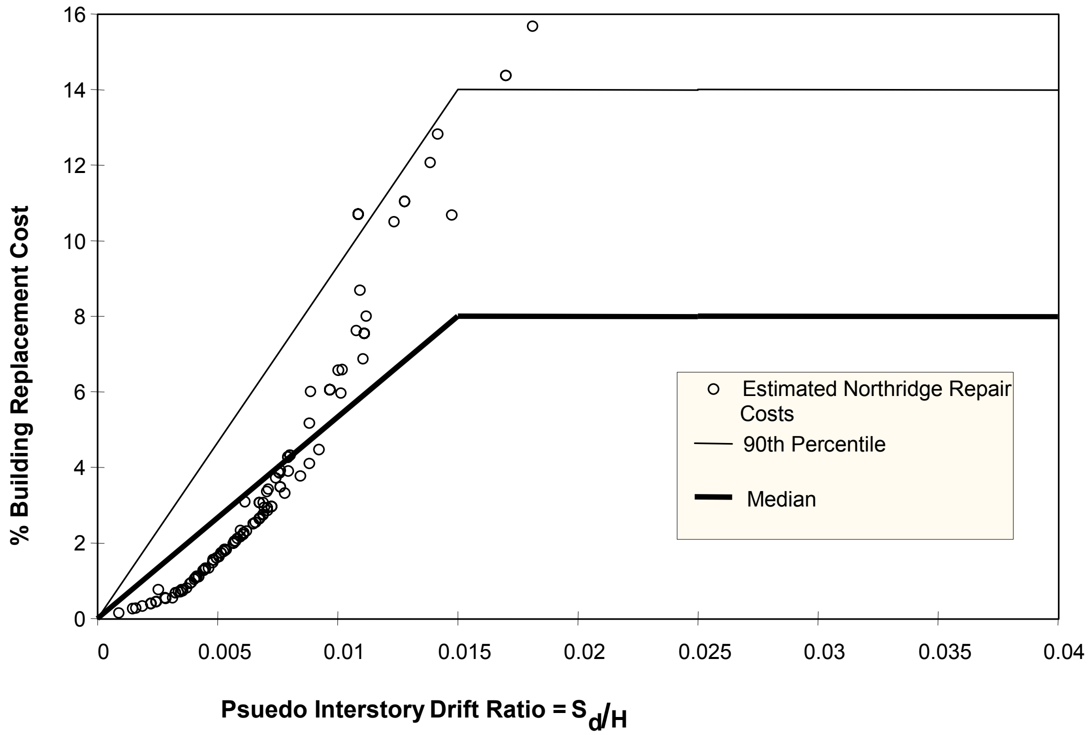


Figure 4-9 Nonstructural Repair Cost vs Building Pseudo Interstory Drift Ratio

Commentary: Nonstructural repair costs rely heavily on the information from the HAZUS project because very sparse quantitative information was available from the Northridge damage surveys. Pseudo (or implied) nonstructural repair costs were generated for each building in the sample and best-fit curves were generated by judgment. The descriptions of nonstructural damage from the Northridge building surveys suggested that the repair costs were generally less than that indicated by the curves. Hence, the curves were adjusted downward based on engineering judgement.

5. Seismic Upgrade

5.1 Scope

Seismic upgrade measures for steel components and elements of welded steel moment-frame (WSMF) structures are described in this chapter. Information needed for simplified and systematic upgrade of steel buildings is presented herein.

5.2 Codes and Standards

Table 5-1 indicates the general codes, standards, and guideline documents that are applicable to seismic upgrades for WSMF structures, and the extent to which they are applicable.

Table 5-1 Applicable Codes, Standards and Guideline Documents

Designation	Title	Applicability
FEMA-273	*NEHRP Guidelines for the Seismic Rehabilitation of Buildings*	Provides general performance-based guidelines, that are modified herein
FEMA-302	*NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*	Governs the detailing, materials and workmanship for new construction employed in upgrade design
AWS D1.1	*Structural Welding Code – Steel*	Governs the requirements for design, materials and workmanship for structural welding
AISC/LRFD	*Specification for the Design of Steel Structures*	Provides design requirements for bolting, welding, computation of member capacities, to the extent referenced herein
NIST/AISC (Gross, et al. 1999)	*Recommendations for Seismic Upgrade of Steel Structures*	Provides design procedures for specific types of connection upgrades, as referenced herein

Commentary: FEMA-273 provides guidelines for determining force and deformation demands for the design of rehabilitation systems for structures to meet specific performance objectives. As described in the commentary to Section 3.1 of this publication, FEMA-273 takes a somewhat different approach to the definition of performance objectives than do these Recommended Criteria. Also, FEMA-273 was published prior to much of the extensive research on WSMFs conducted under this project as well as research conducted by other organizations following the 1994 Northridge earthquake. These Recommended Criteria contain information that specifically updates the recommendations contained in FEMA-273, with regard to the upgrade (rehabilitation) of WSMF structures. FEMA-273 provides a more comprehensive treatment on other *building upgrade issues, including provision of procedures for rehabilitation of foundations, diaphragms and nonstructural components. The guidelines contained in these Recommended Criteria only address the upgrade of the steel frame itself. Refer to FEMA-273 for guidelines on the upgrade of these other systems.*

Prior to performing an upgrade on any existing building it is advisable to discuss the proposed design criteria with the building official. Although the building code for new construction is not intended to apply to existing buildings, in some jurisdictions building officials require that upgrades be designed to conform to the strength requirements of the current prevailing code, or a fraction thereof. In 1991, language was introduced into the Uniform Building Code specifically permitting voluntary seismic upgrades of buildings without requiring complete conformance with the building code design criteria as long as it could be demonstrated that the following conditions did not occur:

- *The upgrade work does not create a structural irregularity or make an existing irregular condition more severe*
- *The upgrade work does not deliver more load to an existing element than it can withstand*
- *The upgrade work does not create an unsafe condition.*

Similar language has recently been introduced into the 2000 International Building Code. The upgrade criteria contained in these Recommended Criteria presume that the above permissive language is incorporated into the local building code or that the building official is willing to accept upgrades designed to criteria other than that contained in the building code.

Although these Recommended Criteria suggest that upgrades designed in accordance with the criteria need not comply with the strength and drift limits specified by the applicable building code for new construction, new work performed as part of the upgrade should conform to all materials, detailing, and workmanship criteria of the code, as supplemented by these Recommended Criteria.

5.3 Upgrade Objectives and Criteria

Two approaches are available for seismic upgrade of steel moment-frame structures – a Simplified approach and a Systematic approach. In the Simplified approach, modifications are made to individual moment-resisting connections to improve their ability to provide ductile inelastic behavior. No analyses or evaluations are performed as part of the design of these modifications to assess whether the overall structural system is capable of meeting specific performance objectives. In the Systematic approach, a complete evaluation of the performance capability of the structure is performed in order to verify the performance capability of the upgraded structure. Upgrades may include connection modifications, providing supplemental lateral force resisting elements, such as braced frames or shear walls, or introducing response modification measures such as base isolation or energy dissipation devices.

Commentary: Throughout the period that steel moment-frame construction has been popular, the objective of the building code has been to provide buildings with the capability to resist the following: minor earthquakes without damage; moderate earthquakes without structural damage but with some nonstructural damage; major earthquakes with potentially significant structural and nonstructural damage, but not so much damage as to pose a significant threat to life safety; and the most severe levels of shaking anticipated to occur at a site without collapse. The ability of code-conforming structures actually to provide this performance has been mixed. In general, most code-conforming buildings have met the latter two goals well, but have experienced more damage at moderate levels of shaking than would seem to be desirable. To the extent that the code provisions that prevailed at the time a building was designed and constructed were adequate to meet these objectives, except that connections were more vulnerable to damage than originally believed, the use of the simplified upgrade approach, as described in these Recommended Criteria, will restore structures to the originally intended performance capability.

In the simplified upgrade approach, individual moment-resisting connections of the structure are upgraded to provide capacity for ductile behavior comparable to that presumed to exist at the time of the original design. The adequacy of other elements of the structure, including its basic configuration, strength, stiffness, and the compactness of sections are not evaluated and are not upgraded. As a result, no specific performance can be associated with structures that are upgraded using the simplified approach, unless a detailed performance evaluation is undertaken, in accordance with the procedures of Chapter 3.

In the systematic upgrade method a performance evaluation is performed as an inherent part of the design evaluation process. This permits upgrade work to be designed for specific performance objectives, which may be the same as, superior to, or less than those originally intended at the time of building design. Regardless of the selected objectives, the systematic approach will provide greater confidence in the ability of the structure to actually achieve the intended performance than does the simplified approach.

5.3.1 Simplified Upgrade

In simplified upgrade, vulnerable connections are upgraded, through a variety of measures, to provide more reliable performance of the individual connections. No overall evaluation of the performance of the structure, with upgrade modifications, is performed. Presuming that the structure, as originally designed and constructed, conformed to the applicable building code requirements, but incorporated fracture-vulnerable connections, this method of upgrade could be used to restore the structure to its originally intended performance capability.

In simplified upgrade, the individual beam-column connections of the existing lateral-force-resisting system for the welded steel moment-frame structure are modified to provide equivalent interstory drift capacity to that required for a new WSMF structure having the same structural system. Existing WSMF structures will typically have been designed, either as Ordinary Moment Frames (OMF) or Special Moment Frames (SMF). Chapter 6 of these *Recommended Criteria* provides design criteria for selected pre-qualified connection upgrades, that are accepted generically as being capable of providing the necessary drift angle capacity for either OMF service, SMF service, or both. Chapter 6 also provides project-specific qualification procedures that may be used to affirm that other connection upgrades provide the desired drift angle capacity.

Commentary: The intent of Simplified Upgrade is to reduce the susceptibility of moment-resisting beam-column connections detailed and constructed in accordance with typical pre-1994 practice to premature, brittle fracture damage. When selecting Simplified Upgrade it is inherently accepted that the susceptibility of such moment-resisting connections to brittle fracture damage is the only significant vulnerability of the structure and that mitigation of this vulnerability will result in a structure with acceptable performance characteristics, relative to those intended at the time of the original design. This may or may not actually be the case, and can be verified only by a detailed performance evaluation.

Unless original design documents are available, and indicate the design intent with regard to the structural system, it should be presumed that the original design intent for the structure was to be equivalent to an SMF. If design documents are available, these may identify the original intended structural system, as being either an SMF, an OMF or a Ductile Moment-Resisting Frame. The original design intent for structures indicated as Ductile Moment-Resisting Frames should be considered equivalent to that for SMF.

5.3.2 Systematic Upgrade

In systematic upgrade, a detailed performance evaluation of the structure is performed in its existing configuration and its ability to meet desired performance objectives is determined in accordance with the procedures of Chapter 3. If it is found that there is an inadequate level of confidence that the structure is capable of meeting the desired performance objectives, then structural modifications are performed to improve the probable performance and increase the level of confidence. These modifications could include connection improvement measures, such as those available for simplified rehabilitation, but could also address systemic issues such as the basic strength and stiffness of the structure, the presence of irregularities or other vulnerabilities. An iterative process is followed in which a performance evaluation of the building in accordance with Chapter 3 is performed assuming proposed modifications are in place, and if the desired confidence of achieving the performance objective is not indicated, additional modifications are performed.

Prior to performing a systematic seismic upgrade, one or more suitable performance objectives must be selected as the basis for design. Performance objectives should be selected in accordance with Section 3.2. A performance evaluation should be conducted of the structure, to determine a level of confidence associated with its ability to meet these performance objectives. If sufficient confidence is not attained, then upgrade modifications should be developed, either to reduce the response of the structure to earthquake ground shaking, such that acceptable confidence of achieving the desired performance is attained, or to increase the capacity of the structure to withstand earthquake response and provide acceptable confidence.

Commentary: Performance objectives, selected in accordance with Section 3.2 are not completely compatible with those selected in accordance with FEMA-273. In FEMA-273, a performance objective is defined as consisting of two parts – a desired performance level, of which there are three (Immediate Occupancy, Life Safety, and Collapse Prevention) and a desired ground shaking spectrum for which this performance level is not to be exceeded. In these guidelines, only two performance levels are defined (Immediate Occupancy and Collapse Prevention) and a level of confidence with regard to providing the desired performance for a given ground shaking hazard is developed.

The Immediate Occupancy level defined in these Recommended Criteria, is similar, but not identical, to the Immediate Occupancy level of FEMA-273. The Collapse Prevention level of these Recommended Criteria may be taken as equivalent to the Collapse Prevention level of FEMA-273. If it is desired to attain performance equivalent to the Life Safety level of FEMA-273, using these Recommended Criteria, this may be attained by using 75% of the acceptance criteria (e.g., for drift capacities, strength capacities) specified in these guidelines for Collapse Prevention.

To create performance objectives, using these Recommended Criteria, that are roughly equivalent to those contained in FEMA-273, it is necessary to associate a probability of exceedance, within a specified period (e.g., 50 years) with the response spectrum used to define the hazard under the FEMA-273 criteria. Upgrade designs that provide a 90% confidence level for the desired performance level based on global interstory drift, column compression and column splice tension and a 50% confidence level for local connection behavior at this probability may be deemed equivalent to the intended performance of FEMA-273.

The global interstory drift, capacities and resistance factors contained in Chapter 3 are based on typical, regular welded steel moment-frame (WSMF) configurations. When adding structural systems that affect the dynamic characteristics of the WSMF (e.g., braced frames or shear walls), these default factors are no longer valid. For such structural upgrades, the demand and resistance factors contained in Chapter 3 may be applied to the calculation of confidence relative to local connection, column compression and column splice *tension behavior. If the new lateral-force-resisting elements, for example, shear walls or braced frames, are designed in accordance with the comparable performance objectives of FEMA-273, they may be presumed to provide adequate confidence with regard to global building behavior. Alternatively, the detailed performance evaluation procedures of Appendix A may be used to confirm global behavior.*

5.4 Upgrade Strategies

A systematic upgrade may be accomplished by any one or more of the following means, as required to obtain a structure that provides suitable confidence of capability to provide the desired performance:

- Connection modifications (Section 5.4.1)
- Removal or lessening of existing irregularities and discontinuities (Section 5.4.2)
- Global structural stiffening (Section 5.4.3)
- Global structural strengthening (Section 5.4.4)
- Mass reduction (Section 5.4.5)
- Seismic isolation (Section 5.4.6)
- Supplemental energy dissipation (section 5.4.7)

Commentary: A building's response to earthquake ground shaking results in the development of forces and deformations in the structure. In Chapter 3 of these Recommended Criteria, a procedure is defined for determining a level of confidence with regard to the ability of a structure to resist these forces and deformations with a defined probability of exceeding one or more performance levels. This confidence level is tied to the confidence parameter λ calculated as the ratio of the factored demands $\gamma\gamma_a D$ to the factored capacity ϕC to resist these demands. Values of the parameter λ less than 1 indicate relatively high confidence, while values above 1 indicate progressively lower confidence.

If upon evaluation in accordance with Chapter 3, it is found that an inadequate level of confidence is obtained with regard to the ability of the structure to meet a desired performance objective, an upgrade can be performed to improve this confidence. To be effective, this upgrade must be able either to increase the capacity of the structure, and its various elements to resist the forces and displacements induced by earthquake response, or alternatively, the amount of force and deformation that a structure develops (the demands) must be reduced. As a third alternative, it may be possible to attain a higher level of confidence with regard to the probable performance of a structure by obtaining better information on the structure's construction and by performing more detailed and certain analyses of the structure's response to ground shaking. The *following sections provide information on alternative methods of modifying a structure to either increase its capacity or decrease the demands. Appendix A provides guidelines for improving confidence with regard to the structure's performance, through the use of more accurate analyses and evaluations.*

5.4.1 Connection Modifications

Connection modifications are intended to upgrade the ability of individual connections to withstand expected rotational deformations with suitably low probability of unacceptable damage. This is judged to have been achieved when the ratio of factored drift angle capacity ϕC of the individual connections to withstand the factored demands $\gamma\gamma_a D$ determined from an analytical evaluation of structural performance, results in an acceptable confidence index λ. Connection upgrades accomplish this in two ways. First, the upgrades directly improve the interstory drift angle capacity of individual connections, resulting in a reduced value of λ for local behavior. Second, if connections are upgraded to Special-Moment-Frame-compatible detailing, the connections are converted from type 2 (brittle behavior) to type 1 (ductile behavior) permitting use of increased global interstory drift capacities and reduced demand factors. Chapter 3 provides more information on these issues. Chapter 6 presents a series of pre-qualified connection upgrades, together with design procedures, the limiting parameters for which these upgrades are pre-qualified, and the drift angle capacities of the upgraded connections. Chapter 6 also presents a project-specific connection qualification procedure for use in determining appropriate drift angle capacities and capacity factors, for connection upgrades that are not included in the prequalifications.

Commentary: Connection upgrades are a method of increasing the local capacity of the individual connections to withstand inelastic deformation demands, as measured by drift angle. These upgrades do not, in general, reduce the demands produced in a structure by earthquake response. Therefore, connection upgrades are not, by themselves, particularly effective in improving the performance of structures that experience excessive demands due to inadequate frame stiffness or strength, or inappropriate frame configuration. Such vulnerabilities are better addressed with other upgrade strategies. For many structures, it may be necessary both to reduce the demands produced by earthquake response as well as increase the capacity of the individual connections to resist this response. In such cases, connection upgrades should be performed together with other upgrade strategies.

Although connection upgrade strategies directly address the single most common vulnerability of steel moment-frame structures – connections prone to premature brittle fracture – these upgrades can be quite costly, particularly in large structures with many connections. In some cases, it may be more cost effective to adopt strategies intended to reduce demands on connections rather than to increase individual connection capacities.

Some connection upgrade details have the potential to grossly affect the inelastic response behavior of frames. For example, some connection upgrades may shift the zones of plastic deformation from the beam column-joint to the beam, column or panel zone. Such modifications of inelastic response behavior will alter the demands placed on the individual connections, as well as the frame as a whole, and should be considered when connection upgrade strategies are adopted.

Connection upgrades that improve the drift angle capacity of the connections compatible with Special Moment-Frame requirements for new construction also result in a decrease in uncertainty relative to probable frame behavior. This is because of the reduced propensity for brittle fracture of the connections. This reduction in uncertainty is reflected in the use of demand factors appropriate for Type 1 connections, as described in Chapter 3.

5.4.2 Lessening or Removal of Irregularities

Many existing welded steel moment-frame buildings incorporate one or more structural irregularities. Some irregularities, such as soft stories, weak stories, torsional irregularities, and discontinuous structural systems can result in poor structural performance. Typically this poor performance occurs due to the concentration of force and inelastic deformation demand in the area of the irregularity. Often, the structural elements in the area of the irregularity are incapable of withstanding these locally increased demands. Structural upgrades that remove or lessen these irregularities have the effect of decreasing this concentrated demand resulting in a more uniform distribution of deformation and energy dissipation throughout the structure.

A structural irregularity should not be considered to be a problem unless a structural performance evaluation, conducted in accordance with Chapter 3 of these *Recommended Criteria*, indicates that structural demands, e.g., interstory drift or column axial load, in the area of the irregularity are in excess of the acceptance criteria for the desired structural performance level. Where an undesirable irregularity exists, it can usually be eliminated or reduced through the local introduction of new structural elements or through strengthening and stiffening of existing elements. When such features are introduced, a re-evaluation of the entire structure should be performed to ensure that the measure will result in adequate performance and that some new irregularity or vulnerability has not been inadvertently introduced into the structure.

5.4.3 Global Structural Stiffening

Damage to both structural and nonstructural elements is closely related to the amount of deformation induced in a building by its response to ground shaking. Global structural stiffening is intended to directly reduce the amount of this lateral deformation through introduction of stiffening elements. Although reinforcement of connections often results in some structural stiffening, this is typically not a significant effect and is not by itself adequate to result in substantial reductions in lateral deformation. In order to have a noticeable effect on performance, substantial stiffening is typically required. In some cases it may be possible to accomplish this by converting some beam-column connections that were not originally connected for moment-resistance, into moment-resisting connections. If this is done, care must be taken to ensure that the beams and columns are adequate for the stresses induced by this approach. The most effective way to increase the stiffness of a WSMF structure is to add braced frames and/or shear walls to the seismic force resisting system.

Although global stiffening is effective in reducing the amount of deformation induced in a structure due to its earthquake response, it also typically results in some increase in the level of forces delivered to the structure and its nonstructural components. When evaluating the performance of the upgraded structure, it is important to evaluate all elements, including those that were determined to be adequate prior to the upgrade, as the additional forces delivered to these elements by the stiffened structure may result in poorer performance than previously indicated in evaluations of the performance of the existing structure, without such upgrades.

FEMA-273 provides modeling guidance and acceptance criteria for bracing and shear wall elements used to structurally stiffen a steel moment-frame structure. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of *FEMA-273*. The performance of WSMF elements of the structure, including connections, columns and column splices shall be evaluated using the procedures of Chapter 3. If the new stiffening elements have been designed in accordance with the guidelines of FEMA-273, it may be presumed that a 90% level of confidence with regard to global building behavior can be attained. If desired, the user may confirm the adequacy of global performance of the upgraded structure, using the procedures of Appendix A of this document to determine the global drift capacity.

5.4.4 Global Structural Strengthening

Typically, WSMF structures do not exhibit poor performance as a result of inadequate strength to resist lateral forces. Rather, they exhibit poor performance because they are excessively flexible, have excessive irregularities or have vulnerable details and connections. However, if a performance evaluation of a WSMF structure indicates inadequate performance due to a global lack of adequate ability to resist lateral forces, such as those produced by ground shaking, strengthening of the structure can be achieved by many of the same means used for structural stiffening, as indicated in Section 5.4.3. In addition, global strengthening can be achieved by cover plating members of the lateral-force-resisting system in order to provide them with additional strength. When global strengthening is performed, the building, including structural and nonstructural elements, is likely to experience greater forces. Therefore, when evaluating the performance of the upgraded structure, it is important to evaluate all elements, including those that were determined to be adequate prior to the upgrade, as the additional forces delivered to these elements by the stiffened structure may result in poorer performance than previously indicated in evaluations of the performance of the existing structure, without such upgrades.

FEMA-273 provides modeling guidance and acceptance criteria for bracing and shear wall elements used to structurally stiffen or strengthen a WSMF structure. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of *FEMA-273*. The performance of elements of the structure, including connections, columns and column splices shall be evaluated using the procedures of Chapter 3. If the new strengthening elements have been designed in accordance with the guidelines of *FEMA-273*, it may be presumed that a 90% level of confidence with regard to global building behavior can be attained. If desired, the user may confirm the adequacy of global performance of the upgraded structure, using the procedures of Appendix A to determine the global drift capacity.

Commentary: Since WSMF structures are anticipated to exhibit significant response within the inelastic range, it can be difficult to determine if the inability of a structure to provide adequate performance is a result of inadequate strength as opposed to stiffness. Generally, global structural strength is closely related to a structure's ability to provide Immediate Occupancy performance, while global stiffness is more closely related to Collapse Prevention performance. An inability of a structure to provide adequate confidence of achievement of Collapse Prevention performance will usually be most effectively mitigated through addition of structural stiffness, rather than strength. Similarly, an inability of a structure to provide adequate confidence of achievement of Immediate Occupancy performance can often best be addressed through addition of global structural strengthening.

5.4.5 Mass Reduction

The reduction of mass in a structure can improve its performance in several ways. One effect of mass reduction is a decrease in the periods of vibration of the structure. Since buildings of decreased period generally exhibit lower deformation response than do buildings of longer period, this results in decreased deformation and damage. The seismic forces experienced by a structure are proportional to the acceleration induced by the earthquake and the structure's mass. By reducing the structure's mass it is possible to reduce directly the amount of seismic force induced in the structure, which also reduces the potential damage.

Methods of reducing the mass of a steel moment-frame structure can include: replacement of heavy exterior cladding systems with lighter systems; removal of unused equipment and storage loads; replacement of masonry partition walls with lighter systems; and removal of one or more stories. As with other upgrade techniques, a complete re-evaluation of the upgraded structure's performance should be conducted, following development of an upgrade alternative.

Commentary: The most beneficial effect of mass reduction as an upgrade strategy is that it leads to a shortening of the structural period, and a corresponding reduction in the spectral displacement demand on the structure, produced by typical earthquake ground motions. However, period is related to mass through a square root relationship. Thus, substantial reductions in mass are necessary to have a meaningful effect on lateral displacement demand.

5.4.6 Seismic Isolation

Seismic isolation is a relatively new method of improving the seismic performance of an existing structure. Seismic isolation improves structural performance through two basic effects. First, it is used to significantly lengthen the period of the structure, potentially in combination with the introduction of significant damping. The combined effect of the change in the structure's period and the introduction of supplemental damping results in greatly reduced seismic inertial forces on the building. Isolation systems are also typically designed such that they are more flexible than the supported structure, such that most of the earthquake induced deformation and energy dissipation is accommodated within the isolation system, rather than being transmitted to the structure. The result is that the components of the isolation system experience very large deformation and energy dissipation demands, while the structure above the isolation system sees relatively low levels of seismic induced lateral forces and deformations, and therefore, low levels of damage.

Seismic isolation tends to be most effective as an upgrade measure when a relatively heavy and stiff superstructure is mounted on relatively flexible isolators. Typically the period of the isolated structure (including the isolation system) is on the order of 2 to 3 seconds. Isolation is most effective when the initial period of the non-isolated structure is on the order of 1 second or less. Since most steel moment-frame (WSMF) structures have periods in excess of 1 second, this will not often be an effective method of upgrading WSMF structures, unless it is combined with supplemental global stiffening of the structure.

FEMA-273 provides modeling guidelines and acceptance criteria for isolation systems for use in performance evaluation of isolated structures. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of *FEMA-273*. The performance of elements of the structure shall then be evaluated using the procedures of Chapter 3, with the mathematical model modified to include the effects of the upgrade elements on structural response. For purposes of performance evaluation, the interstory drift of the isolation system shall be neglected. Global interstory drift demand shall be taken as the maximum of the interstory drifts predicted for the superstructure, considering the effects of the isolation system in the model.

> *Commentary: Performance evaluation conducted in accordance with the procedures of Chapter 3 uses maximum predicted interstory drift demand as one of the primary parameters evaluated. The primary effect of base isolation is to substantially reduce the interstory drift demand within the structure. The base isolation system should be designed in accordance with the procedures of FEMA-273. The performance of the superstructure should be evaluated using the procedures of Chapter 3 and taking the interstory drift demand as that predicted for the frame, in an analysis in which the base isolation system as well as the frame is modeled.*

5.4.7 Supplemental Energy Dissipation

The intent of seismic upgrades employing supplemental energy dissipation devices, also called dampers, is to reduce the amount of deformation induced in the structure during its response to ground shaking. In this respect, it is similar to upgrades accomplished through global structural stiffening. However, rather than introducing stiffening to a structure, this upgrade technique reduces deformation through the dissipation of energy within a series of devices that are introduced into the structure as part of the upgrade. The effect of this dissipated energy is to increase the structure's effective damping, and thereby, to reduce its lateral displacement response.

A number of different types of energy dissipation devices are commercially available. These include fluid-viscous dampers, visco-elastic dampers, friction dampers, and hysteretic dampers. Each of these devices has unique force-displacement-velocity relationships, and therefore affect the structure's response in a somewhat different manner.

The energy dissipated by a damping device is the integrated product of the amount of force the device exerts on the structure (or is exerted on the device by the structure) and the distance through which this force acts. In many ways, welded steel moment-frame structures are ideal candidates for upgrades employing energy dissipation devices because they are inherently flexible structures permitting damper elements to dissipate large amounts of energy at relatively low force levels. This is important because large damper forces can create large concentrated forces in the structure.

Energy dissipation devices are typically introduced into a structure as part of a braced frame, where the devices are either introduced in series with the braces in the frame, or actually serve as the braces in the frame. Upgrades using this strategy should be conducted by designing the upgrade elements using the guidelines of *FEMA-273*. The performance of elements of the structure should then be evaluated using the procedures of Chapter 3, with the mathematical model modified to include the effects of the upgrade elements on structural response.

5.5 As-Built Conditions

5.5.1 General

Prior to performing an upgrade design, sufficient information on the configuration and material properties of the existing structure must be obtained to permit a detailed evaluation, in accordance with Chapter 3. Refer to Chapter 2 for criteria on obtaining as-built information.

Quantification of in-place material properties and verification of the existing system configuration and condition are necessary to analyze or evaluate a building. Chapters 2 and 3 identify properties requiring consideration and provide criteria for their acquisition. Condition assessment is an important aspect of planning and executing seismic upgrade of an existing building. One of the most important steps in condition assessment is a visit to the building for visual inspection.

The extent of in-place materials testing and condition assessment that must be accomplished is related to availability and accuracy of construction and as-built records, the quality of materials used and construction performed, and the physical condition of the structure. Data such as the properties and grades of material used in component and connection fabrication may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

5.5.2 Material and Section Properties

Material and section properties of existing components shall be determined in accordance with the criteria outlined in Chapter 2.

5.6 Upgrade Components

New components, constructed as part of upgrades of existing WSMF structures shall conform to the requirements of this section.

5.6.1 Material Specifications

Structural steel should conform to the specifications and grades permitted by the building code, unless a project-specific qualification testing program is performed to demonstrate acceptable performance of alternative materials.

5.6.2 Material Strength Properties

The *AISC Seismic Provisions* (AISC, 1997) state:

> *When required by these provisions, the required strength of a connection or related member shall be determined from the Expected Yield Strength F_{ye} of the connected member, where*

$$F_{ye} = R_y F_y \qquad (5\text{-}1)$$

The Provisions further state that *"R_y shall be taken as 1.5 for ASTM A36 and 1.3 for A572 Grade 42. For rolled shapes and bars of other grades of steel and for plates, R_y shall be taken as 1.1. Other values of R_y are permitted to be used if the value of F_{ye} is determined by testing that is conducted in accordance with the requirements for the specified grade of steel."*

ASTM has recently issued a new specification, A992, for structural steel shape. This specification is similar to the ASTM A572 specification for Grade 50 steels, except that more restrictive limits apply to the permissible variation in yield strength, the ratio of yield to tensile strength and certain other properties, than contained in ASTM A572. This material specification was specifically developed by the steel industry in response to concerns raised by structural engineers with regard to the large variations in properties inherent in the A572 specification, and the difficulties this presented with regard to design for inelastic behavior and seismic resistance. The A992 material will eventually become the recommended basic grade of steel for use in seismic force resisting systems. Since material has only recently been produced under this specification, statistical data on the actual variation of strength properties produced by the mills is not currently available. Until such data does become available, use of the R_y values indicated for ASTM A572, Grade 50 is recommended.

5.6.3 Mathematical Modeling

The stiffness and strength of upgrade elements shall be included in the mathematical model using the same criteria provided for modeling of existing elements as outlined in Chapter 3.

6. Connection Qualification

6.1 Scope

This chapter provides performance qualification data for various types of connections, together with criteria for analysis and design of connections for the upgrade of existing steel moment-frame (WSMF) structures. Included herein are general criteria that are generic to most connection upgrade types, and recommendations for specific connection upgrade details of connections intended to be prequalified for use in seismic upgrades. 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 procedures for qualification of connections and connection upgrades, which have not been prequalified or are proposed for use outside the limits of their prequalification as set forth herein.

Commentary: The 1988 Uniform Building Code (ICBO, 1988) introduced a single pre-qualified moment connection design, 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 1970's, and the belief that this connection was capable of providing acceptable strength and ductility for service in all frames that otherwise met the provisions of the building code. The UBC pre-qualified connection was subsequently adopted into the 1992 AISC Seismic Provisions and then into model codes nationwide. Although the building codes did not formally adopt the pre-qualification of this standard connection until the late 1980s and early 1990s, this connection detail had seen widespread use in WSMF construction since the 1970s.

The discovery of many fractures in buildings incorporating this standard detail, following the Northridge earthquake, demonstrated the ineffectiveness of the pre-qualified connection as it was being used in modern practice. Subsequent research conducted under this project, and by others, has demonstrated that many types of connections that have the strength to develop the plastic moment capacity of the connected elements, do not have the capability to do so in a ductile manner over repeated cycles of loading. Further, this research has shown that inelastic deformation demands in some frame structures can be significantly larger than those that have historically been presumed as the basis for the codes.

Following the 1994 Northridge earthquake, the pre-qualified connection contained in the building code was deleted by means of an emergency code change. In its place, a provision was substituted requiring that the designer demonstrate that whatever connection was used is capable of sustaining the necessary inelastic deformation demands. Qualification of this capacity was by prototype testing. In the time since, a significant number of connection assemblies have been tested, allowing new prequalifications to be developed.

Those prequalifications that are applicable to the upgrade of existing structures appear in this document.

Although a number of prequalified connection upgrades are available, it is conceivable that designers may wish to utilize other connection upgrade designs or to use a pre-qualified design under conditions that are outside those for which they have been prequalified. In these cases, a project-specific, qualification-by-test procedure is still required. The requirements for such a qualification procedure are also given in this chapter.

Finally, this chapter presents qualification and modeling data needed for the assessment of performance of the typical pre-Northridge style connection and of various types of simple gravity connections, for use in performance evaluation of existing structures.

6.2 Performance Data for Existing Connections

This section provides modeling criteria and performance data for use in assessing the performance of existing moment-resisting and simple connections typically found in existing welded steel moment-frame buildings. These connections are not prequalified for use in the lateral-force-resisting systems of new structures. For each connection type, the following quantities are defined:

θ_{SD} = median total connection drift angle at which strength degradation occurs, radians. For existing brittle connections, this corresponds to the median estimate of drift angle at which brittle fracture initiates

θ_{IO} = median drift angle capacity for Immediate Occupancy performance, radians

θ_U = median drift angle at which connection looses gravity load carrying ability, used as the limit state for Collapse Prevention performance

ϕ = a resistance factor applied to θ_{IO}, or θ_U, as appropriate

6.2.1 Welded Unreinforced Fully Restrained Connection

The data contained in this section applies to performance evaluation of existing buildings with the typical welded, unreinforced, moment-resisting connection, commonly present in WSMF buildings constructed prior to the 1994 Northridge earthquake. Figure 6-1 presents a detail for this connection. It is characterized by rolled wide flange beams connected to the strong axis of wide flange column sections, with the connection of the beam flanges to column flange through complete joint penetration (CJP) groove welds. Welding has typically been performed using the Flux Cored Arc Welding process and with weld filler metals without specific rated notch toughness. Weld backing and weld tabs are commonly left in place. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. In some forms of the connection, there are supplemental welds of the shear tab to the beam. Doubler plates, reinforcing the shear capacity of the column panel zone, and beam flange continuity plates at the top and bottom of the panel zone may or may not be present.

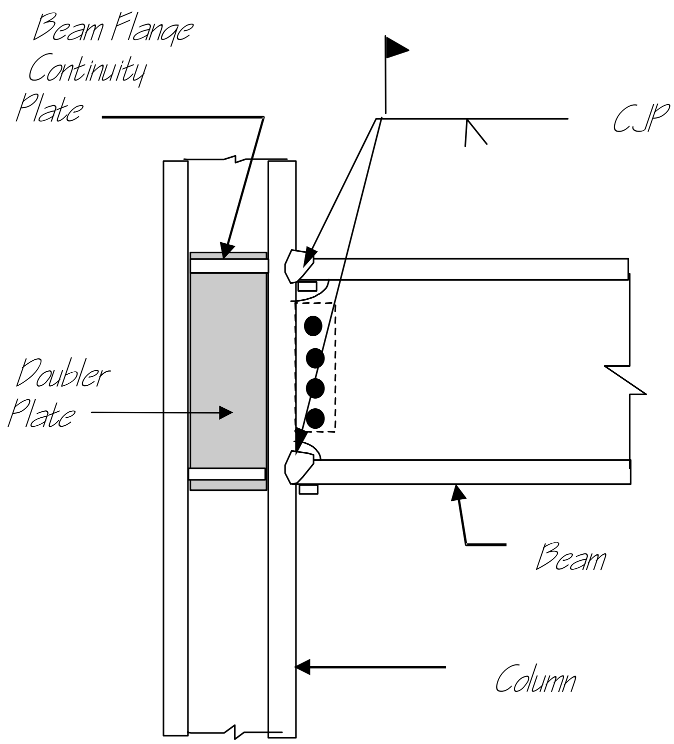


Figure 6-1 Welded Unreinforced Fully Restrained Connection (pre-1994)

Commentary: The data presented in this section is not specifically applicable to forms of this connection that employ weld metals with significant notch toughness. Some older buildings, particularly those erected prior to about 1964, may have welds deposited by the Shielded Metal Arc Welding (SMAW) process. Some such welds may have significant notch toughness, on the order of 40 ft-lbs at normal service temperatures. Limited testing of such connections indicates that they may have better inelastic deformation capacity than do connections employing weld material with lower notch toughness. Refer to Section 6.6.1 for data on connections with notch-tough weld metal.

The performance data provided in this section also is not specifically applicable to forms of the connection in which the beam web is directly welded to the column flange. Limited testing of such connections indicates that they are capable of providing somewhat better inelastic deformation capacity than similar connections with bolted beam webs. However, there are not sufficient data available to permit separate performance qualification of this connection type. The performance data provided herein may be conservatively applied to that connection type, or alternatively, project-specific qualification testing of such connections may be performed.

The connection performance data contained herein has been based on testing of connection assemblies in which the beams are connected to the major axis of *the column. Connections in which beams are connected to the minor axis of columns are known to have similar, and perhaps, more severe vulnerability than major axis connections. However, insufficient data are available to permit quantification of this performance. Connections employing box columns are beyond the scope of this section.*

6.2.1.1 Modeling Assumptions

6.2.1.1.1 Linear Analysis

Elastic analysis models of structures with Welded Unreinforced 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 3.5.2.2.

6.2.1.1.2 Nonlinear Analysis

Nonlinear analysis models of structures with Welded Unreinforced 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 the connection panel zone, the beam or the column yields, or a total interstory drift angle θ_{SD}, from Table 6-1 is reached. The expected yield strength of the material, as indicated in Section 2.5 should be used to calculate the yield capacity of beams, columns, and panel zones. If yielding occurs at total interstory drift angles less than θ_{SD}, the yielding element should be assumed to exhibit plastic behavior. At interstory drifts greater than θ_{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 θ_U, obtained from Table 6-1, occurs. At interstory drift angles greater than θ_U, the connection should be presumed to have negligible strength.

6.2.1.2 Performance Qualification Data

Table 6-1 presents the applicable performance qualification data for welded unreinforced fully restrained moment-resisting connections, conforming to typical practice prior to the Northridge earthquake.

6.2.2 Simple Shear Tab Connections – with Slabs

The data contained in this section applies to the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design, and when concrete slabs are present. Figure 6-2 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. A concrete floor slab, or slab on metal deck is present at the top flange of the beam.

**Table 6-1 Performance Qualification Data –
Welded Fully Restrained Connection (pre-1994)**

Data Applicability Limits	
Hinge location distance s_h	At distance $d_b/3$ from face of column, unless shear strength of panel zone is less than shear corresponding to development of the flexural strength of beams at the connection, in which case, the hinge location should be taken at the column centerline.
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - θ_{SD}, radians	0.061-0.0013d_b
Immediate Occupancy rotation - θ_{IO}, radians	0.01 radian, but not greater than θ_{SD}
Resistance factor, Immediate Occupancy, ϕ	0.8
Collapse Prevention drift angle - θ_U – radians	0.053-0.0006d_b
Resistance factor, Collapse Prevention, ϕ	0.8

Notes: d_b= beam depth, inches

Commentary: Although shear tab connections of the type shown in Figure 6-2 are not typically included in design calculations as part of the lateral-force-resisting system, research conducted in support of these Recommended Criteria (FEMA-355D) indicates that they are capable of providing both non-negligible strength and stiffness. Since the typical steel moment-frame structure will have many such connections, the presence of these connections converts the gravity load framing into a highly redundant reserve system to provide additional stiffness and strength for the building after the primary system comprised of fully restrained connected framing has been damaged.

When these connections are loaded such that the top beam flange acts in compression, the slab can act compositely with the beam. When this behavior occurs, the slab will bear against the column and significant moments can develop through a couple consisting of the slab in compression and the shear tab in tension. This behavior is limited by local crushing of the slab in compression, which behavior initiates at moderate interstory drift angles. Following crushing *of the slab, the connections acts as if the slab were not present, and provides relatively modest flexural resistance until very large rotations. Ultimately, at very large rotations, the beam compressive flange will bear against the column, again resulting in development of large moments. Since the beam flange does not crush, this typically results in failure of the shear tab, in tension.*

The criteria for modeling these connections, presented here, neglects the effect of the slab as described above. This is because this behavior occurs only for one direction of loading, and also, because at large deformations, this behavior degrades. However, nothing in this document would preclude more accurate modeling of these connections, that accounts for the slab effects. FEMA-355D provides information that may be useful for this more complex modeling.

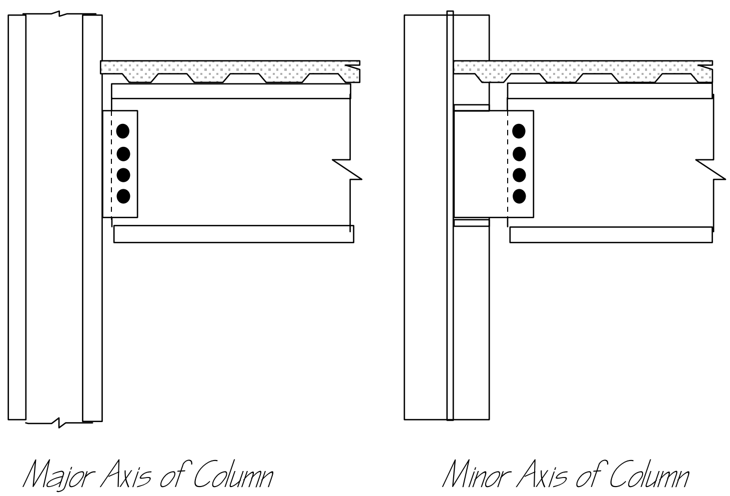


Figure 6-2 Typical Simple Shear Tab Connection with Slab

6.2.2.1 Modeling Assumptions

When included in the analytical model used to predict earthquake induced demands, the stiffness and hysteretic characteristics of framing with simple shear tab connections should be taken in accordance with the recommendations of this section.

6.2.2.1.1 Linear Analysis

The connection stiffness should be explicitly modeled as a rotational spring that connects the beam to the column. The spring stiffness, K_θ should be taken as:

$$K_\theta = 28000\left(d_{bg} - 5.6\right) \tag{6-1}$$

where d_{bg} is the depth of the bolt group in inches and K_θ 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} taken as:

$$EI_{eq} = \frac{1}{\dfrac{6h}{l_b^2 K_\theta} + \dfrac{1}{EI_b}} \tag{6-2}$$

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

6.2.2.1.2 Nonlinear Analysis

The connection should be explicitly modeled as an elastic-perfectly-plastic rotational spring. The elastic stiffness of the spring should be taken as given by Equation 6-1. 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.

6.2.2.2 Performance Qualification Data

Table 6-2 presents the applicable performance qualification data for shear tab connections of beams to columns, with slabs present.

6.2.3 Simple Shear Tab Connections – Without Slabs

The data contained in this section applies to the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design and slabs are not present. Figure 6-3 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. Diaphragms may not be present, and if present consist of wood sheathing, unfilled metal deck, or horizontal steel bracing.

Commentary: Shear tab connections without slabs present behave in a very similar manner to shear tabs with slabs, except that the composite behavior with the slab discussed in the previous section does not occur. Since the modeling criteria for connections with slabs neglect the strength contribution of the slab, the criteria presented herein for connections without slabs are essentially identical to those presented in the previous section.

Table 6-2 Performance Qualification Data – Shear Tab Connections with Slabs

Data Applicability Limits	
Hinge location distance s_h	at center line of bolts
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - $\theta_{SD,}$ radians	0.039-0.0002d_{bg}
Immediate Occupancy rotation - θ_{IO}, radians	0.025, but not greater than θ_{SD}
Resistance factor, Immediate Occupancy, ϕ	0.90
Collapse Prevention drift angle - θ_U – radians	0.16-0.0036d_{bg}
Resistance factor, Collapse Prevention, ϕ	0.80

Note: d_{bg} = bolt group depth, measured from center of top bolt to center of bottom bolt, inches

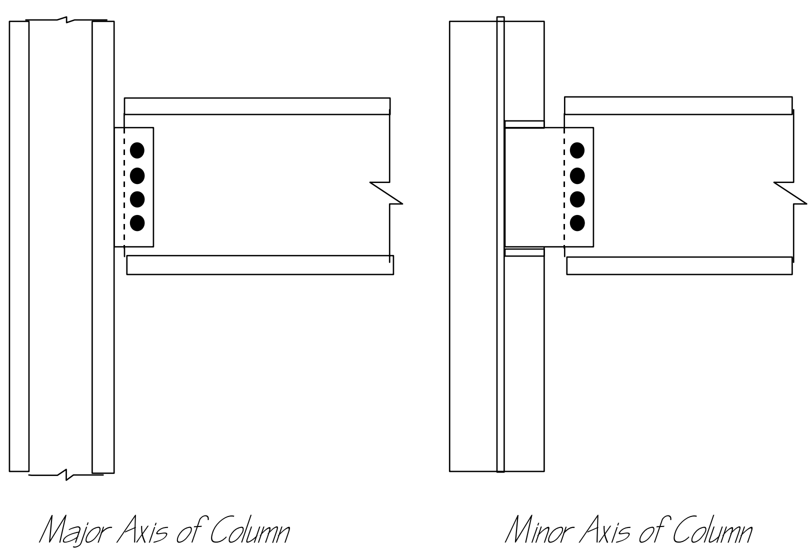


Figure 6-3 Typical Simple Shear Tab Connection Without Slab

6.2.3.1 Modeling Assumptions

Shear tab connections without slabs present should be modeled the same as shear tab connections with slabs present, as indicated in Section 6.2.2.1, except that for nonlinear analysis, performance qualification data shall be as indicated in Table 6-3.

6.2.3.2 Performance Qualification Data

Table 6-3 presents the applicable performance qualification data for shear tab connections of beams to columns, without slabs present.

Table 6-3 Performance Qualification Data – Shear Tab Connections (No Slab)

Data Applicability Limits	
Hinge location distance s_h	At center line of column
Maximum beam size	Unlimited
Beam material	A36, A572, Gr. 50
Maximum column size	Unlimited
Column steel grades	A36, A572, Gr. 50
Performance Data	
Strength degradation rotation - θ_{SD}, radians	0.16-0.0036d_{bg}
Immediate Occupancy rotation - θ_{IO}, radians	0.030, but not greater than θ_{SD}
Resistance factor, Immediate Occupancy, ϕ	0.90
Collapse Prevention drift angle - θ_U – radians	0.16-0.0036d_{bg}
Resistance factor, Collapse Prevention, ϕ	0.80

Note: d_{bg} = bolt group depth, measured from center of top bolt to center of bottom bolt, inches

6.3 Basic Design Approach for Connection Upgrades

This section provides recommended criteria on basic principles of connection upgrade design, including selection of an appropriate connection upgrade detail, estimation of locations of inelastic behavior (formation of plastic hinges), determination of probable plastic moment at hinges, determination of shear at the plastic hinge, and determination of design strength demands at critical sections of the assembly. The designer should utilize these basic principles in the calculations for all connection upgrades, unless specifically noted otherwise in these *Recommended Criteria*.

6.3.1 Frame Configuration

Upgraded frames should be proportioned and detailed so that the required drift angle of the frame can be accommodated through elastic deformation and the development of plastic hinges at pre-determined locations within the frame. Figure 6-4 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 type FR connections, 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. Other locations at which 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 shear tab type framing connections, within the column panel zone, or within the column. The total interstory drift angle, as used in these *Recommended Criteria* is equal to the sum of the plastic drift, as described herein, and the elastic interstory drift.

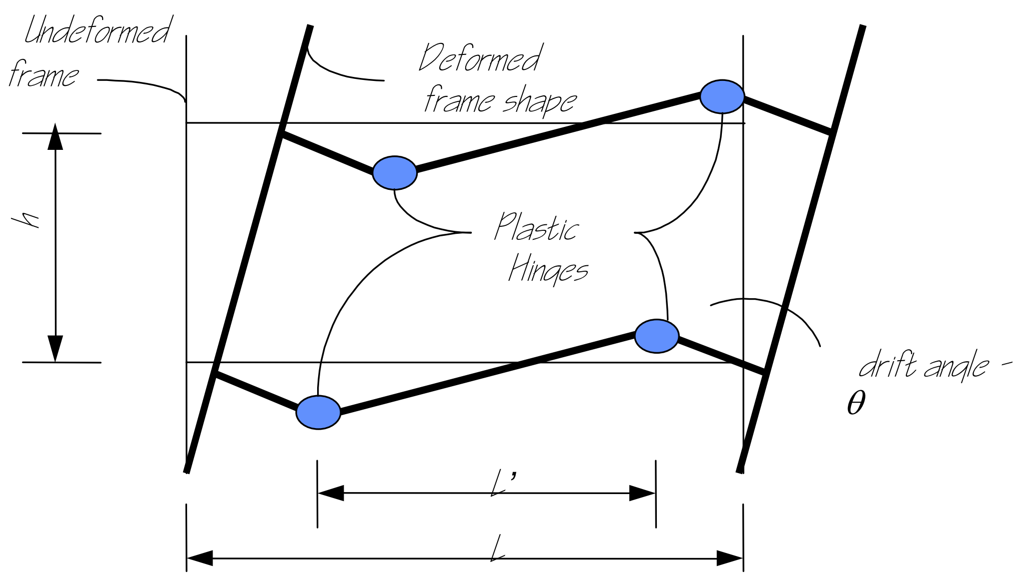


Figure 6-4 Inelastic Behavior of Frames with Hinges in Beam Span

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, which 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 generally 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 based on the development of plastic hinges within the beams at the face of the column, or within the column panel zone. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition that can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in large strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle failure.

Welded steel moment-frame 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 avoid a strong beam-weak column design that can lead to column hinging and story collapse mechanisms. Further, beam-column connections should be configured to force the inelastic action (plastic hinge) away from the column face, where its performance is less dependent on the material and workmanship of the welded joint. This can be done either by local reinforcement of the connection, or local reduction of 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, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done through reinforcement of the connection, 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.

Many existing WSMF structures were not configured in the original design to produce a strong-column, weak-beam condition. In these structures, connection upgrades that reinforce the beam section locally at the connection, to shift the location of plastic hinging into the beam span, will have little effect, as plastic behavior of the frame will be controlled through plastic hinging of the columns. In such structures, upgrade should include strengthening of the columns with cover plating or other similar measures, or alternatively, the provision of supplemental lateral force resisting elements such as braced frames or shear walls. Upgrade recommendations are discussed in Chapter 5.

Connection upgrades 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 upgraded as described in this document should experience many fewer 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 concurrent 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 connection fracture damage of the types experienced in the Northridge earthquake. The primary difference is that life safety protection will be significantly enhanced and most upgraded structures 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 upgrade systems should be considered, which will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, shear walls, energy dissipation systems, base isolation systems, and similar structural systems.

6.3.2 Required Drift Angle Capacity

For systematic upgrade design, the required drift angle capacity of connection assemblies should be sufficient to withstand the total (elastic and plastic) interstory drift likely to be induced in the frame by earthquake ground shaking, as predicted by analysis, while providing sufficient confidence with regard to achievement of the desired performance, in accordance with the procedures of Chapter 3. Section 6.6 provides data on the drift angle capacity of several prequalified connection upgrade details, together with design guidelines for these connection upgrades and limits on the applicability of the prequalification. Section 6.7 provides performance data for several types of moment-resisting connections that have been prequalified for use in new steel moment-frame construction. Section 6.8 provides descriptive information on several types of proprietary connection technologies that may be considered for seismic upgrade applications. Section 6.9 provides recommended criteria for determining the factored drift angle capacity of connection upgrades that are not prequalified.

For the purposes of Simplified Upgrade, frames shall be classified either as Ordinary Moment Frames (OMF) or Special Moment Frames (SMF) and connection upgrade details that are prequalified for the appropriate system, as indicated in Section 6.6 of these guidelines, should be selected. For purposes of simplified upgrades, a frame should be considered an SMF system if the construction documents indicate it was designed as a Special Moment Resisting Frame, a Ductile Moment Resisting Frame, or if the original design documents indicate that any of the design values indicated in the column labeled “SMF” in Table 6-4 were used in determining the design seismic forces for the frame in the original design. A frame should be considered an OMF if the design documents indicate it was designed as an OMF or if any of the design values indicated in the column labeled "OMF" in Table 6-4 were used in determining the design seismic forces for the frame in the original design. If sufficient documentation is not available to permit determination of the original intended system for the structure, an SMF should be assumed.

Table 6-4 Design Coefficients for SMF and OMF Systems

Design Coefficient	OMF	SMF
K (buildings designed to 1985 or earlier edition of UBC, or 1990 or earlier editions of BOCA or SSBC.)	1.0	0.67
R_w (buildings designed to UBC editions 1988 - 1994)	6	12
R (buildings designed to 1997 UBC, or 1993 or later editions of BOCA or SSBC.)	4	8

Commentary: In Systematic Upgrades, a complete analysis of the structure is performed, in accordance with the criteria of Chapter 3. In this analysis, an estimate is developed of the forces and deformations induced by response to earthquake ground shaking, and based on these estimated forces and deformations, and the estimated capacity of the frame and its individual components to resist these demands, a level of confidence with regard to the ability of the frame to provide desired performance is estimated.

In Simplified Upgrades, performance evaluation of the structure, in accordance with Chapter 3, is not performed. Rather than providing a specific level of confidence that the structure is capable of a particular performance, simplified upgrades are intended only to provide the structure with the level of reliability implicitly presumed by the code provisions under which it was originally designed. Until recently, the building codes only recognized two types of moment-resisting steel frame systems: a system with significant intended inelastic response capability called either a Special Moment Frame, or in some codes, a Ductile Moment-Resisting Frame; and frames having only limited inelastic response capability, typically called an Ordinary Moment Frame.

Table 6-4 classifies framing systems, using the terminology contained in the 1997 NEHRP Recommended Provisions for New Buildings and 1997 AISC Seismic Design Specification, as either an SMF or an OMF.

In addition to these two categories of moment-resisting frames, some steel moment-resisting frames are part of a dual structural system, in which the frames provide a secondary system of lateral-force resistance for a primary system *comprised of braced frames or shear walls. Upgrade of such structures, using the Simplified procedure is not recommended.*

6.3.3 Connection Configuration

For Simplified Upgrade, a connection upgrade configuration should be selected that is compatible with the appropriate structural system. No further qualification of the design is necessary, other than to ensure that the connection configuration does not create any of the following conditions, as defined in the building code, or make an existing such condition more severe:

a. Weak column - strong beam

b. Weak story

c. Soft story

d. Torsional Irregularity

For Systematic Upgrade, a connection configuration that is capable of providing sufficient factored drift angle capacity to provide a suitable level of confidence should be selected. Section 6.6 presents data on a series of prequalified connection upgrade details, from which an appropriate detail may be selected. These connection upgrades details are prequalified for use within certain ranges of member sizes and frame configuration. If these connection upgrade details are to be employed outside the range of applicability, project specific connection qualification should be performed. If project-specific connection qualification is to be performed, a connection of any configuration may be selected and qualified for acceptability using the procedures of Section 6.9.

6.3.4 Determine Plastic Hinge Locations

Based on the data presented in these *Recommended Criteria* for prequalified connection upgrades, 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 6-5 should be identified. The plastic hinge locations presented for prequalified connection upgrades are valid for beams with gravity loads representing a small portion of the total flexural demand and for conditions of strong column, weak beam. For frames in which gravity loading produces significant flexural stresses in the members, or frames that do not have strong-column, weak-beam configurations, locations of plastic hinge formation should be determined based on methods of plastic analysis.

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, or for frames in which yielding will actually occur in the beam, rather than in the column panel zone or the column itself. If significant gravity load is present, or if panel zones or columns are the weak links in the frame, this can shift the locations of the plastic hinges, and in *the extreme case, 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, as long as beam flexure, rather than panel zone shear, column flexure, or beam shear is the dominant inelastic behavior for the frame. If gravity demands significantly exceed this level then plastic analysis of the girder should be performed to determine the appropriate hinge locations.*

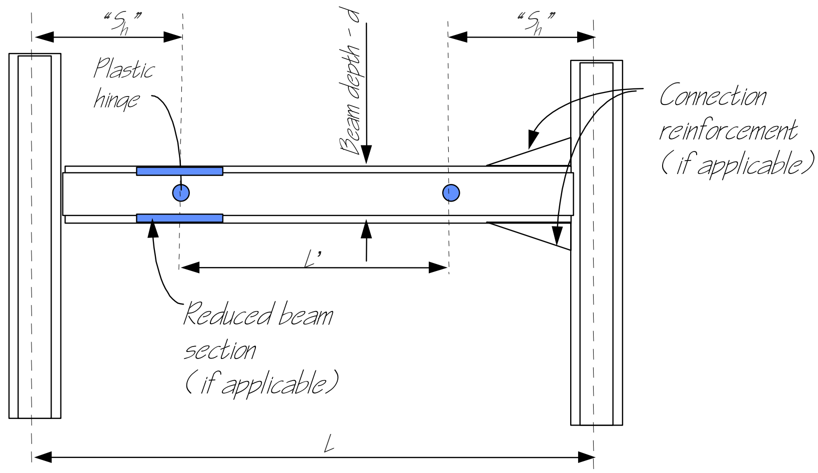


Figure 6-5 Location of Plastic Hinge Formation

6.3.5 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:

$$M_{pr} = C_{pr} R_y Z_e F_y \tag{6-3}$$

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:

$$C_{pr} = \frac{F_y + F_u}{2F_y} \tag{6-4}$$

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 *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, or determined from tests, for the pertinent yield mechanism, considering the variation in material properties of the yielding elements. For prequalified connection upgrades and connections, calculation methods to determine the yield strengths of the various active mechanisms are given in the design procedure accompanying the individual prequalification.

Commentary: The AISC Seismic Provisions use the formulation $1.1R_yM_p+M_v$ for calculation of the quantity ΣM^*_{pb}, which is used in calculations for column strength (strong-column, weak-beam), and for required shear strength of panel zones. As described in FEMA-355D, research has shown that, for most connection types, the peak moment developed is somewhat higher than the 1.1 factor would indicate. Therefore, for these guidelines, the factor C_{pr}, calculated as shown, is used for individual connections, with a default value of 1.2 applicable to most cases.

6.3.6 Determine Shear at the Plastic Hinge

The shear at the plastic hinge should be determined by 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 6-6 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 6.5.1.

6.3.7 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection upgrade, 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 6-7 demonstrates this procedure for two critical sections for the beam shown in Figure 6-6.

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, as well as evaluating shear demands on the column panel zone. 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 conditions. Other critical sections should be selected as appropriate.

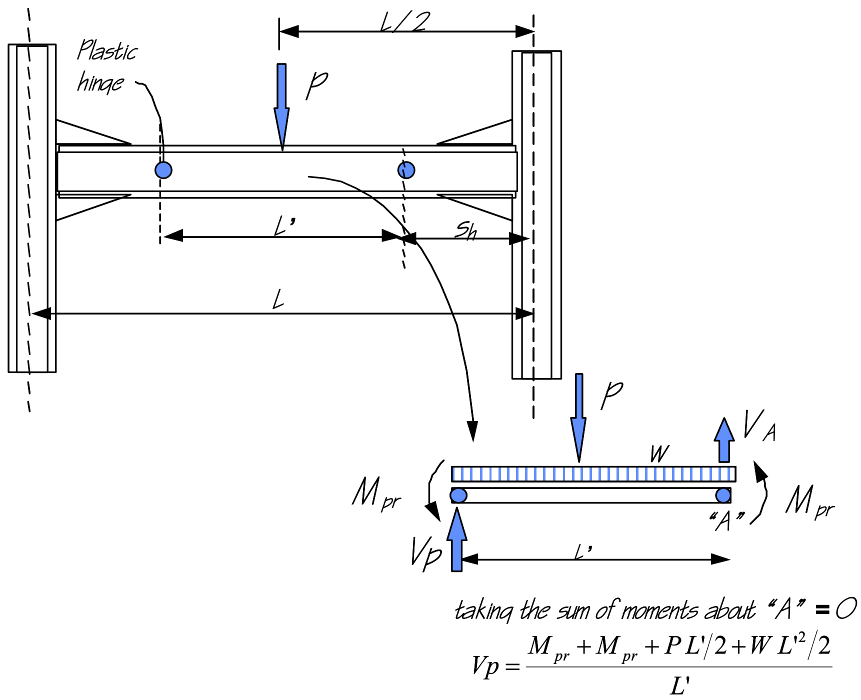


Figure 6-6 Sample Calculation of Shear at Plastic Hinge

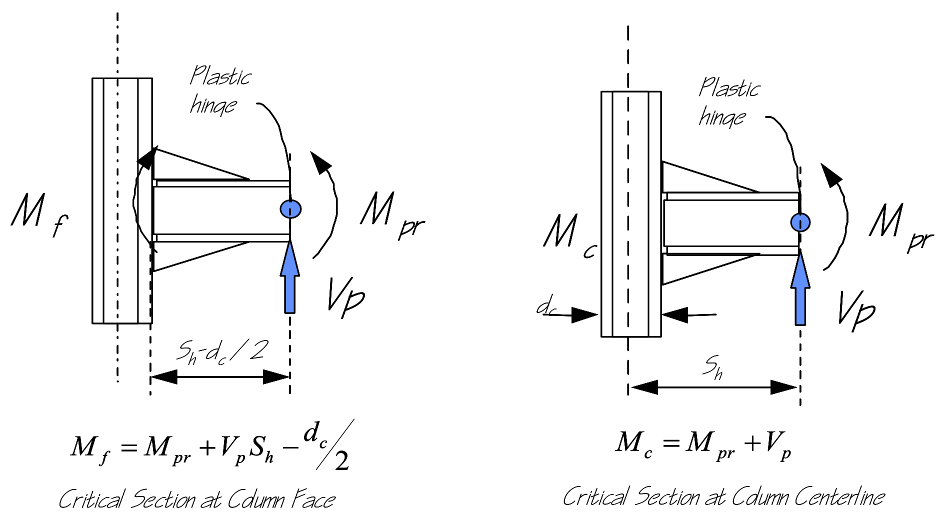


Figure 6-7 Calculation of Demands at Critical Sections

6.3.8 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:

$$M_{yf} = C_y M_f \tag{6-5}$$

where:

$$C_y = \frac{1}{C_{pr}\frac{Z_{be}}{S_b}} \tag{6-6}$$

C_{pr} = the peak connection strength coefficient defined in Section 6.3.5
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.

6.4 General Requirements

This section provides criteria for connection upgrade design conditions that are considered to be general, that is, those conditions which, when they occur in a connection upgrade, 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*.

6.4.1 Framing

6.4.1.1 Beam and Column Strength Ratio

For multistory SMF systems, frames should be configured with a strong-column, weak-beam configuration, to avoid the formation of single-story mechanisms. As a minimum, Equation 9-3 of *AISC Seismic Provisions* should be satisfied. In the application of Equation 9-3, the quantity M_c as defined in Section 6.3.7 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-Δ instability and collapse.

Building codes permitted frames to be designed with weak-column, strong-beam configurations until 1988. Therefore, many existing steel moment-frame buildings have such configuration. Further, some types of connection upgrades, through local strengthening of the beam ends, have the potential to create weak-

column, strong-beam systems in frames that originally did not have such configuration. Although weak-column, strong-beam designs are not desirable, AISC Seismic does permit their use under certain conditions, even for SMF systems. Before utilizing weak-column, strong-beam configurations, designers should be aware that the prequalified connections for SMF systems contained in these Recommended Criteria are based on tests using strong columns.

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 big problem, provided that the columns are designed as compact sections, are properly braced and axial loads are not too high. It is well understood that a column hinge will form at the base of columns that are continuous into a basement, or that are rigidly attached to a stiff and strong foundation.

6.4.1.2 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 *AISC Seismic Provisions*. For moment frame beams with Reduced Beam Section (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 the 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. Widespread buckling of beam flanges in a moment resisting frame can result in development of frame strength degradation increasing both story drifts and the severity of P-Δ effects and therefore should be avoided. Local flange buckling results in very 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 the AISC Seismic Provisions, except as specifically modified by individual connection prequalifications or qualification tests.*

6.4.1.3 Beam Web Stability

Moment-frame beams should be selected that have web height-to-thickness ratios, h_c/t_w of not greater than $418/\sqrt{F_y}$.

Commentary: The AISC Seismic Provisions permits use of beams with web h_c/t_w up to as high as $520/\sqrt{F_y}$, for beams without axial load. Most of the testing under this project has been conducted on beams such as W30x99 and W36x150, both of which barely conform to $h_c/t_w \leq 418/\sqrt{F_y}$. Since many of the specimens exhibited significant web buckling in the area of plastic hinges, it is not considered prudent to utilize beams with relatively thinner webs in moment frames. Although stiffening of the webs could be done to limit web buckling, it is possible that stiffeners could be detrimental to connection performance. Since connections with web stiffeners were not tested, such connections have not been prequalified. See FEMA-355D, State of the Art Report on Connection Performance, for further discussion of web buckling of moment-frame beams.

6.4.1.4 Beam Span and Depth Effects

The performance of moment-resisting beam-column connections is strongly related both to beam depth and beam span-to-depth ratio. Data accompanying each of the prequalified connection upgrades presented in Section 6.6 includes specification of maximum beam depths and minimum beam span-to-depth ratio. Connection upgrade details presented in Section 6.6 should not be used for cases where beam depth exceeds the indicated limit unless project-specific qualification, in accordance with Section 6.9 is performed. For Simplified Upgrade, connection upgrade details should not be used in cases where the beam span-to-depth ratio is less than the indicated amount unless project-specific qualification, in accordance with Section 6.9, is performed. For Systematic Upgrade, connection upgrade details may be used on beams with spans that have smaller span-to-depth ratio than the limiting value indicated in the prequalification provided that the acceptance criteria used in performance evaluation for interstory drift capacity θ as limited by local connection behavior is modified as indicated by the equation:

$$\theta' = \frac{8d}{L}\left(1 + \frac{L - L'}{L}\right)\theta \tag{6-7}$$

where:

θ' = the median interstory drift angle capacity for connection behavior for beams with small span-to-depth ratio

θ = the median interstory drift angle capacity listed in the prequalification for connection behavior for beams meeting the span to depth limitations of the prequalification

L = the span of the beam, center-line-to-center-line of columns, inches

L' = the effective span of the beam between plastic hinge locations, inches

d = the beam depth in inches

Where the effective span L' of the beam between points of plastic hinging, is such that shear yielding of the beam will occur, rather than flexural yielding, the web of the beam should be stiffened between the points of plastic hinging, and braced as required by the *1997 AISC Seismic Provisions* for long links in eccentric braced frames.

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. Equation 6-7 approximately accounts for these effects. Additional information may be found in FEMA-355D, State of the Art Report on Connection Performance.

6.4.1.5 Beam Flange Thickness Effects

The connection upgrade prequalifications contained in 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 above, 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 ensure their performance, and quality control may be more difficult. Additionally, residual stresses are likely to be higher in thicker material with thicker welds.

6.4.1.6 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 upgrade 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, great care must be taken in detailing and installation of such bracing to ensure that 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.

6.4.1.7 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. Standard arc-spot weld attachments may be made in the hinging area, but shot-in, or screwed attachments should not be permitted.

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 that involve penetration of the flanges in the hinging region.

6.4.2 Welded Joints

6.4.2.1 Through-Thickness Strength

The through-thickness strength demands on existing column material should be limited to the values given in Table 6-5. Through-thickness demands should be calculated as the applied flange force, divided by the projected area of the welded joint on the column flange, using the procedures of Section 6.3.7 to calculate the applied force at this critical section.

Table 6-5 Column Flange Through-Thickness Strength

Column Flange Material Specification	$F_{t\text{-}t}$
Hot rolled wide flange columns conforming to A36, ASTM A572 Grade 50, or ASTM A992, or ASTM A913 rolled later than 1994 and having sulfur content not in excess of 0.05% by weight.	No limit
All other material	$0.8F_u$

Commentary: Early investigations of connection fractures in the 1994 Northridge earthquake identified a number of fractures (types C3 and C5 Section 2.3.2) 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 FR 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 type C3 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 C3 damage occurred when fractures initiated in defects present in the complete joint penetration (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 the C3 damage initially identified as through-thickness failures are likely to have occurred as a result of certain combinations of material strength and 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 showed that due to the restraint inherent in welded beam flange to column flange joints, the through thickness yield and ultimate 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 with controlled sulfur contents, as required by the FEMA-353 Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.

Notwithstanding the above, it is known that in the past, lamellar tearing of thick column flanges occasionally occurred during the fabrication and erection process. This lamellar tearing was a result of high through thickness strains induced by welding on material that had excessive sulfur inclusions. These sulfur inclusions, which were flattened and elongated during the shape rolling process could form planes of weakness within the shape that were susceptible to this tearing. It is known that steel with relatively high sulfur content is more susceptible to this behavior than shapes with lower sulfur contents. Also, it is known that shapes that undergo a significant amount of working during the rolling process are more susceptible as well, as the rolling process tends to flatten the sulfide inclusions and align them in the rolling direction. Modern steel production often uses a continuous casting process in which the steel is cast in a shape that is near that of the final product, resulting in the sulfur being uniformly distributed throughout the shape and therefore less susceptibility to lamellar tearing.

Table 6-5 recommends a limit of $0.8F_u$ *for through-thickness stress on older steels, that may be susceptible to through-thickness tearing, based on a statistical survey of the relationship of through-thickness strength to longitudinal strength for structural steels (Barsom, 1996).*

6.4.2.2 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 toughness of 20 ft-lbs, at 70 degrees 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 specified 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, be checked for notch toughness. 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 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 toughness. However, it is judged that base metal notch 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 Charpy V-Notch (CVN) value of 20 ft.-lbs. at 70 degrees 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. 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 currently requiring 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 notch toughness is required, but testing to verify it on a project basis is not judged to be necessary as it is routinely achieved.

No specific notch toughness requirements are specified for existing materials in steel moment frames. This is because testing of the notch toughness of these materials is costly and difficult and also because there is no practical way to improve the notch toughness of an existing material, other than to replace it. The importance of base material notch toughness with regard to steel moment-frame behavior is not clear, however. High material notch toughness is beneficial in preventing the propagation of minor fractures and flaws into unstable brittle fractures, when such defects are present. However, base metals typically are free *of such defects and therefore, less susceptible to the initiation of the brittle fractures that material notch toughness is effective in preventing.*

6.4.2.3 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 the *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 6.4.3 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 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 practice is frequently changed, it is prudent to assume that all rolled sections are rotary-straightened.

6.4.2.4 Weld Filler Metal Matching and Overmatching

The use of weld filler metals and welding procedures that will produce welds with matching or slightly overmatching tensile strength relative to the connected steel is recommended. Welding consumables specified for Complete Joint Penetration (CJP) groove welds of beam flanges and flange reinforcements should have yield and ultimate strengths at least slightly higher than the expected values of yield and ultimate strength of the beam or girder flanges being welded. Significant overmatching of the 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 electrodes commonly used in structural construction and conforming to the E70 specifications provide adequate overmatching properties for structural steels conforming to ASTM A36, A572, Grades 42 and 50, A913, Grade 50 and A992. Welded splices of columns of

A913-Grade 65 steel should be made with electrodes capable of depositing weld metal with a minimum ultimate 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. 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 weld filler metals. The majority of the successful connection tests have used weld metals with yield and tensile strengths in the range of 58 and 70 ksi respectively, which provide matching to moderate overmatching with beams of Grade 50 steel. For additional information refer to FEMA-355B, State of the Art Report on Welding and Inspection.

6.4.2.5 Weld Metal Toughness

For structures in which the steel frame is normally enclosed and maintained at a temperature of 50°F or higher, critical welded joints in seismic force resisting systems, including complete joint penetration (CJP) groove welds of beam flanges to column flanges, CJP welds of shear tabs and beam webs to column flanges, column splices, and similar joints, should be made with weld 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 in *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. For structures with lower service temperatures than 50°F, 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, which may be preexisting or inherent, or which may be caused by applied or residual stresses. The 1997 AISC Seismic Provisions requires the use of welding consumables with a rated Charpy V-Notch (CVN) toughness of 20 ft.-lbs. at -20 °F, for CJP groove welds used in the Seismic Force Resisting System. Seismic Provisions for Structural Steel Buildings (1997) 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 weld filler metal is as determined by the American Welding Society classification or manufacturer certification.

Studies conducted under this project have indicated that not all weld consumables that are rated for 20 ft-lbs of toughness at –20°F will provide adequate toughness at anticipated service temperatures. The supplemental toughness requirements contained in FEMA-353 are recommended to ensure that weld metal of adequate toughness is obtained in critical joints. Most of the beam-column connection tests conducted under this project were made with weld filler metal conforming to either the E70T6 or E70TGK2 designations. These filler *metals generally conform to the recommended toughness requirements. Other weld filler metals may also comply.*

6.4.2.6 Weld Backing, Weld Tabs, and other Welding Details

Weld backing and runoff tabs should be removed from complete joint penetration 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. It is not recommended that backing and runoff tabs be removed from existing connections in buildings, unless other upgrades or modifications of the affected connections are being made, in which case such removal is recommended.

The following general procedures may be considered for backing removal. Steel backing may be removed either by grinding or by the use of air arc or oxy-fuel cutting. The zone just beyond the theoretical 90-degree intersection of the beam-to-column flange should be removed either by air arc or oxy-fuel cutting followed by a thin grinding disk, or by a grinding disk alone. This shallow gouged depth of weld and base metal should then be tested by magnetic particle testing (MT) to determine if any linear indications remain. If the area is free of indications the area may then be re-welded. The preheat should be maintained and monitored throughout the process. If no further modification is to be made or if the modification will not be affected by a reinforcing fillet weld, the reinforcing fillet may be welded while the connection remains at or above the minimum preheat temperature and below the maximum interpass temperature.

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, a far more significant factor 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.

In new construction, as stated in FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications, or in modification of existing joints conducted as part of an upgrade project, it is recommended that backing be removed from beam bottom flange joints, to allow identification and correction of weld root flaws. This is not recommended for top flange joints because the stress condition at the top flange 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 position, it is far less likely that significant flaws will be incorporated in top flange joints.

Weld tabs represent another source of 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 weld 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 runoff tabs not be, of itself, a cause of further stress concentrations, and therefore, FEMA-353 recommends that the workmanship result in smooth surfaces, free of defects.*

Removal of existing backing and weld tabs as a sole means of building upgrade is not recommended. Laboratory testing demonstrates that existing unreinforced welded type FR connections made with low notch toughness weld metal are incapable of ductile performance, even with the removal of these stress rising features. However, they should be removed as part of any program of more substantial upgrades of connections.

6.4.2.7 Reinforcing Fillet Welds and Weld Overlays

When weld backing is removed, the weld should be reinforced with a fillet weld. The size of the weld should be sufficient to cover the root of the existing Complete Joint Penetration weld, and not less than ¼ -in. The profile of the fillet should be as described in Section 5.4 of *AWS D1.1* with a transition free from undercut, except as permitted by *AWS D1.1*.

One method for improving the performance of existing unreinforced connections with low notch toughness weld metal is to reinforce the existing welded joints with weld overlays. This method, which is described in *FEMA-352 Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings,* is not prequalified for any specific performance capability, though it is known to be capable of some significant performance improvement.

Commentary: Limited testing on the use of built-up welds (overlay welds) as a means of repairing and reinforcing welded connections of smaller-sized beams in existing buildings has been performed. This upgrade technique has not been prequalified with regard to performance capability as insufficient laboratory test data are available at this time to qualify its use and provide the necessary statistical data on its performance.

6.4.2.8 Weld Access Hole Size, Shape, Workmanship

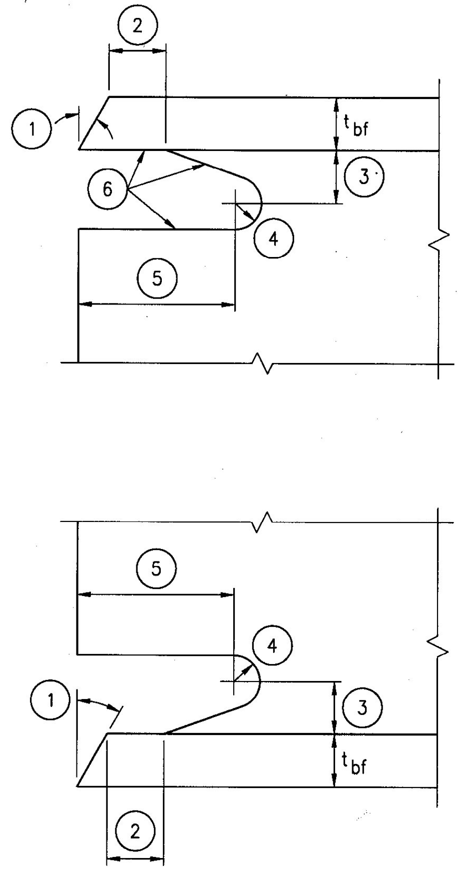
New welded moment-resisting connections should utilize weld-access hole configurations as shown in Figure 6-8, except as otherwise noted in specific details in these *Recommended Criteria*. Criteria for cutting and finishing of weld access holes are provided in *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

Commentary: The size, shape, and workmanship of weld-access holes can affect connection strength 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 weld 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, as a result of the strain concentrations introduced by this feature. The configuration shown in Figure 6-8 was developed as part of the program of research conducted under this project 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.*

6.4.2.9 Welding Quality Control and Quality Assurance

FEMA-353, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* contains 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 inspection are specified depending on the anticipated severity of loading, or demand (Seismic Weld Demand Category) and the consequences 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 document. For the prequalified connection upgrades described in these *Recommended Criteria*, the appropriate categories have been preselected and are designated in information accompanying the prequalification.

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



Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.

Notes:
1. Bevel for groove weld selected.
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" min (± ¼ inch).
4. 3/8" min. 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 6-8 Recommended Weld Access Hole Detail

6.4.3 Other Design Issues for Welded Connections

6.4.3.1 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 6-8 or 6-9:

$$t_{cf} < 0.4\sqrt{(1.8 b_f\, t_f\, F_{yb} / F_{yc})} \tag{6-8}$$

$$t_{cf} < b_f / 6 \tag{6-9}$$

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} = minimum specified yield stress of the beam flange, ksi
- F_{yc} = minimum specified yield stress of the column flange, ksi

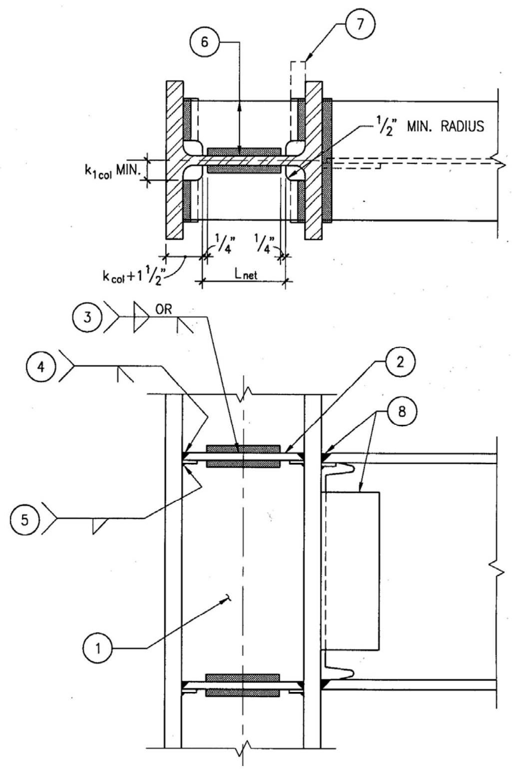
Where continuity plates are required, the thickness of the plates should be determined according to the following:

- For one-sided (exterior) connections, continuity plate thickness should be at least one-half of the thickness of the beam flanges.
- For two-sided (interior) connections, the continuity plates should be equal in thickness to the thicker of the two beam flanges entering the connection 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 complete joint penetration (CJP) welds as shown in Figure 6-9. Continuity plates should be welded to the web, as required, to transmit the shear forces corresponding to development of the axial strength — of the CJP weld at one end of the connection, for one-sided connections, and that at both ends, for two-sided connections.

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 AISC Seismic Provisions (AISC, 1997), which was published after FEMA-267, relaxed this criteria and states that continuity plates should be provided to match those in connections tested to obtain qualification.

Research conducted by this project tends to confirm that where the flange thickness of columns is sufficiently thick, continuity plates may not be necessary. Equation 6-8 was the formula used by AISC to evaluate column flange continuity plate requirements prior to the 1994 Northridge earthquake. It appears that this *formula is adequate to control excessive column flange prying provided that 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 6-9 was selected by judgment based on these tests.*



Notes

1. Web doubler plate where required by Section 6.4.3.2. See the *AISC Seismic Provisions* Section 9.3c, Commentary C9.3, and Figures C-9.2 and C-9.3 for options and connection requirements. QC/QA Category *BL/L* requirement for all welds.
2. Continuity plate as required per 6.4.3.1.
3. Required total weld strength = $0.6t_{pl}\left(L_{net}\right)F_{y\,pl}$. QC/QA Category *BL/L*.
4. CJP typical. QC/QA Category *BM/T*.
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 (250 μ-in), not flush. Do not gouge column flange.
8. Beam connection, see individual prequalifications.

Figure 6-9 Typical Continuity and Doubler Plates

6.4.3.2 Panel Zone Strength

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

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

$$t = \frac{C_y M_c \frac{h - d_b}{h}}{(0.9)\ 0.55 F_{yc} R_{yc} d_c \left(d_b - t_{fb}\right)} \tag{6-10}$$

where:

h is the average story height of the column, measured from the midpoint of the column above the beam to the midpoint of the column below the beam.

R_{yc} is 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 6.3.7 and Section 6.3.8 of these *Recommended Criteria*, respectively, and other terms are as defined in the *AISC-LRFD Specifications.*

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 this design criteria varies 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.*

6.4.3.3 Connections to Column Minor Axis

Connections to the minor axis of a column should be qualified by testing following the procedures of Section 6.9. If minor-axis connections are to be used in conjunction with major-axis connections to 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, and 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.

6.4.3.4 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 the described area, a calculation should be made to ensure 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, when made in an area of high stress, can lead to stress concentrations and potential fracture.

6.4.4 Bolted Joint Requirements

6.4.4.1 Existing Conditions

When evaluating existing structures, the condition of bolted connections should be determined based on the AISC and Research Council on Structural Connections (RCSC) specifications appropriate to the design and construction years, and on the following criteria:

- Representative samples of bolts should be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples should be removed and tested to determine tensile strength in accordance with *ASTM F606* and the bolt classified accordingly. Alternatively, bolts may be assumed to be A307.
- Any evidence of yielding in the connection plates indicates that the high-strength bolts are effectively in the snug-tight condition regardless of their original installation condition. If bolts have been identified as ASTM A325 and are not in a snug-tight condition they should be re-tightened or replaced. If bolts have been identified as ASTM A490 and are not in a snug-tight condition, they should be replaced. Re-tightening or installation of bolts should be to a pretensioned condition in accordance with the 1997 AISC or 1996 RCSC criteria.

6.4.4.2 Connection Upgrades

When upgrading existing connections, the capacity of bolted elements of the connection shall be determined based on the AISC and RCSC specifications appropriate to the design and construction years, and the following criteria:

- Bolts intended to transfer load in the shear/bearing mode should be installed according to the slip critical criteria.
- Bolts intended to transfer load by tension should be pre-tensioned.
- Bolts intended for use in proprietary connections, such as a viscous damping system, should be installed using the instructions applicable to the test data for the system.
- Bolted joints should not be upgraded by sharing loads with weld reinforcement. Any welded reinforcement shall be designed to transfer all the load, independent of the bolt capacity.

6.5 Prequalified Connection Details – General

Prequalified connection and connection upgrade 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 each prequalification description. Project-specific testing should be performed to demonstrate the adequacy of connection and upgrade 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 6.9 for use of nonprequalified connection and upgrade details.

Commentary: The following criteria were applied to connection and upgrade details 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, the 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 connection and upgrade details in the following sections are only prequalified for use in Ordinary Moment Frames (OMFs), while others are prequalified for both 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. Very little extrapolation has been applied in the prequalification limitations for SMFs, while some judgement has been applied to permit extrapolation for OMFs, based on the significantly lower rotational demands applicable to those systems.

6.5.1 Load Combinations and Resistance Factors

Design procedures for prequalified connection upgrades contained in Section 6.6 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.

6.6 Prequalified Connection Upgrades

This section provides prequalification data for various alternative types of welded steel moment-frame (WSMF) connection upgrade details. Table 6-6 lists the various alternative connection upgrade details that have been prequalified, together with the structural system (SMF or OMF) for which they are prequalified for use in Simplified Upgrade, and reference to the section of these *Recommended Criteria* where detailed information may be found. Refer to these individual reference sections for specific limits on the applicability of the prequalification, for specific performance data for use with Systematic Upgrade and for specific design procedures and details.

Table 6-6 Prequalified Welded Fully Restrained Connection Upgrade Details

Connection Type		Criteria Section	Structural System
Improved welded unreinforced flange	IWURF	6.6.1	OMF
Welded bottom haunch	WBH	6.6.2	OMF, SMF
Welded top and bottom haunch	WTBH	6.6.3	OMF, SMF
Welded cover plated flange	WCPF	6.6.4	OMF, SMF

Commentary: FEMA-355D – State of the Art Report on Connection Performance, provides extensive information on the testing and performance of these connections that is not repeated in this document. The data presented in FEMA-355D have been used in support of development of the prequalification performance data, design procedures, and limitations on design parameters for these connections presented herein.

6.6.1 Improved Welded Unreinforced Flange (IWURF) Connection

This section provides recommended criteria for design of connection upgrades intended to improve existing unreinforced, welded flange connections by improving the existing welded joints in the connection. This connection upgrade is prequalified only for Ordinary Moment Frame applications. Upgrade is accomplished through replacement of existing complete joint penetration groove welds of low-notch-toughness material and potentially having significant root defects, with new welds conforming to current construction requirements for welded steel moment-frame construction as shown in Figures 6-10 and 6-11. In addition, other elements of the connection, including panel zones and column flanges are reinforced, as required, to conform to the general recommendations of Section 6.4. Table 6-7 tabulates the limits of applicability of this prequalified connection upgrade and associated performance qualification data.

Commentary: This connection upgrades the typical pre-Northridge "prescriptive connection" commonly in use prior to the 1994 Northridge earthquake. After significant study, it has been concluded that with several improvements this connection can be made to perform reliably in frames designed as Ordinary Moment Frames as long as beam sizes are limited as indicated in Table 6-7.

The improvements required for this connection include the following:

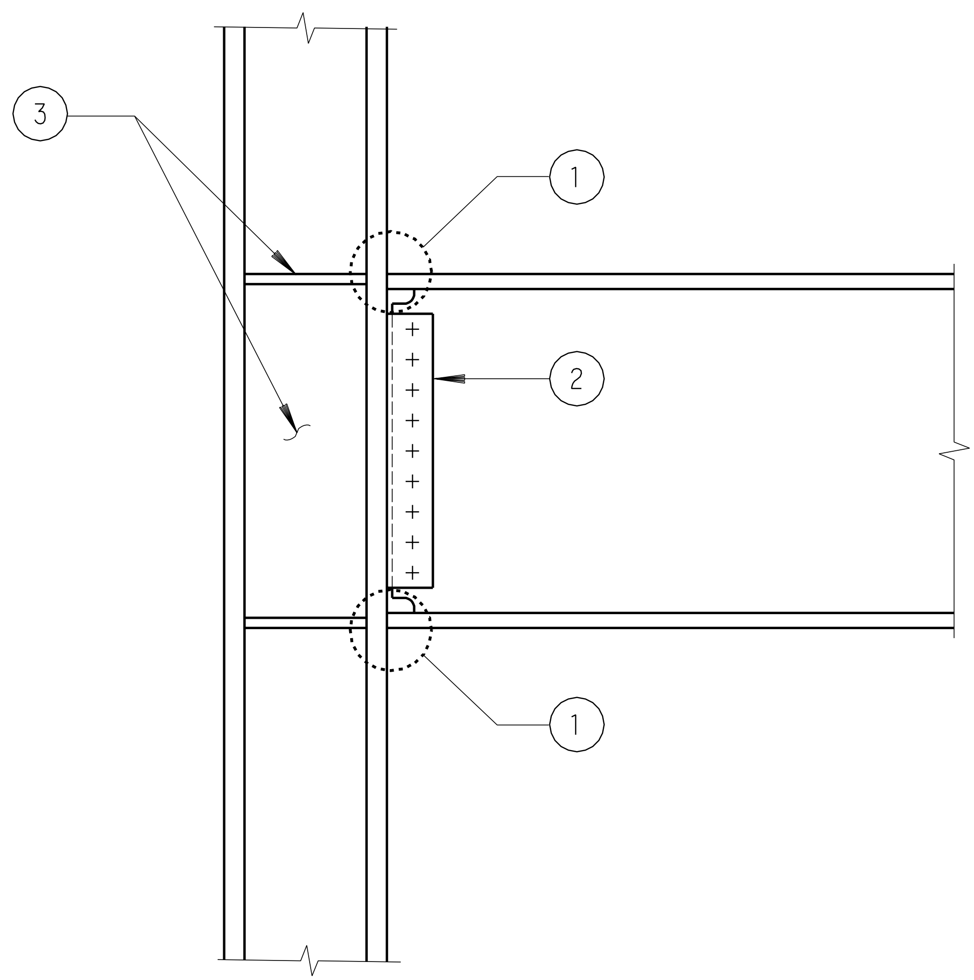
1. *Removal of existing low-toughness weld metal and replacement with weld metal with appropriate toughness;*

2. *Removal of bottom flange weld backing, back-gouging and addition of a reinforcing weld;*

3. Removal of weld tabs;

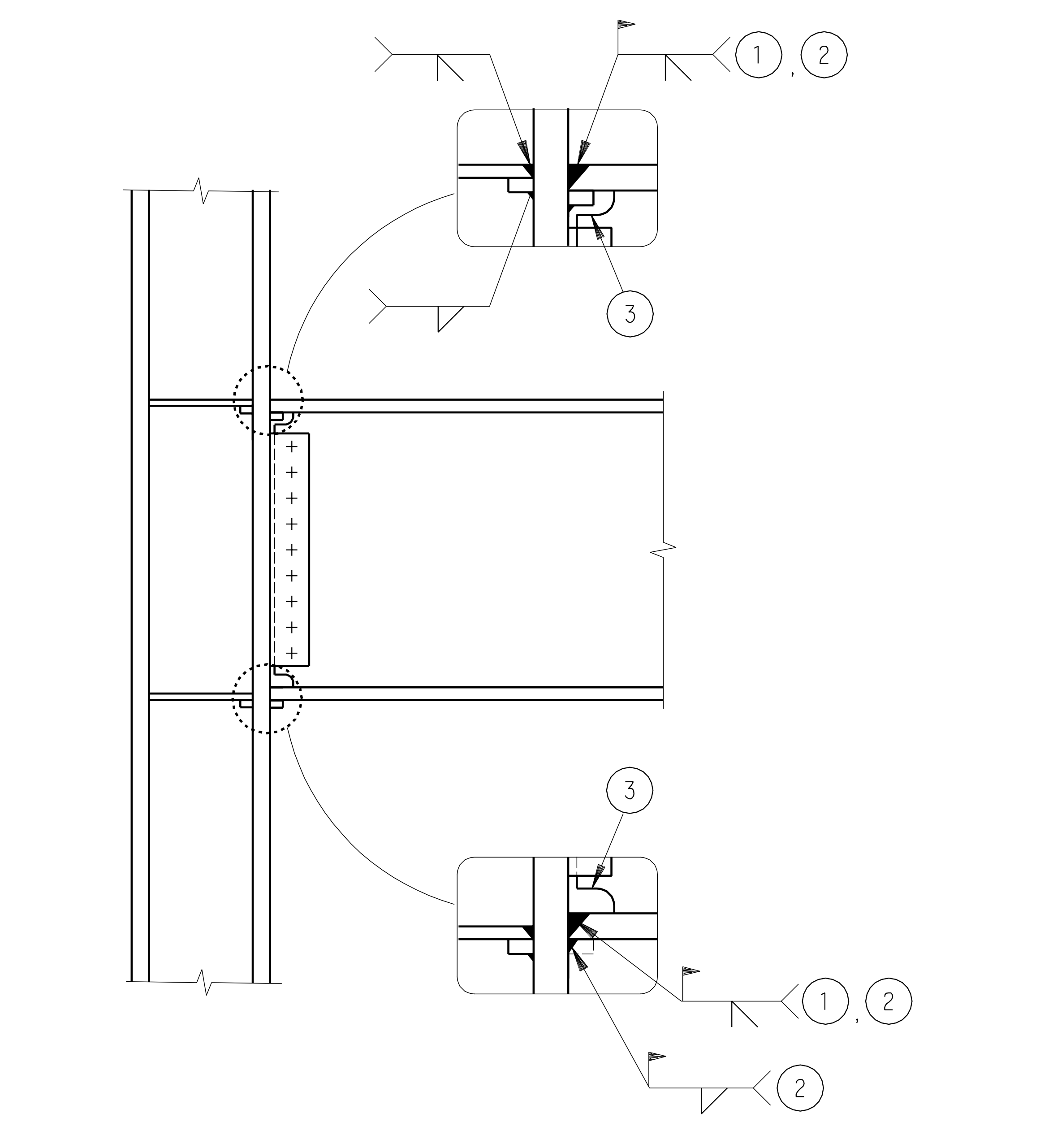
4. Improvements to weld quality control and quality assurance requirements and methods.

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



Notes:
1. See Figure 6-11 for welding requirements at these locations.
2. Existing bolted shear tab.
3. Existing or added continuity plates and web doubler plate. See Figure 6-9.

Figure 6-10 Improved Welded Unreinforced Flange Connection



Notes:

1. Gouge out existing weld at both the top and bottom flange and prepare joints for new weld.
2. Complete joint penetration groove weld at top and bottom flanges. At top flange, either (A), remove weld backing, backgouge, and add 5/16" minimum fillet weld, or (B), 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 is QC/QA Category AH/T.
3. Existing weld access hole to remain unmodified.

Figure 6-11 Welding Requirements at Improved Welded Unreinforced Flange Connection

Table 6-7 Prequalification Data for Improved Welded Unreinforced Flange Connections

Applicability Limits	
General:	
Applicable systems	OMF
Hinge location distance s_h	$d_c / 2 + d_b / 2$
Critical Beam Parameters:	
Depth	W36 and shallower
Minimum span-to-depth ratio	7
Flange thickness	1” maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Critical Column Parameters:	
Depth	Not limited
Permissible material specifications	A7, A36, A572 Gr. 50
Beam/Column Relations:	
Panel zone strength	Section 6.4.3.2, $C_{pr} = 1.1$
Column/beam bending strength	No requirement (OMF)
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	Section 6.4.3.1
Flange welds	Figures 6-10 and 6-11
Weld electrodes	Sections 6.4.2.4 and 6.4.2.5
Weld access holes	Existing weld access hole
Performance Data:	
Strength degradation rotation - θ_{SD}, radians	$0.031 - 0.0003d_b$
Immediate Occupancy rotation - θ_{IO}, radians	0.015, but not greater than θ_{SD}
Resistance factor, Immediate Occupancy, ϕ	0.9
Collapse Prevention drift angle - θ_U, radians	$0.060 - 0.0006d_b$
Resistance factor, Collapse Prevention, ϕ	0.9

Notes: d_b= beam depth, inches; d_c = column depth, inches.

6.6.1.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Detail the connection as shown in Figure 6-10 and 6-11.

Commentary: There is more research information available on unreinforced beam-to-column connections than there is on any other type of steel moment-frame connection. Not only were these connections extensively studied prior to the 1994 Northridge earthquake, they have been even more extensively studied in the aftermath. Many of the studies focused on the connection as used in pre-1994 practice, with bolted web connection, and flange welds with unrated or low notch toughness and with backing left in place, while other studies have been focused on improvements to the connection, including those improvements recommended in this section.

These tests give widely scattered results, but in general, indicate that when weld metal with sufficient notch toughness is used and workmanship is maintained at an appropriate level, these connections can reliably perform adequately for service in Ordinary Moment Frame, if not Special Moment Frame systems. Additional information may be found in FEMA-355D, State of the Art Report on Connection Performance.

6.6.2 Welded Bottom Haunch (WBH) Connection

This connection upgrade is accomplished by converting the existing welded unreinforced (WUF) connection into a haunched connection, with a single haunch present at the bottom beam flange. This connection upgrade is prequalified for both OMF and SMF applications. If the weld of the top beam flange to the column is made with weld metal with low or unclassified notch toughness, then, in addition to welding the new haunch at the bottom beam flange, this top beam flange weld must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4 to obtain SMF service. The general requirements of Section 6.4 should be complied with. Figure 6-12 provides a typical detail for this connection. Table 6-8 presents performance qualification data for the connection. Refer to *AISC Steel Design Guide Series 12* (Gross et al., 1999) for supplemental information to the design procedure given in Section 6.6.2.1.

6.6.2.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

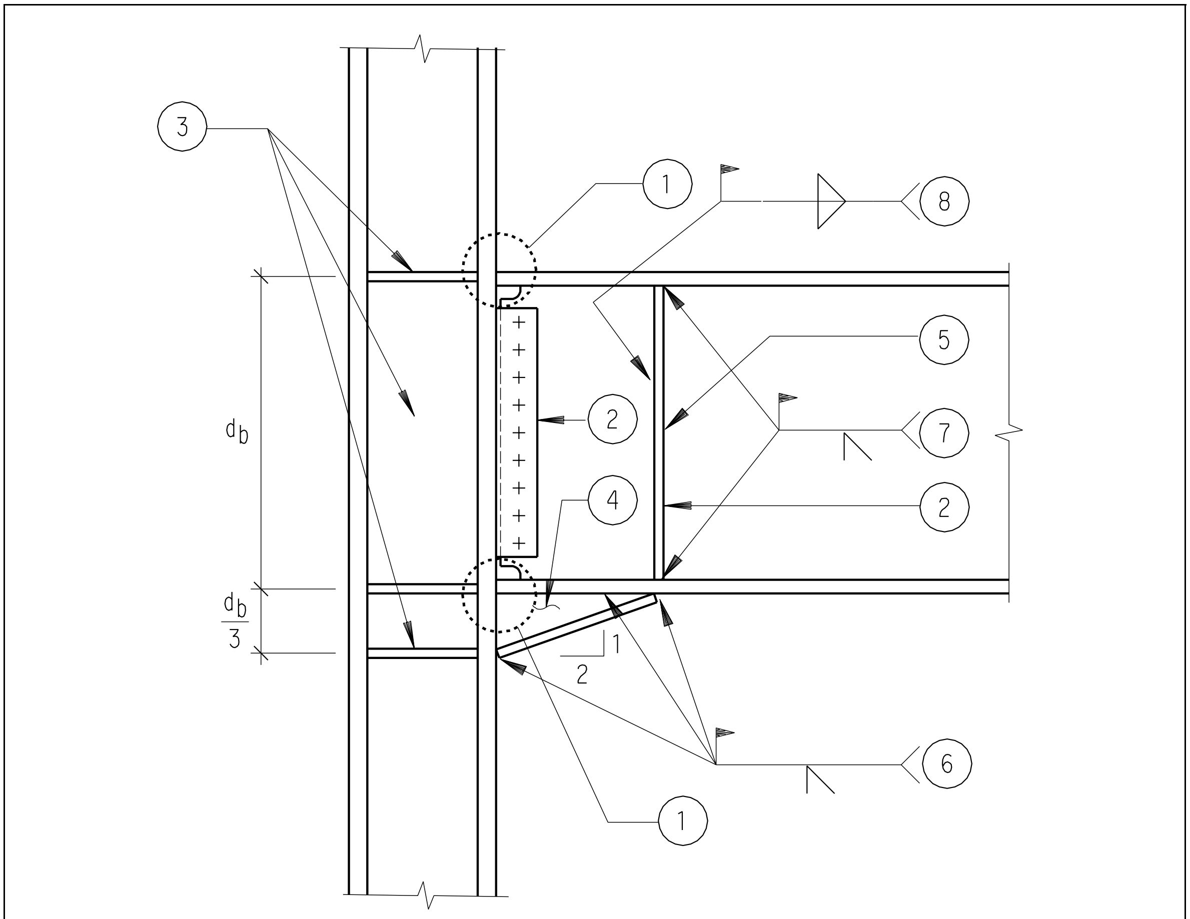
Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the haunch according to the criteria outlined in AISC Steel Design Guide Series 12.

Step 7: Detail the connection as shown in Figure 6-12.



Notes

1. For OMF connection, existing weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-9.
4. WT haunch.
5. New ½”-minimum stiffener plates each side.
6. Haunch welds, see Sections 6.4.2.3 and 6.4.2.4, QC/QA category AH/T.
7. Stiffener CJP welds; see Sections 6.4.2.3 and 6.4.2.4, QC/QA Category BM/T.
8. Stiffener fillet welds, 5/16” minimum. QC/QA Category CL/L.

Figure 6-12 Welded Bottom Haunch (WBH) Connection

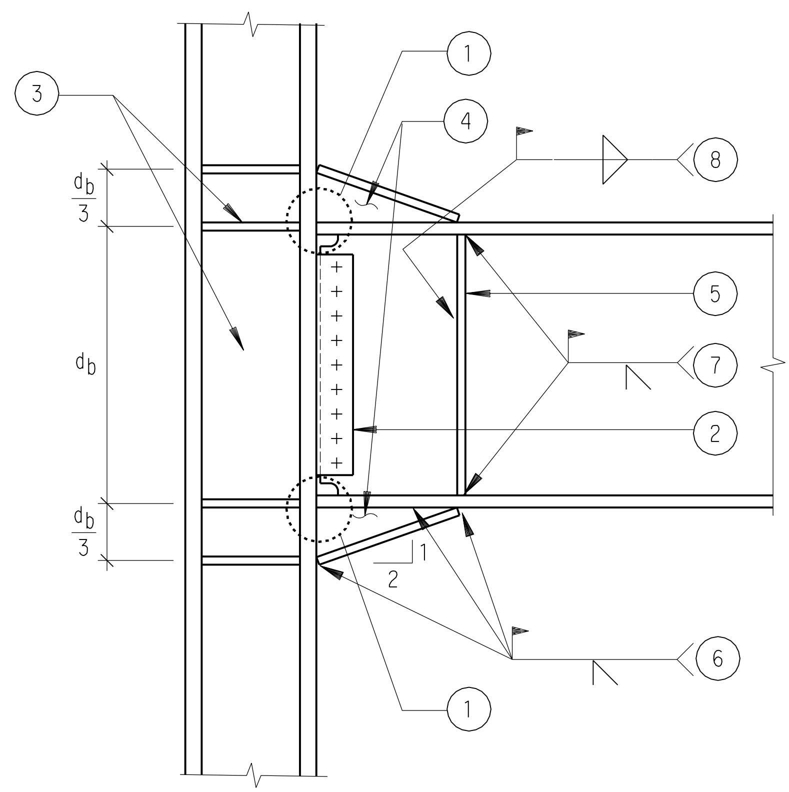
Table 6-8 Prequalification Data for Welded Bottom Haunch (WBH) Connection

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + l_h$ from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2” maximum SMF: 1” maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	At beam flanges: Section 6.4.3.1 At haunch: match haunch width and thickness
Haunch welds	Sections 6.4.2.3 and 6.4.2.4
Details of Haunch Design:	
Haunch size and strength criteria	Haunch to be sized by criteria as outlined in *AISC Steel Design Guide Series 12* (Gross et al., 1999)
Performance Data:	
Strength degradation rotation - θ_{SD}, radians	0.038
Immediate Occupancy rotation - θ_{IO}, radians	0.020
Resistance factor, Immediate Occupancy, ϕ	0.9
Collapse Prevention drift angle - θ_U – radians	0.06
Resistance factor, Collapse Prevention, ϕ	0.9

Note: d_c = column depth

6.6.3 Welded Top and Bottom Haunch (WTBH) Connection

This connection upgrade is accomplished by attaching a new welded haunch to both the top and bottom flanges of the existing beam connection. This connection upgrade is prequalified for both OMF and SMF applications. Existing welds in the connection need not be gouged out, nor replaced, for OMF applications. For SMF applications, in addition to installing the new haunches, if the beam flange welds to the column are made with weld metal of unclassified or low notch toughness, these welds must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4. Design is accomplished to accommodate the general requirements of Section 6.4. Figure 6-13 shows a typical detail for this connection. Table 6-9 provides performance qualification data.



Notes
1. For OMF connection, weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-9.
4. WT haunches.
5. New ½"-minimum stiffener plate each side.
6. Haunch welds, see Sections 6.4.2.3 and 6.4.2.4, QC/QA category AH/T.
7. Stiffener CJP welds; see Sections 6.4.2.3 and 6.4.2.4, QC/QA Category BM/T.
8. Stiffener fillet welds, 5/16” minimum. QC/QA Category CL/L.

Figure 6-13 Welded Top and Bottom Haunch (WTBH) Connection

Table 6-9 Prequalification Data for Welded Top and Bottom Haunch (WTBH) Connections

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + l_h$ from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2" maximum SMF: 1" maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Web connection	Existing bolted shear tab
Continuity plate thickness	At beam flanges: Section 6.4.3.1 At haunch: match haunch width and thickness
Haunch welds	Section 6.4.2.3 and 6.4.2.4
Details of Haunch Design:	
Haunch size and strength criteria	Haunch to be sized by criteria as outlined in *AISC Steel Design Guide Series 12* (Gross et al., 1999)
Performance Data:	
Strength degradation rotation - $\theta_{SD,}$ radians	0.038
Immediate Occupancy rotation - θ_{IO}, radians	0.02
Resistance factor, Immediate Occupancy, ϕ	0.9
Collapse Prevention drift angle - θ_U – radians	0.058
Resistance factor, Collapse Prevention, ϕ	0.9

Note: d_c = depth of column, inches

6.6.3.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the haunches according to the criteria outlined in *AISC Steel Design Guide Series 12* (Gross, et al., 1999).

Step 7: Detail the connection as shown in Figure 6-13.

6.6.4 Welded Cover Plated Flange (WCPF) Connection

This connection upgrade is accomplished by attaching new cover plates to both the top and bottom flanges of the existing beam. This connection upgrade is prequalified for both OMF and SMF applications. Existing welds in the connection need not be gouged out, nor replaced, for OMF applications. In addition to welding the new cover plates, if the beam flange welds to the column are made with welds having notch toughness that is either not classified or low, this weld must be gouged out and replaced with weld metal conforming to the recommendations of Sections 6.4.2.3 and 6.4.2.4 to obtain SMF service. Design is accomplished to accommodate the general requirements of Section 6.4. Figure 6-14 shows a typical detail for this connection. Table 6-10 provides prequalification limitations.

6.6.4.1 Design Procedure

Step 1: Calculate M_{pr}, at hinge location, s_h, according to methods of Section 6.3.5.

Step 2: Calculate V_p, at hinge location, s_h, according to methods of Section 6.3.6.

Step 3: Calculate M_c, M_f, and C_y as described in Section 6.3.7 and 6.3.8.

Step 4: Calculate the required panel zone thickness using the procedures of Section 6.4.3.2.

Step 5: Check requirements for Continuity Plates according to Section 6.4.3.1.

Step 6: Size the cover plates. When cover plates are to be field welded, the top cover plate should be narrower than the beam flange and the bottom cover plate should be wider. The area of the cover plates should be sized to satisfy the following relationship:

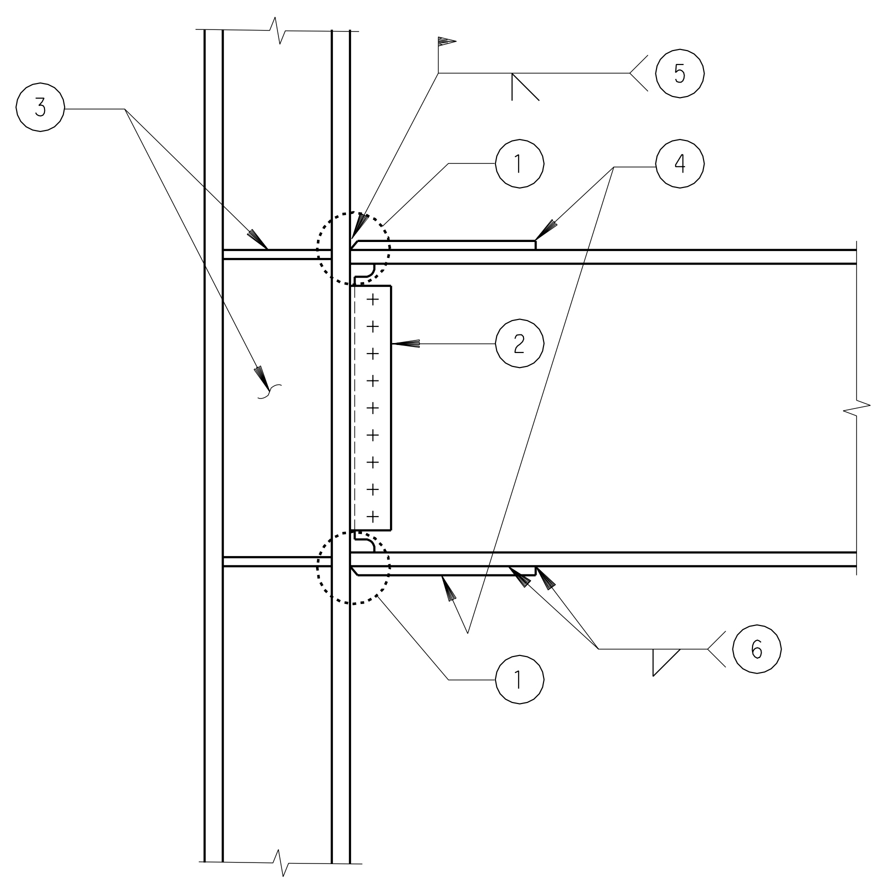
$$\left(kZ_b + A_{cp}\left(d_b + t_{cp}\right)\right)F_y \geq M_f \tag{6-11}$$

where:

k = 0.4 for OMF and 1.0 for SMF connections
A_{cp} = cross-section area of the cover plate, square inches
d_b = depth of the beam, inches
t_{cp} = thickness of the cover plate, inches

The remainder of the terms are as defined in Section 6.3 and 6.4.

Step 7: Detail the connection as shown in Figure 6-14.



Notes:
1. For OMF connection, weld can remain. For SMF connection, see Figure 6-11.
2. Existing bolted shear tab.
3. Existing continuity plates and web doubler plate. See Figure 6-8.
4. Cover plates.
5. Cover plate CJP welds, see Section 6.4.2.3 and 6.4.2.4, QC/QA Category AH/T.
6. Cover plate fillet welds, QC/QA Category BH/L.

Figure 6-14 Welded Cover Plated Flange (WCPF) Connection

Table 6-10 Prequalification Data for Welded Cover Plated Flange Connections

Applicability Limits	
General:	
Applicable systems	OMF, SMF
Hinge location distance s_h	$d_c/2 + l_{cp}$ from center of column
Critical Beam Parameters:	
Depth range	Up to W36
Minimum span-to-depth ratio	OMF: 5 SMF: 7
Flange thickness	OMF: 1-1/2: maximum SMF: 1" maximum
Permissible material specifications	A7, A36, A572 Gr. 50
Beam flange welds	OMF: Existing welds can remain. SMF: Sections 6.4.2.3 and 6.4.2.4.
Critical Column Parameters:	
Depth	OMF: Not limited SMF: W12, W14
Permissible material specifications	A7, A36, A572 Gr. 50
Beam / Column Relations:	
Panel zone strength	OMF: Section 6.4.3.2, C_{pr} = 1.1 SMF: Section 6.4.3.2
Column/beam bending strength ratio	OMF: No requirement SMF: Section 6.4.1.1
Connection Details:	
Relative size and proportions of cover plate	Section 6.6.4.1, Step 6.
Web connection	Existing bolted shear tab.
Continuity plate thickness	Section 6.4.3.1
Cover plate welds	Section 6.4.2.3 and 6.4.2.4
Performance Data:	
Strength degradation rotation - θ_{SD}, radians	$0.066 - 0.0011d_b$
Immediate Occupancy rotation - θ_{IO}, radians	0.02, but not greater than θ_{SD}
Resistance factor, Immediate Occupancy, ϕ	0.9
Collapse Prevention drift angle - θ_U, radians	$0.066 - 0.0011d_b$
Resistance factor, Collapse Prevention, ϕ	0.9

Notes: d_b= beam depth, inches, d_c= column depth

6.7 New Moment Frames and Moment-Resisting Connections

In some cases, it may be desirable to upgrade an existing steel moment-frame building by introducing new steel moment frames. This can be accomplished either with the addition of new framing, or the modification of existing framing not originally intended to participate in lateral resistance. New moment-resisting connections, introduced for such purpose, should be designed in accordance with the design procedures presented in *FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings,* and constructed in accordance with *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.* Table 6-11 presents performance data for connections that have been prequalified for use in new construction. The table may be used in assessing the effectiveness of new or modified framing employing these connections to achieve desired performance goals.

Commentary: Upgrade of existing WSMF buildings with the addition of new steel moment frames, or the modification of existing gravity frames to provide lateral resistance, will typically not be an effective upgrade strategy. This is because steel moment frames are inherently flexible and it is unlikely that the addition of new frames, by themselves, will be sufficient to control building drifts to levels that will protect existing WSMF connections from damage.

6.8 Proprietary Connections

This section presents information on several types of fully restrained connection technologies that have been developed on a proprietary basis. These connection technologies 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.

Table 6-11 Performance Data for Prequalified Moment-Resisting Connections for New Framing

	Strength Degradation[1]	Immediate Occupancy		Collapse Prevention[1]	
Connection Type	θ_{SD}	θ_{IO}[2]	ϕ	θ_U	ϕ
Welded Unreinforced Flange, Bolted Web (WUF-B)	$0.031-0.0003d_b$	0.020	0.9	$0.060-0.0006d_b$	0.9
Welded Unreinforced Flange, Welded Web (WUF-W)	0.051	0.020	0.9	0.064	0.9
Free Flange (FF)	$0.077-0.0012d_b$	0.020	0.9	$0.104-0.0016d_b$	0.9
Reduced Beam Section (RBS)	$0.060-0.0003d_b$	0.020	0.9	$0.080-0.0003d_b$	0.9
Welded Flange Plate (WFP)	0.04	0.020	0.9	0.07	0.9
Bolted Unstiffened End Plate (BUEP)	$0.071-0.0013d_b$	0.020	0.9	$0.081-0.0013d_b$	0.9
Bolted Stiffened End Plate (BSEP)	$0.071-0.0013d_b$	0.020	0.9	$0.081-0.0013d_b$	0.9
Bolted Flange Plate (BFP)	$0.12-0.0023d_b$	0.020	0.9	$0.10-0.0011d_b$	0.9
Double Split Tee (DS)	$0.12-0.0032d_b$	0.020	0.9	$0.14-0.0032d_b$	0.9

Notes:

Values in this table apply only to connections and framing that comply in all respects with the prequalification limits indicated in *FEMA-350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* and *FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications.*

1. For connections that are prequalified in *FEMA-350* for either SMF or OMF service, the values indicated apply for framing and connections that comply with the applicability limits for SMF service. When framing and connections comply with the applicability limits for OMF service but not for SMF service, ½ the tabulated values shall be used.
2. The value of θ_{IO} shall not be taken greater than the value for θ_{SD}.

6.8.1 Side Plate (SP) Connection

The proprietary Side Plate connection system is a patented technology shown schematically in Figure 6-15 for its application to upgrade of existing construction. Physical separation between the face of the column flange and the end of the 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 the flange widths of the beam and the column.

This connection system uses all fillet-welded fabrication. All fillet welds are made in either the flat or horizontal position using column tree construction. For new 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.

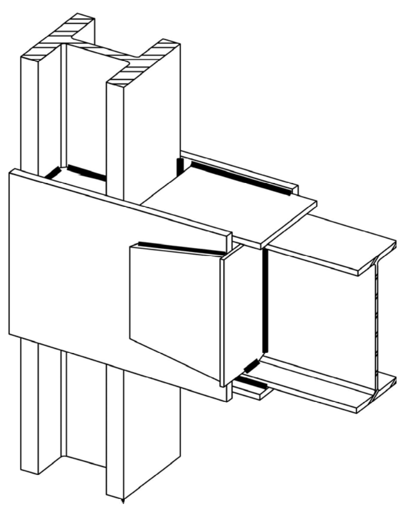


Figure 6-15 Proprietary Side Plate Connection – Application to Existing Construction

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 included 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. These 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 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.

The Side Plate connection for upgrade construction differs from its configuration for new construction by featuring an initial opening in each side plate to permit welding access, saving the cut-out pieces of plate for use as closure plates to close the access window after welding is completed. All new welds are fillet welds loaded principally in shear along their length. The existing Complete Joint Penetration (CJP) welds joining the beam flanges to the column flange are removed by airarcing to eliminate reliance on through-thickness properties and triaxial stress concentrations. The existing shear tab of the steel moment-frame beam(s) is left in place to provide gravity support. Existing continuity plates may be left in place to act as horizontal shear plates as depicted in Figure 6-15.

6.8.2 Slotted Web (SW) Connection

This proprietary connection (Seismic Structural Design Associates, Inc. US Patent No. 5,680,738 issued 28 October 1997) is shown schematically in Figure 6-16. 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.

The slotted web (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 Design Provisions for Steel Buildings*.

Seismic Structural Design Associates (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.

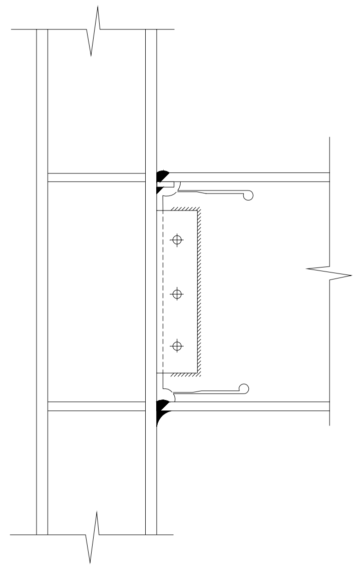


Figure 6-16 Proprietary Slotted Web Connection

Both analytical studies and ATC-24 protocol tests have demonstrated that the Seismic Structural Design Associates (SSDA) 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 retrofit of existing seismic moment frames. Specific qualification and design information for the Slotted Web connection may be obtained from the licensor.

6.8.3 Bolted Bracket (BB) Connection

This connection type is shown schematically in Figure 6-17. 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, but generic prequalification data have not been developed for this connection. 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 may be obtained from the licensor.

6.9 Project-Specific Testing of Nonprequalified Connections

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

Commentary: While it is not the intent of these Recommended Criteria to require testing for most situations, there will arise circumstances where proposed connections do not satisfy prequalification requirements. In these situations, the *requirement for testing reflects the view that the behavior of connections under severe cyclic loading cannot be reliably predicted by analytical means alone.*

This suggests that for nonprequalified connections, both laboratory testing and the development of an analytical procedure that predicts the behavior are required. Requiring an analytical procedure, based on testing, develops a design methodology applicable to the design of connections employing slightly different members than actually tested.

Testing is costly and time consuming, and it is the intent of these Recommended Criteria to minimize testing requirements to the extent possible. Test conditions should match the conditions in the structure as closely as possible.

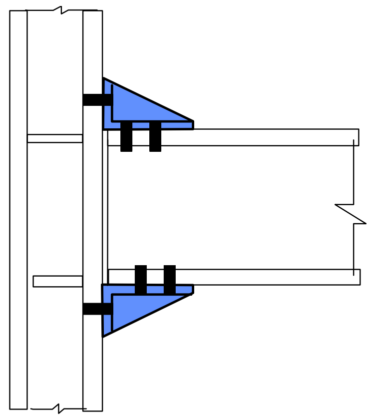


Figure 6-17 Bolted Bracket Connection

6.9.1 Testing Procedure

The testing program should follow the requirements of Appendix S of the 1997 *AISC Seismic Provisions* 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 interstory drift angle capacity for the performance states described in Table 6-12. The drift angle capacity θ shall be defined as indicated in Figure 6-18. Acceptance criteria should be as indicated in Section 6.9.2.

Table 6-12 Interstory Drift Angle Limits for Various Performance Levels

Performance Level	Symbol	Drift Angle Capacity
Peak Strength	θ_{IO}	Taken as that value of θ in Figure 6-18 at which peak load resistance occurs.
Strength degradation	θ_{SD}	Taken as that value of θ in Figure 6-18 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	θ_U	Taken as that value of θ in Figure 6-18 at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

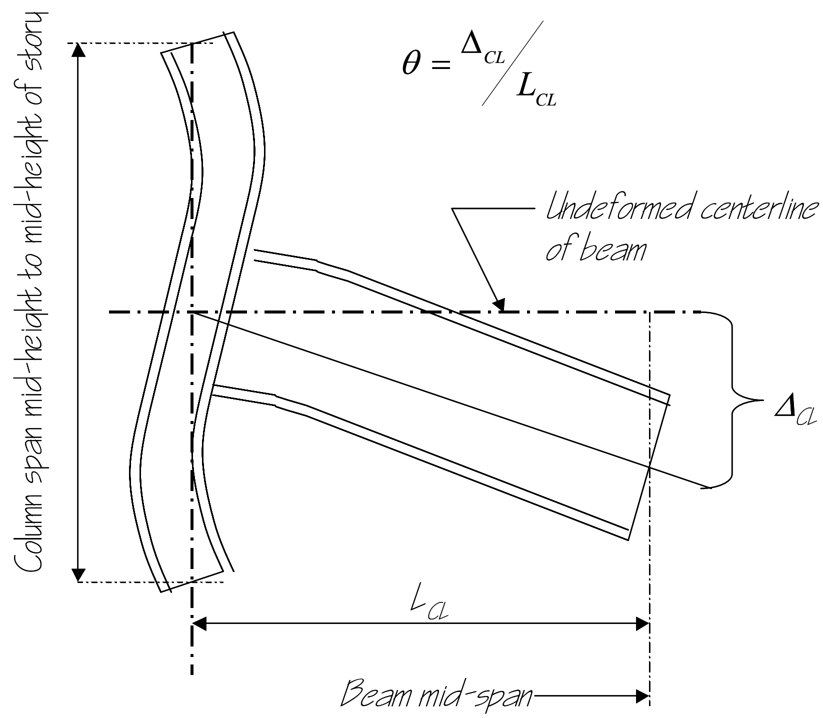


Figure 6-18 Drift Angle

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. Once the beam is chosen, the test column shall be selected to represent properly the inelastic action anticipated of the column in the real structure, given the chosen beam. 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 θ_j, a predetermined value of the drift angle. The loading history, shown in Table 6-13, is defined by the following parameters:

  θ_j = the peak deformation in load step *j*

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

Table 6-13 Numerical values of θ_j and n_j

Load Step #	Peak deformation θ_j	Number of cycles, n_j
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 falls to 20% of that obtained at θ_{IO} 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 FEMA-302, 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 pre-qualified. 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 permitted weight and size limits contained in Section S5.2 of Appendix S have been eliminated.

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 the appropriate adjustments are made when using the AISC loading history with these Recommended Criteria. In general, total drift angle is approximately equal to plastic rotation 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 measurements indicated in Figure 6-18.

The calculation of θ illustrated in Figure 6-18 assumes that the top and the bottom of the test 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 θ from flexure of the column.

6.9.2 Acceptance Criteria

For Simplified Upgrade, the median value of the drift angle capacity at strength degradation, θ_{SD}, and at connection failure, θ_U, obtained from qualification testing shall not be less than indicated in Table 6-14. 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 6-14.

Table 6-14 Minimum Qualifying Total Interstory Drift Angle Capacities, θ_{SD} and θ_U, for OMF and SMF Systems

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

Where the clear-span-to-depth ratio of beams in the moment-resisting frame is less than 8, the qualifying total drift angle capacities indicated in Table 6-14 shall be increased to θ'_{SD} and θ'_U, given by Equations 6-12 and 6-13:

$$\theta'_{SD} = \frac{8d}{L}\left(1+\frac{L-L'}{L}\right)\theta_{SD} \qquad (6\text{-}12)$$

$$\theta'_{U} = \frac{8d}{L}\left(1+\frac{L-L'}{L}\right)\theta_{U} \qquad (6\text{-}13)$$

where:
- θ'_{SD} = Qualifying strength degradation drift angle capacity for spans with $L / d < 8$
- θ_{SD} = the basic qualifying strength degradation drift angle capacity, in accordance with Table 6-14
- θ'_U = the qualifying ultimate drift angle capacity, for spans with $L / d < 8$
- θ_U = the basic qualifying ultimate drift angle capacity, in accordance with Table 6-14
- L = the center-to-center spacing of columns, per Figure 6-4, inches.
- L' = the distance between points of plastic hinging in the beam, inches.
- d = depth of beam in inches

For Systematic Upgrade, the median drift angle capacity for Immediate Occupancy performance level shall be taken as the median value of the drift angle, θ_{IO}, at which the peak connection strength occurs, in accordance with Table 6-12. The median drift angle capacity for the Collapse Prevention performance level shall be taken as the median value of the drift angle,

θ_U, in accordance with Table 6-12. Resistance factors, ϕ, shall be determined in accordance with the procedures of Appendix A of these Recommended Criteria. For any connection, the value of ϕ need not be taken as less than 0.75 for the Immediate Occupancy Level or less than 0.5 for the Collapse Prevention Level.

Commentary: This section sets criteria for use in project-specific qualification of connection and connection upgrade details, in accordance with Section 6.9 and for development of new connection and connection upgrade prequalifications in accordance with Section 6.10 of these Recommended Criteria. Two interstory drift angle capacities are addressed. The values indicated in Table 6-14 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. They indicate 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 may be suitable for upgrades to performance criteria other than those that form the basis for FEMA-302. This suitability requires demonstration using the performance evaluation procedures contained in Chapter 3 and Appendix A of these Recommended Criteria.

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 Equations 6-12 and 6-13, unless the frames are designed to experience proportionally lower drifts than permitted by FEMA-302.

6.9.3 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 demands, 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 the 1997 *AISC Seismic Provisions*.

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

6.10 Prequalification Testing Criteria

This section provides criteria for development of new prequalifications for connection and connection upgrade details for which there is no current prequalification or to extend the parametric limitations for prequalification listed in Section 6.5, for general application. Prequalification includes a program of connection assembly prototype testing supplemented by a suitable analytical procedure that permits prediction of behavior identified in the testing program.

Commentary: The purpose of this section is to provide recommended procedures for prequalification of a connection or connection upgrade detail that is not currently prequalified in these Recommended Criteria or to extend the range of member sizes that may be used with currently pre-qualified 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 have wide application. Prequalification of a connection should incorporate both the testing described in this section and due consideration of the following four criteria:

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

2. *Rational models should be developed and validated for predicting the resistance associated with each mechanism and failure mode.*

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

4. *Given the models and procedures, there should be an adequate data base of experiments to permit assessment of the statistical reliability of the connection.*

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 as required by Section 6.9.1 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 deformations 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.

6.10.1 Prequalification Testing

Testing and acceptance criteria should follow the recommendations in Section 6.9 except that at least five nonidentical 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.

6.10.2 Extending the Limits on Prequalified Connections

Once a connection has been prequalified, with its parameters lying within certain ranges, extending this limitation for general use requires further testing. Testing and acceptance criteria should follow the recommendations in Section 6.9 except that at least two nonidentical test specimens shall be tested. The resulting range of member size that will be prequalified should be limited to those contained in the database of tests for the connection type.

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 3. 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 3 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 3 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 3 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 3.

A.2 Performance Evaluation Approach

A.2.1 Performance Objectives and Confidence

As defined in Section 3.2 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 3. 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 a year's time of the probability that damage will exceed that permitted within a performance level. Mathematically, this may be expressed as:

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

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 3 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 3, 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 3, 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 3, 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, λ, 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, λ, is determined from the factored-demand-to-capacity equation:

$$\lambda = \frac{\gamma \gamma_a D}{\phi C} \tag{A-2}$$

where:

C = median estimate of the capacity of the structure. This estimate may be obtained either by reference to default values contained in Chapters 3 and 6, 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,

γ = 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,

γ_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,

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

λ = 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-Δ 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 the 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 3. 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 3 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 Chapters 3 and 6 of these *Recommended Criteria* may be estimated using the default values of Chapters 3 and 6, 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 3.

5. **A factored-demand-to-capacity ratio, λ 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 λ. 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 3 are used to determine either demand or median capacity estimates, than the corresponding values of the demand factors γ and resistance factors ϕ should also be determined in accordance with the procedures of Chapter 3. 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 should be the lowest value determined using the values of λ determined in accordance with step 5 above, back-calculated from the equation:

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

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,

β_{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, λ, and β_{UT}.

The values of the parameter β_{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 β_{UT} can be calculated using the equation:

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

where: β_{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 β_{ui} values associated with demand estimation, beam-column connection assembly behavior, and building global stability capacity prediction, respectively.

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

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 3 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.

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 U.S. 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:

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

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 γ and analysis uncertainty factor γ_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 β_{D_R}.

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

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

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 we use likely values for each of these effects in the model, and 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:

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

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 3 are summarized in Table A-3. The bias factors C_B used in Chapter 3 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 β_{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 β_{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 3.6, 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, ϕ, for various prequalified connection types are presented in Chapters 3 and 6. These values were determined from cyclic tests of full-size connection assemblies using the testing protocols indicated in Section 6.9. 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 θ_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
θ_{IO}	Immediate Occupancy	The lowest drift angle at which any of behaviors 1, 2, or 3, occur (see Section A.5, above)
θ_U	Ultimate	The drift angle at which behavior 4 occurs
θ_{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* 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, 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 Type 1 or Type 2 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 θ_{IO} and θ_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 ϕ, 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 θ_{IO} or θ_U values available from the laboratory data. That is, take the standard deviation of the natural logarithms of the θ_{IO} or θ_U values respectively, obtained from each laboratory test. Logarithmic standard deviation may be determined from the formula:

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

where:

β = 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 ϕ_R due to randomness, the observed variation in connection behavior, from laboratory testing, using the equation:

$$\phi_R = e^{-\frac{k}{2b}\beta^2} \tag{A-11}$$

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
β = 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:

$$\phi = \phi_R \phi_U = \phi_R e^{-\frac{k}{2b}(0.2)^2} \tag{A-12}$$

where:

ϕ_R = the resistance factor accounting for random behavior
ϕ_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 β_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 ϕ 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 Δ_1.

4. Increase the scaling amplitude of the accelerogram slightly and repeat Step 3. Plot this point as Δ_2. Draw a straight line between points Δ_1 and Δ_2.

5. Repeat Step 4 until the straight line slope between consecutive points Δ_i and Δ_{i+1}, is less than 0.2 S_e. When this condition is reached, Δ_{i+1} is the global drift capacity for this accelerogram. If $\Delta_{i+1} \geq 0.10$ 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 β 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 ϕ_R due to randomness in the predicted global collapse capacity for various ground motions from the equation:

$$\phi_R = \mathrm{e}^{-\frac{k}{2b}\beta^2} \tag{A-13}$$

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

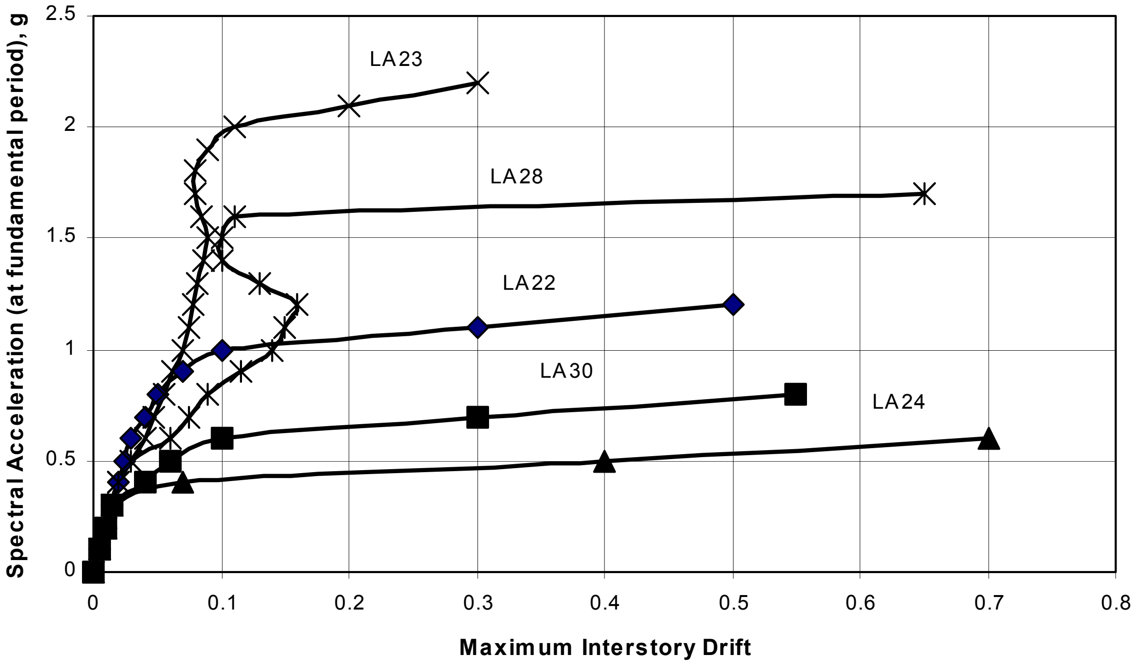


Figure A-1 Representative Incremental Dynamic Analysis Plots

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

$$\phi = \phi_U \phi_R = \mathrm{e}^{-\frac{k}{2b}\beta_U^{\,2}} \phi_R \tag{A-14}$$

where:

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

β_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-Δ instabilities to develop. Prediction of the onset of P-Δ instability due to ground shaking is quite complex. It is possible that during an acceleration record a structure will displace to a point where static P-Δ instability would initiate, only to have the structure straighten out again before collapse can occur, due to a reversal in ground shaking direction.*

The basic effect of P-Δ instability is that a negative tangent stiffness is induced in the structure. That is, P-Δ 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. These effects can be approximately accounted for by increasing the amount of dead load on the structure, to produce artificially the appropriate negative stiffness.

B. Detailed Procedures for Loss Estimation

B.1 Introduction

This appendix describes detailed loss estimation procedures for developing structural damage functions and related direct economic loss functions for welded, steel moment-frame (WSMF) buildings. These procedures are compatible with the *HAZUS* (NIBS, 1997a) methodology, a complex collection of modules that work together to estimate casualties, loss of function and economic impacts on a region due to a scenario earthquake. The *HAZUS* methodology was developed for the Federal Emergency Management Agency (FEMA) by the National Institute of Building Sciences (NIBS) and is documented in a three-volume Technical Manual (NIBS, 1997b). One of the main components of the methodology estimates the probability of various states of structural and nonstructural damage to buildings. Other modules of the methodology use the damage state probabilities to estimate various types of building-related losses. The *HAZUS* methodology is intended primarily for use in estimation of earthquake losses in regions with a large inventory of buildings represented by generic building types.

The procedures presented in this appendix utilize the results of WSMF building performance evaluations conducted in accordance with Chapter 3 of these *Recommended Criteria*, supplemented by default values of parameters provided in this appendix, to construct structural damage and loss functions. Specifically, structural analysis using the nonlinear static method must be performed as a precursor to the application of the loss estimation methods presented herein. Default values of damage and loss parameters are provided for typical 3-story, 9-story and 20-story WSMF buildings. Example loss estimates that illustrate application of the detailed methods are developed for typical 9-story WSMF buildings.

Commentary: To support mitigation efforts, FEMA funded NIBS to develop "Procedures for Development of HAZUS-Compatible Building-Specific Damage and Loss Functions" (Kircher, 1999). These procedures are an extension of the more general methods of HAZUS, but allow users to incorporate building-specific data including capacity and fragility values developed by nonlinear static (pushover) analysis of the building of interest. The purpose of such evaluations is to understand better the response behavior of the structure, the modes of structural damage and failure, and the amount of structural damage (e.g., connection damage) as a function of the level of earthquake ground shaking. These so-called "building-specific" methods provide the primary basis for the detailed loss-estimation procedures of this appendix.

Implementation of the detailed procedures requires users to have certain levels of expertise and knowledge. It is anticipated that users will be structural engineers:

1. *familiar with evaluation of the earthquake behavior of buildings,*
2. *experienced with nonlinear building analysis,*

3. *familiar with basic methods of statistical analysis, and*
4. *familiar with the HAZUS methodology and building-specific procedures.*

In addition to the HAZUS Technical Manual (NIBS, 1997b), further references on the HAZUS methodology may be found in papers contained in a 1997 special issue of Earthquake Spectra on loss estimation published by Earthquake Engineering Research Institute (EERI). Pertinent papers include Whitman et al. (1997), and Kircher et al. (1997a,b).

B.2 Scope

B.2.1 General

The scope of the detailed loss-estimation procedures is limited to steel moment-frame (WSMF) building damage caused by ground shaking. While ground shaking typically dominates earthquake loss, other hazards, such as ground failure, due to either liquefaction or land-sliding, and surface fault rupture, can also cause building damage. Although less prevalent, when building damage due to ground failure or surface fault rupture occurs it is typically more severe than building damage caused by ground shaking.

The scope of detailed loss-estimation procedures is further limited to damage to the structural system of WSMF buildings. While structural (connection-related) damage is the primary focus of this report, significant damage and loss can occur to nonstructural components and to building contents. Typically, at lower states of damage, nonstructural and contents losses are greater, by several times, than structural losses. This is due to the fact that damage usually begins to occur in nonstructural systems and can become severe before any damage occurs to the structural system. At higher states of damage, the structure becomes more important to economic loss estimation since damage to the structure can affect a complete loss of both structural and nonstructural systems (and contents), and cause long-term closure of the building (that is, loss of function).

The scope of detailed loss-estimation procedures is still further limited to direct economic losses associated with repair and replacement of damaged structural elements and to building loss of function.

Commentary: Other types of losses, such as casualties, may also be important to the user. In those cases for which users require loss estimates for hazards other than ground shaking, the HAZUS Technical Manual (NIBS, 1997b) should be used to develop appropriate loss models. In those cases for which users require loss estimates for building damage other than structural and loss types other than economic, Kircher (1999) should be used to augment the detailed procedures of this section.

B.2.2 Typical Welded, Steel Moment-Frame (WSMF) Buildings

Detailed loss-estimation methods permit the development of building-specific loss functions, based on the configuration and structural details of a specific building. In order to allow more general application, this appendix also presents a series of default loss functions, derived using these methods for use in prediction of damage to WSMF buildings of different height, different seismic force design and different connection type, without needing to resort to detailed structural analyses of individual buildings. Default values of various damage and loss parameters are provided for typical 3-story, 9-story and 20-story buildings. Default values are provided for buildings located in different regions (having different design codes and practice) and having different connection conditions, as identified in Table B-1.

Table B-1 Connections in Typical WSMF Buildings in Three Regions

Connection Condition	Los Angeles Region	Seattle Region	Boston Region
Pre-Northridge	X	X	X
Post-Northridge Special Moment Frame (SMF)	X	X	
Damaged Pre-Northridge	X		

A pre-Northridge connection condition assumes that the building has beam-column connections typical of buildings designed and built prior to the 1994 Northridge earthquake, but which have not been damaged by earthquake ground shaking. A post-Northridge connection condition assumes that the building has either new or retrofitted beam-column connections that comply with the recommendations of *FEMA-350 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, as applied to Special Moment-Resisting Frame Systems. A damaged pre-Northridge connection condition assumes the building has beam-column connections that are typical of pre-Northridge buildings and that have sustained substantial earthquake damage, but have not been repaired.

B.3 Damage States

Structural damage is described by one of four discrete damage states: Slight, Moderate, Extensive and Complete. Of course, actual building damage varies as a continuous function of earthquake demand. Ranges of damage are used to describe damage, since it is not practical to have a continuous scale, and damage states provide users with an understanding of the structure's physical condition. Descriptions of structural damage states for WSMF buildings (*HAZUS* model building type S1), based upon but modified from the *HAZUS Technical Manual* are indicated in Table B-2.

Table B-2 Descriptions of Structural Damage States

Damage State	Buildings with Pre-Northridge Connections	Buildings with Post-Northridge Connections
Slight structural damage	No permanent interstory drift. Minor deformations in some connection elements and fractures in less than 10% of the connections at any floor level.	No permanent interstory drift. Minor deformations in some connection elements. No fractures in connections.
Moderate structural damage	Permanent interstory drift as large as 0.5%. Perhaps as many as 25% of the connections on any floor level have experienced fracture.	Permanent interstory drift as large as 0.5%. Moderate amounts of yielding and distortion of some column panel zones. Minor buckling of some girders.
Extensive structural damage	Many connections have failed with a number of fractures extending into and across column panel zones. Some connections may have lost ability to support gravity load, resulting in partial local collapse. Large permanent interstory drifts occur in some stories.	Many steel members have exceeded their yield capacity, resulting in significant permanent lateral deformation of the structure. Some structural members or connections may have major permanent member rotations at connections, buckled flanges and failed connections. Some connections may have lost ability to support gravity load, resulting in partial local collapse.
Complete structural damage	A significant portion of the structural elements have exceeded their ultimate capacities and/or many critical structural elements or connections have failed resulting in dangerous permanent lateral displacement, partial collapse or collapse of the building. Approximately 15% (of the total square footage) of all WSMF buildings with complete damage are expected to have collapsed.	

General guidance to users regarding selection of damage parameters, taken from Kircher (1999), is provided in Table B-3. Additional steel moment-frame (WSMF) building-specific guidance is given in Table B-4 for determining the structural damage state based on the fraction of damaged connections.

Table B-3 General Guidance for Expected Loss Ratio and Building Condition in Each Damage State

Damage State	Likely Amount of Damage, Loss, or Building Condition			
	Range of Possible Loss Ratios	Probability of Long-Term Building Closure	Probability of Partial or Full Collapse	Immediate Postearthquake Inspection
Slight	0% - 5%	P = 0	P = 0	Green Tag
Moderate	5% - 25%	P = 0	P = 0	Green Tag
Extensive	25% - 100%	$P \cong 0.5$	$P \cong 0$[1]	Yellow Tag
Complete	100%	$P \cong 1.0$	P > 0	Red Tag

1. Extensive damage may include some localized collapse of the structure.

Table B-4 Specific Guidance for Selection of Damage State Based on Connection Damage

Fraction of All Connections Likely to be Damaged[1]		Damage State
Average Fraction	**Fraction Range**	
0.02	0.0 – 0.05	Slight
0.10	0.05 – 0.25	Moderate
0.50	0.25 – 0.75	Extensive
≅1.0	0.75 – 1.0	Complete

1. Connections having indications of flaws at the root of the Complete Joint Penetration (CJP) weld of beam flanges to columns are not considered as having damage.

B.4 Basic Approach

For the detailed procedures, maximum interstory drift is the basic parameter used to assess structural (i.e., connection) damage. Based on the calculated maximum interstory drift demand, the probability that a structure will be damaged sufficiently to be classified as conforming to each of the four damage states described in Section B.3, is determined. For example, at a maximum interstory drift demand of 3%, a structure may be found to have a low probability, only 10%, of having only slight damage, a 30% probability of moderate damage, a 40% probability of extensive damage and a 20% probability of complete damage. This probabilistic approach is taken in recognition of the fact that due to inherent uncertainties in the prediction of ground motion, structural response and structural damage, it is not possible to quantify precisely how much damage a structure will have for a given earthquake. In this methodology, the probabilistic relationship between structural damage and maximum interstory drift is termed a fragility function. Fragility functions are defined by median estimates of the maximum interstory drift at which a damage state will initiate in a structure (damage state medians) and a parameter β that represents the uncertainty associated with these estimates.

Maximum interstory drift is defined as the peak drift (throughout the duration of earthquake shaking) occurring in any story in the building. Maximum interstory drift is assumed to be about the same as the drift angle demand on nearby beam-column connections. On this basis, damage states of buildings with pre-Northridge connection conditions are related (and calibrated) to observed building response and damage. Similarly, users can define damage states (fragility medians) of buildings with post-Northridge connection conditions using the results of laboratory testing of connections.

In general, the maximum interstory drift in a structure will be greater than the average drift calculated over the height of the building due to various building characteristics (e.g., modes of vibration, nonlinearity, etc.) and the specific nature of the earthquake ground shaking. While response history analyses (of complex multi-degree-of-freedom nonlinear models) provide the most accurate and complete set of building response data, such analyses are rarely practical for engineering applications and are not required for this methodology.

The detailed procedures rely on nonlinear static (pushover) analysis to estimate peak interstory drift and damage. Height-dependent factors are used to adjust pushover drift results for higher-mode effects and other effects not explicitly included in the nonlinear static analysis. Similarly, other height-dependent modal factors are used to relate maximum interstory drift to spectral displacement demand, so that damage (fragility) functions may be expressed in terms of spectral displacement but still be based on the drift angle limits of connections at the story (or stories) experiencing the maximum drift.

The overall approach or process used to estimate economic loss involves a number of steps, as illustrated in the flowchart of Figure B-1. Users are expected to select an appropriate scenario earthquake and to develop the 5%-damped response spectrum of this earthquake using, for example, the generalized spectrum shape and soil amplification factors described in *FEMA-273* or *FEMA-302*.

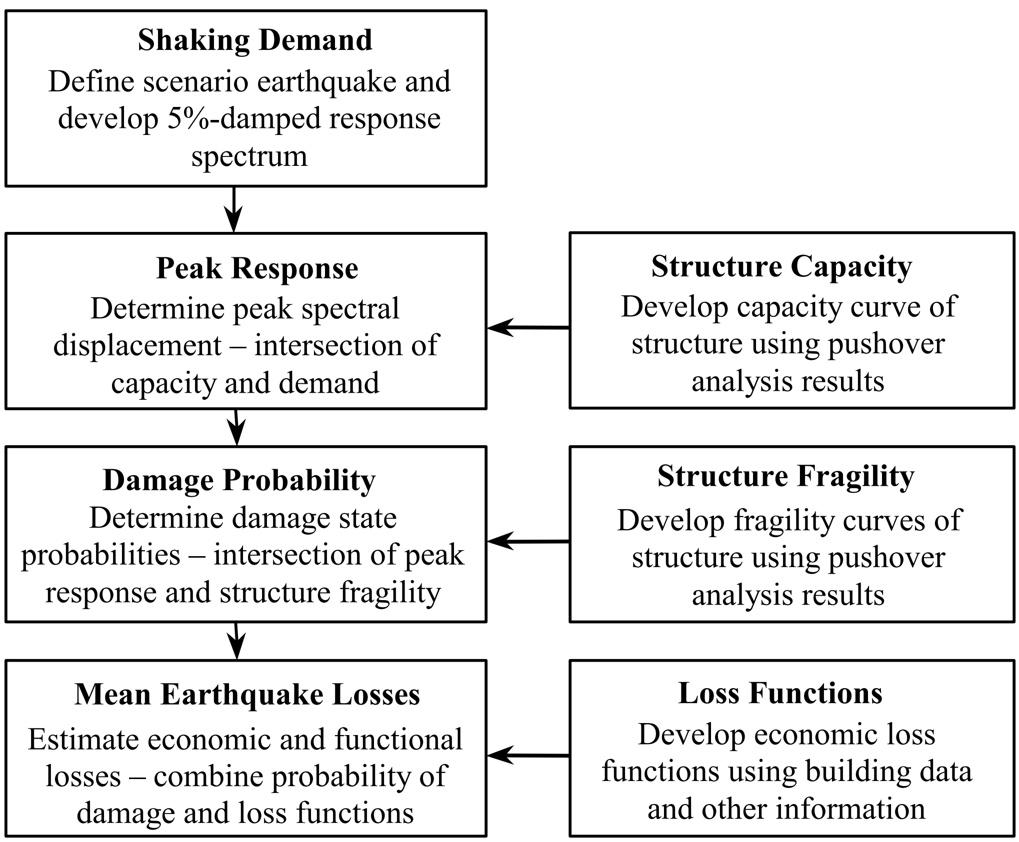


Figure B-1 Flowchart of Detailed Loss Estimation

Users are also expected to provide other information and data for the building. This can range from basic structural data obtained from the construction documents to results obtained from a nonlinear static analysis of the building, conducted in accordance with the guidelines of

Chapter 3. Section B.5 summarizes required input data to be supplied by the user. Subsequent sections provide guidance for developing structure capacity, structure response, structure fragility and building loss functions.

B.5 Required Data — User Input

The accuracy of loss estimates performed using the detailed methodology depends primarily on the extent and quality of the information provided by the user. While default data is provided and may be used if considered appropriate, the more effort the user puts into the determination of building data, the more reliable the results will be.

It is expected that the user will have seismic hazard data available. Although not required for development of damage and loss functions, seismic hazard data, including site soil conditions, are important and must be input by the user when developing loss estimates. It is also expected that, as a minimum, the user will have basic data on the building characteristics, such as the building size, occupancy (that is, use, rather than the number of occupants) and replacement cost.

Users are expected to calculate a pushover curve for the building at displacements up to complete failure of the structure. This may require pushing the building beyond the target displacement used in performance evaluation, as in Chapter 3, particularly if the evaluated performance objective was based on a low hazard level. The pattern of applied lateral loading should be based on the fundamental mode in the direction of interest and pushover results should represent both horizontal directions of building response (i.e., both principal axes of the building). If pushover results are significantly different for the two different directions, separate pushover curves should be developed and used to estimate losses for each direction. Three-dimensional models that permit rotation as well as translation should be used for pushover analysis of structures with plan irregularities that affect torsion.

Users are expected to have an understanding of the expected performance of the components of the structural system and the modes of failure as a function of building interstory drift. In addition to drift, Chapter 3 has identified other key performance parameters including column axial-load capacity and column tension-splice capacity that should be considered when determining at what drift level various failure modes and damage states are expected to occur.

Users are expected to provide the total replacement value of the structural system, expressed in terms of dollars/square foot. Although not required (default values are included in this appendix), users should also provide input on the repair of structural damage. That is, for each damage state, the user could review the associated damage to the structure and develop a cost and schedule for elements and components requiring repair. This may be done judgmentally, or more thoroughly by developing actual repair schemes, and obtaining estimates of, for example, construction costs, schedule, and building interruption.

B.5.1 Building Capacity Curve

The building capacity curve is derived from the pushover curve using modal properties for the building and a standard shape compatible with the *HAZUS* methodology. Specifically, the capacity curve is the pushover curve transformed from coordinates of base shear and roof displacement to coordinates of spectral acceleration (S_A) and spectral displacement (S_D). This coordinate transformation is accomplished on a point by point basis, by using the formulas:

$$S_{Di} = \alpha_2 \Delta_i \tag{B-1}$$

$$S_{Ai} = \frac{V_i / W}{\alpha_1} \tag{B-2}$$

where:
- α_1 = fraction of building weight effective in the fundamental mode in the direction under consideration (Equation B-3),
- α_2 = fraction of building height at the elevation where the fundamental-modal displacement is equal to spectral displacement (Equation B-4),
- Δ_i = displacement at point "i" on the pushover curve,
- V_i = base shear force at point "i" on the pushover curve (kips),
- W = building weight (kips),

and:

$$\alpha_1 = \frac{\left(\sum_{i=1}^{N} (w_i \phi_{ip}) / g \right)^2}{\left[\sum_{i=1}^{N} (w_i) / g \right] \left[\sum_{i=1}^{N} (w_i \phi^2_{ip}) / g \right]} \tag{B-3}$$

$$\alpha_2 = \frac{1}{PF_p \phi_{cp,p}} = \frac{\sum_{i=1}^{N} (w_i \phi^2_{ip}) / g}{\left[\sum_{i=1}^{N} (w_i \phi_{ip}) / g \right] \phi_{cp,p}} \tag{B-4}$$

where:
- w_i / g = mass assigned to the i^{th} degree of freedom,
- ϕ_{ip} = amplitude of modal shape at i^{th} degree of freedom,
- $\phi_{cp,p}$ = amplitude of mode shape at control point,
- N = number of degrees of freedom.

Some structural analysis software programs have the capability of automatically converting pushover curves to capacity curves using these formulas. As a simpler approximation to the formulas for α_1 and α_2 given above, these modal factors may be reasonably well estimated based only on the number of stories, N, using the following formula:

$$\frac{1}{\alpha_1} \cong \frac{1}{\alpha_2} \cong N^{0.14} \leq 1.5 \tag{B-5}$$

In the *HAZUS* methodology, two control points define a standard shape for the capacity curve. These are the yield capacity control point and the ultimate capacity control point, as shown in Figure B-2. The yield point (normally designated by D_y, A_y) defines the limit of the elastic domain and the ultimate point (normally designated by D_u, A_u) defines the point along the curve where the structure is assumed to be fully plastic.

The user is expected to define capacity curve control points from the actual capacity curve using both judgment and the following rules:

- Yield capacity control point (D_y, A_y) is selected as the point where significant yielding is just beginning to occur (slope of capacity curve is essentially constant up to the yield point).
- The expected period, T_e, of the building, at or just below yield, should be the true "elastic" fundamental-mode period of the building:

$$T_e \cong 0.32\sqrt{\frac{D_y}{A_y}} \tag{B-6}$$

- The ultimate capacity control-point acceleration, A_u, is selected as the point of maximum spectral acceleration (maximum building strength), not to exceed the value of spectral acceleration at which the structure has just reached its full plastic capacity.
- The ultimate capacity control-point displacement, D_u, is selected as the greater of either the spectral displacement at the point of maximum spectral acceleration or the spectral displacement corresponding to Equation B-7:

$$D_u = 2D_y \frac{A_u}{A_y} \tag{B-7}$$

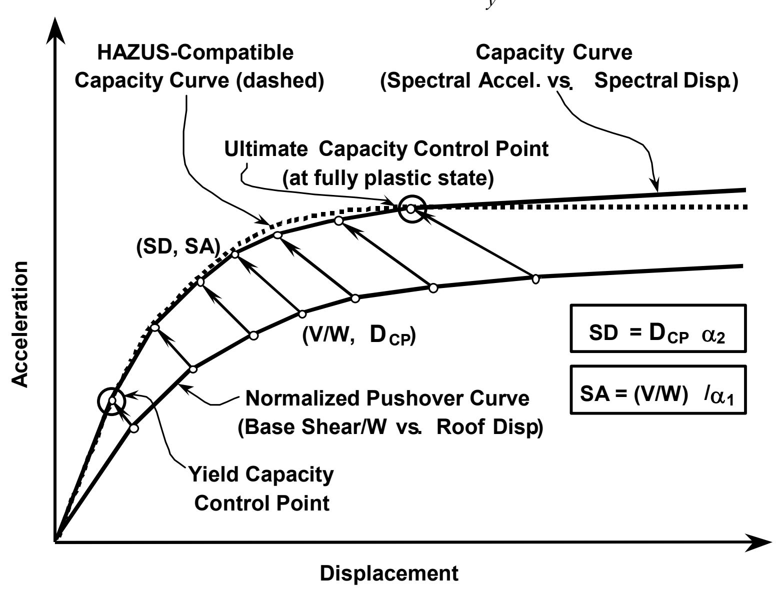


Figure B-2 Example Development of Standard (*HAZUS*-Compatible) Capacity Curve from a Normalized Pushover Curve

Commentary: The HAZUS definition of the elastic period T_e is the same as the initial period, and must not be confused with the definition of the effective period T_e contained in FEMA-273. The effective period T_e of FEMA-273 is based on *stiffness at 60% of the ultimate strength of the building and should not be used for loss estimation since it generally overestimates the displacement of the building.*

Table B-5 summarizes the elastic period and capacity curve control points for typical steel moment-frame buildings studied in this project. Capacity was derived from pushover analyses using modal properties based on Equation B-5. Building period and pushover properties were based on analyses reported in FEMA-355C and pertain to buildings conforming to the 1994 Uniform Building Code requirements. Individual buildings conforming to these same code provisions may be either stronger or weaker than those analyzed and buildings designed to other code requirements are likely to have substantially different characteristics than those indicated.

Table B-5 Capacity Curve Properties of Typical Welded Steel Moment-Frame Buildings

Capacity Parameter	Pre-Northridge Connections			Post-Northridge Connections		
	3-Story	9-Story	20-Story	3-Story	9-Story	20-Story
Buildings Located in Los Angeles						
Elastic Period (sec.)	1.01	2.24	3.74	1.02	2.21	3.65
Yield Point Disp. (in.)	2.6	8.0	11.7	2.7	7.7	11.1
Yield Point Accel. (g)	0.26	0.16	0.09	0.26	0.162	0.085
Ultimate Point Disp. (in.)	7.5	23	33	8.1	26	44
Ultimate Point Accel. (g)	0.37	0.23	0.12	0.40	0.27	0.167
Buildings Located in Seattle						
Elastic Period (sec.)	1.36	3.06	3.46	1.30	3.06	3.52
Yield Point Disp. (in.)	3.3	7.9	15.0	3.0	7.9	15.5
Yield Point Accel. (g)	0.18	0.09	0.13	0.18	0.086	0.128
Ultimate Point Disp. (in.)	9.3	22	43	12.0	25	48
Ultimate Point Accel. (g)	0.26	0.12	0.18	0.36	0.14	0.198
Buildings Located in Boston						
Elastic Period (sec.)	1.97	3.30	3.15	1.62	3.17	2.97
Yield Point Disp. (in.)	2.2	5.8	8.9	3.6	8.0	15.8
Yield Point Accel. (g)	0.058	0.054	0.091	0.140	0.082	0.183
Ultimate Point Disp. (in.)	7.1	20	33	10.2	29	47
Ultimate Point Accel. (g)	0.093	0.095	0.167	0.198	0.150	0.274

B.5.2 Structural Response

In the *HAZUS* methodology, structural response to ground motion is estimated based on elastic system properties modified using “effective” stiffness and damping properties of the structure to simulate inelastic response. Effective stiffness properties are based on secant stiffness at each displacement and effective damping is based on combined viscous and hysteretic measures of dissipated energy, assuming cyclic response of the structure to the given displacement. Effective damping greater than 5% of critical is then used to reduce spectral demand, in a manner similar to that followed in *ATC-40* (ATC, 1997).

Figure B-3 illustrates the process of developing an inelastic response (demand) spectrum from the 5%-damped elastic response (input) spectrum. The demand spectrum is based on elastic response divided by amplitude-dependent damping reduction factors (i.e., R_A at periods of constant acceleration and R_V at periods of constant velocity). In Figure B-3, the demand spectrum intersects the building's capacity curve at the point of peak building response (i.e., spectral displacement, *D*, and spectral acceleration, *A*). The amount of spectrum reduction typically increases for buildings that have reached yield and that dissipate hysteretic energy during cyclic response.

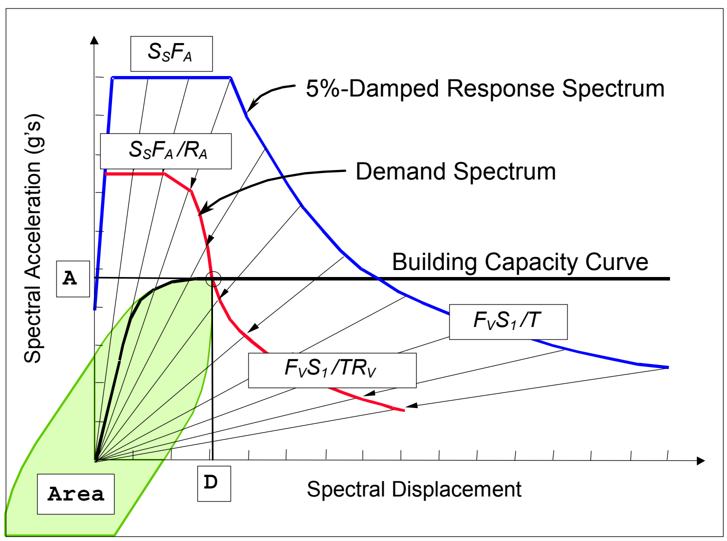


Figure B-3 Example Demand Spectrum Construction and Calculation of Peak Response Point (*D, A*)

Spectrum reduction factors are functions of the effective damping β_{eff} of the building as defined by Equations B-8 and B-9:

$$R_A = 2.12/\left(3.21 - 0.68\,ln(\beta_{eff})\right) \tag{B-8}$$

$$R_V = 1.65/\left(2.31 - 0.41\,ln(\beta_{eff})\right) \tag{B-9}$$

Effective damping β_{eff} is defined as the total energy dissipated by the building during peak earthquake response and is the sum of an elastic damping term β_E and a hysteretic damping term β_H associated with post-yield, inelastic response:

$$\beta_{eff} = \beta_E + \beta_H \tag{B-10}$$

The elastic damping term β_E is assumed to be constant (i.e., amplitude independent) and represents response at, or just below, the yield point. For most steel moment-frame (WSMF) buildings the value of the elastic damping term should be taken as 5% of critical, assuming nonstructural components (e.g., cladding) help dampen the structure. The value of the elastic damping term should be taken as 3% of critical for bare steel frames or WSMF buildings with limited nonstructural damping.

The hysteretic damping term β_H is dependent on the amplitude of post-yield response and is based on the area enclosed by the hysteresis loop at peak building displacement D and acceleration A as shown in Figure B-3. Hysteretic damping β_H is defined in Equation B-11:

$$\beta_H = \kappa \left(\frac{Area}{2\pi DA} \right) \tag{B-11}$$

where: *Area* is the area enclosed by the hysteresis loop, as defined by a symmetrical push-pull of the building capacity curve up to peak positive and negative displacements, ± D, assuming no degradation of components,
D is the peak displacement response of the capacity curve,
A is the peak acceleration response at the peak displacement, D
κ is a degradation factor that defines the fraction of the *Area* used to determine hysteretic damping.

The κ (kappa) factor in Equation B-11 reduces the amount of hysteretic damping as a function of anticipated structure performance (e.g., connection condition) and shaking duration, to simulate degradation (e.g., pinching) of the hysteresis loop during cyclic response. Shaking duration is described qualitatively as either short, moderate or long, and is assumed to be primarily a function of earthquake magnitude, although proximity to fault rupture can also influence the duration of the level of shaking that is most crucial to building damage. For example, ground shaking close to the zone of fault rupture can be strong, but typically contains only a few strong pulses. Values of the degradation factor for typical WSMF buildings are suggested in Table B-6.

Table B-6 Values of the Degradation Factor κ for Typical WSMF Buildings

Connection Condition	Peak Response Amplitude and Post-Yield Shaking Duration				
	At One-Half Yield	At or Below Yield	Post-Yield Shaking Duration		
			Short	Moderate	Long
Post-Northridge	1.0	1.0	1.0	0.9	0.7
Pre-Northridge	1.0	0.9	0.8	0.5	0.3
Damaged	1.0	0.7	0.6	0.3	0.1

As shown in Figure B-3, peak building displacement D is determined by the intersection of the capacity curve and the demand spectrum. The intersection requires either a graphical solution or a (spreadsheet) calculation that evaluates the area of the hysteresis loop as a function of amplitude. Alternatively, the target displacement of Section 3.4.5.3.1, divided by the modification factor α_2 calculated in accordance with Equation B-5 may used to estimate peak nonlinear spectral displacement of the building. In this case, the effective fundamental mode period, T_e, should be taken as equal to elastic fundamental-mode period T_i and the values of the coefficients C_1, R, C_2 and C_3 in Section 3.4.5 should be consistent with structural properties and the actual amount of nonlinear response corresponding to the target displacement.

B.5.3 Structure Fragility

Building fragility curves are lognormal functions that describe the probability of reaching, or exceeding, structural damage states, given deterministic (median) estimates of spectral displacement. These curves take into account the variability and uncertainty associated with structural response prediction, capacity curve properties, damage states and ground shaking. The fragility curves distribute damage among the Slight, Moderate, Extensive and Complete damage states. For any given value of spectral response, discrete damage-state probabilities are calculated as the difference of the cumulative probabilities of reaching, or exceeding, successive damage states. Discrete damage-state probabilities are used as inputs to the calculation of building-related losses.

Each fragility curve is defined by a median value of building response (i.e., spectral displacement) that corresponds to the threshold of that damage state and by the uncertainty associated with that damage state. The conditional probability of being in, or exceeding, a particular damage state *ds*, given the spectral displacement S_d (or other seismic demand parameter), is defined by Equation B-12:

$$\mathrm{P}\left[ds \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left(\frac{S_d}{\hat{S}_{d,ds}}\right)\right] \tag{B-12}$$

where: $\hat{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, *ds*,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage state *ds*, and

Φ is the standard normal cumulative distribution function.

Development of damage-state medians requires users to:

- select specific values of maximum interstory drift of the structure that best represent the threshold of each of the discrete damage states (consistent with the descriptions of damage states provided in Section B.3), and
- convert damage-state threshold values (e.g., maximum interstory drift) to spectral displacement coordinates (i.e., same coordinates as those of the capacity curve).

Default values of maximum interstory drift that may be used for typical steel moment-frame buildings are provided in Table B-7. These values of drift are consistent with observations of damage and loss that occurred in the 1994 Northridge earthquake (pre-Northridge connection conditions) and with the interstory drift criteria of Section 3.6 for post-Northridge connection conditions. The values of drift given in Table B-7 do not necessarily reflect thresholds of damage states of buildings with significant plan or height irregularity. Buildings with a significant irregularity would be expected to have substantially smaller values of drift defining the thresholds of damage states.

Table B-7 Maximum Interstory Drift Values Defining Damage-State Thresholds of Typical WSMF Buildings

Connection Condition, Building Height and Location	Structural Damage State			
	Slight	Moderate	Extensive	Complete
Pre-Northridge – All Heights/Locations	0.01	0.015	0.025	0.04
Post-Northridge – 3-Story – Los Angeles	0.01	0.02	0.040	0.100
Post-Northridge – 9-Story – Los Angeles	0.01	0.02	0.040	0.080
Post-Northridge – 20-Story – Los Angeles	0.01	0.02	0.040	0.060
Post-Northridge – 3-Story – Seattle	0.01	0.0175	0.030	0.080
Post-Northridge – 9-Story – Seattle	0.01	0.0175	0.030	0.060
Post-Northridge – 20-Story – Seattle	0.01	0.0175	0.030	0.050
Post-Northridge – All Heights – Boston	0.01	0.015	0.025	0.04

Conversion of maximum interstory drift to damage-state medians is based on the building height and other factors and the following equation:

$$\hat{S}_{d,ds} = \frac{\alpha_2 \Delta_{ds} H_R}{\alpha_3 \alpha_{4,ds}} \qquad \text{(B-13)}$$

where:
$\hat{S}_{d,ds}$ = median spectral displacement value of damage state, *ds* (inches)
Δ_{ds} = maximum interstory drift ratio at the threshold of damage state *ds*, determined by user (e.g., typical building values of Table B-8)
H_R = height of building at the roof level (inches)
α_2 = pushover modal factor from Equation B-4 or Equation B-5
α_3 = higher-mode factor (Equation B-14)
$\alpha_{4,ds}$ = mode-shape factor (Equation B-15)

The higher-mode factor, α_3, is the ratio of interstory drift due to all modes of vibration to the interstory drift of the fundamental (pushover) mode at the story with maximum fundamental-mode drift. The value of the higher mode factor may be determined by explicit calculation (e.g., ratio of peak drift values of response history and pushover analyses), or may be approximated based on the number of stories, *N*, and the following formula:

$$\alpha_3 \cong N^{0.14} \le 1.5 \qquad \text{(B-14)}$$

The mode-shape factor, $\alpha_{4,ds}$, is the ratio of maximum fundamental-mode (pushover-mode) interstory drift to the average pushover-mode interstory drift (i.e., average drift over all stories). Maximum pushover-mode interstory drift is the value of drift of those stories contributing to the damage state of interest. For tall buildings with Slight structural damage of a localized nature, maximum pushover-mode interstory drift is simply the drift of the story with the most displacement. As the extent of the damage increases (with damage state) or the building height decreases, or both, the difference between maximum pushover-mode interstory drift and average pushover-mode interstory drift decreases. The value of the mode-shape factor is 1.0 for Complete damage, since damage would typically be pervasive throughout the building. The value of the mode-shape factor may be determined directly from the shape of the pushover mode or may be approximated based on the number of stories, *N*, the following formula:

$$\alpha_{4,ds} \cong N^{0.10} \tag{B-15}$$

Limits of $\alpha_{4,S} \leq 1.5$ for Slight damage, $\alpha_{4,M} \leq 1.25$ for Moderate damage, $\alpha_{4,E} \leq 1.1$ for Extensive damage, and $\alpha_{4,C} \leq 1.0$ for Complete damage are suggested.

Lognormal standard deviation (β) values describe the total uncertainty inherent in the fragility-curve damage states. Three primary sources contribute to the total uncertainty of any given state, namely, the uncertainty β_C associated with the capacity curve, the uncertainty β_D associated with the demand spectrum, and the uncertainty $\beta_{T,ds}$ associated with the discrete threshold of each damage state. Since the demand spectrum is dependent on building capacity, a convolution process is required to combine their respective contributions to total uncertainty. To avoid this rather complex calculation, the *Procedures for Developing HAZUS-Compatible Building-Specific Damage and Loss Functions* (Kircher, 1999) provides pre-calculated values of total damage-state uncertainty for different values of capacity, demand and damage state variability. Users may refer to this document when developing values of damage-state uncertainty or use the β values given in Table B-8 for typical steel moment-frame (WSMF) buildings.

Table B-8 Structural Damage-State Variability (β) Factors of Typical WSMF Buildings

Building Location	Pre-Northridge Connections			Post-Northridge Connections		
	3-Story	9-Story	20-Story	3-Story	9-Story	20-Story
Los Angeles	0.90	0.85	0.80	0.70	0.65	0.60
Seattle	0.95	0.90	0.85	0.75	0.70	0.65
Boston	0.95	0.90	0.85			

Commentary: The structural damage state uncertainty factors β given in Table B-8 include a large, dominant contribution to the total variability from the variability associated with ground shaking demand. A large amount of ground shaking variability is appropriate when the fragility functions are to be used to estimate damage and loss for a scenario earthquake characterized by median predictions of ground shaking. Ground shaking uncertainty accounts for the inherent differences between actual and median predictions of ground shaking. The structural damage state uncertainty factors β given in Table B-8 would not be appropriate for estimating damage when ground shaking is actually known, or for estimating probabilistic losses that include ground shaking variability directly in the hazard function.

B.5.4 Loss Functions

Loss functions convert damage to loss by taking the sum over all four damage states of the products of the probability that a building will be damaged within a given damage state multiplied by the expected loss given that the damage state is experienced. In the case of economic loss, the expected losses can be normalized by dividing by the total replacement value to obtain an estimate of the mean loss ratio.

As discussed in Section B.4, users are expected to provide economic loss data in terms of the value of the building (structure), and the costs and associated construction time that would be required to repair Slight, Moderate and Extensive damage. These loss parameters would most appropriately be based on estimated costs of repair schemes developed to correct Slight, Moderate and Extensive damage, as predicted by a performance evaluation (pushover analysis) of the structure. Alternatively, default economic loss ratios are provided at the end of this section for typical steel moment-frame (WSMF) buildings.

Repair and replacement costs are the expected dollar costs (per square foot) that would be required to repair (or replace) damaged structural elements. In general, the cost of the structural system (and related repairs) will vary based on building occupancy (for example, hospital structures cost more per square foot then industrial buildings).

Commentary: Some consideration should be given to prevailing codes and ordinances that would govern the repair work. Do prevailing regulations require strengthening as well as repair?

Replacement value is the preferred measure of direct economic loss, although other measures could be used, such as loss of market value. Market value would, in general, produce entirely different loss estimates. For example, an older building of no special importance or historical significance is to be vacated and completely renovated, but instead an earthquake occurs and destroys the structure. Should economic loss be based on the replacement value (e.g., cost of a new building of comparable size and function), on the near zero value of the existing building, or on the market value of the building (which would also *include value of the land)? These types of question are crucial to the estimation of economic loss, but are beyond the scope of this section. For steel moment-frame (WSMF) buildings, economic loss functions used here are based on repair and replacement value of the structure, consistent with HAZUS methodology.*

Table B-9 provides mean structural repair costs (loss ratios and corresponding loss rates) for damage states of typical WSMF buildings. These rates are based on a number of assumptions. First, typical WSMF buildings are assumed to have a total replacement value of $125/sq. ft. and the structure is assumed to be worth 20% of total building value ($25/sq. ft.).

Inspection costs of 5% of the cost of the structural system are included in the loss ratios and loss rates for buildings with Pre-Northridge connection conditions. The 5% value is based on an assumed inspection cost of $1,500 per connection and the assumption that on average about one-half of the connections of these types of buildings would be inspected following an earthquake. The cost of repair of damaged connections is assumed to be $20,000 per connection. On the basis of this amount, the cost of repairing all connections would be about one and one-half times the cost of a new structural system.

The cost of repair of Slight damage to buildings with Post-Northridge connection conditions is assumed to be zero on the basis that, for example, minor distortion of flanges, or other incidental structural damage would not require repair. The cost of repair of Moderate and Extensive structural damage of typical WSMF buildings is assumed to be 10% and 50% of the value of the structural system. However, the actual repair cost of a specific building could be very different due, for example, to the building's configuration, and the repair's interference with nonstructural systems and finishes.

Table B-9 Mean Structural Loss Ratios and Rates of Typical WSMF Buildings

Building Connection Condition	**Structural Damage State**			
	Slight	**Moderate**	**Extensive**	**Complete**
Mean Structural Loss Ratio (Repair Cost / Replacement Cost)				
Pre-Northridge	8%	20%	80%	100%
Post-Northridge	0%	10%	50%	100%
Mean Structural Loss Rates (Dollars per Square Foot)				
Pre-Northridge	$2.00	$5.00	$20.00	$25.00
Post-Northridge	$0.00	$2.50	$12.50	$25.00

Repair time is the time required for cleanup and construction to repair or replace damage to the structural system. Recovery time is the time required to make repairs, considering, for example, delays in decision-making, financing, and inspection, and typically takes much longer than the actual time of repair. Loss of function is the time that the facility is not available for use and is typically less than repair (recovery) time. Loss of function is less than repair time due to temporary solutions, such as the use of alternative space, or simply because buildings with Slight or Moderate damage can remain partially or fully operational while repairs are made. Table B-

10 provides time for cleanup and construction, and loss of function multipliers for typical steel moment-frame (WSMF) buildings (mixed occupancy). The loss-of-function multipliers represent the fraction of the repair time for each damage state that the building would not be functional.

Table B-10 Cleanup and Construction Time and Loss-of-Function Multipliers for Typical WSMF Buildings

Building Connection Condition and Height	Structural Damage State			
	Slight	Moderate	Extensive	Complete
Mean Time of Repairs in Days (Cleanup and Construction)				
Pre-Northridge – 3-Story	5	30	90	180
Post-Northridge – 3-Story	0	20	90	180
Pre-Northridge – 9-Story	10	50	180	360
Post-Northridge – 9-Story	0	40	180	360
Pre-Northridge – 20-Story	15	75	240	480
Post-Northridge – 20-Story	0	60	240	480
Loss-of-Function Multipliers (Fraction of Building Cleanup and Construction Time)				
All Buildings	0.0	0.1	0.3	1.0

Commentary: The values given in Table B-10 are based on the default values of HAZUS adjusted for building height (size) and include time required for inspection of WSMF buildings with pre-Northridge connection conditions. HAZUS cleanup and repair times and the fractions of repair time that the building will not be functional vary widely, depending on the occupancy of the building. Values given in Table B-10 are considered appropriate for most commercial office buildings. In contrast to HAZUS default values, Slight structural damage was assumed to have no impact on building function (loss-of-function multiplier is equal to 0.0, in all cases), since structural inspections and repair of connections can typically be made while the building is in operation. The loss-of-function multiplier for Complete structural damage is 1.0, and assumes that the building is closed and that alternative space is not available.

B.6 Example Loss Estimates

This section develops example estimates of losses for typical 9-story Los Angeles buildings, designed to conform to the 1994 *Uniform Building Code*. Three building types are considered: (1) buildings with pre-Northridge connection conditions, (2) buildings with post-Northridge connection conditions, and (3) buildings with damaged pre-Northridge connection conditions. The example considers three levels of earthquake ground shaking that represent the Maximum Considered Earthquake (MCE), the Design Earthquake (DE) and one-half of the DE (½ DE) for regions of high seismicity (e.g., Los Angeles). The example first estimates peak building response (spectral displacement) as the intersection of building capacity curves and earthquake demand spectra. Building fragility damage and loss functions are then developed using default parameters of typical 9-story building properties provided in previous sections. Finally, mean building loss functions are developed as a function of building spectral displacement that illustrate a range of losses for MCE, DE, ½ DE, and other levels of spectral demand.

Commentary: The user is expected to have available an estimate of scenario earthquake ground shaking at the building site. Such an estimate may be obtained from site-specific hazard studies or from the 1997 USGS/NEHRP spectral contour design maps. For this example, 5%-damped response spectra were developed from the spectral contour maps representing a typical Los Angeles stiff soil site (Soil Profile Type D), not near an active fault. MCE ground shaking represents a sufficiently large magnitude event of long shaking duration that its approximate return period is between 1,000 to 2,500 years. The DE and ½ DE represent ground shaking of a large magnitude event and moderate magnitude event, respectively with approximate return periods of 500 and 100 years, respectively. Most of the steel moment-frame (WSMF) buildings damaged by the 1994 Northridge earthquake felt ground shaking that ranged between the ½ DE and DE levels illustrated in this example.

Figure B-4 shows the 5%-damped spectrum of the ½ DE, the capacity curves of buildings with pre-Northridge and post-Northridge connection conditions (solid symbols), and the demand curves of buildings with pre-Northridge, post-Northridge and damaged pre-Northridge connection conditions (open or shaded symbols).

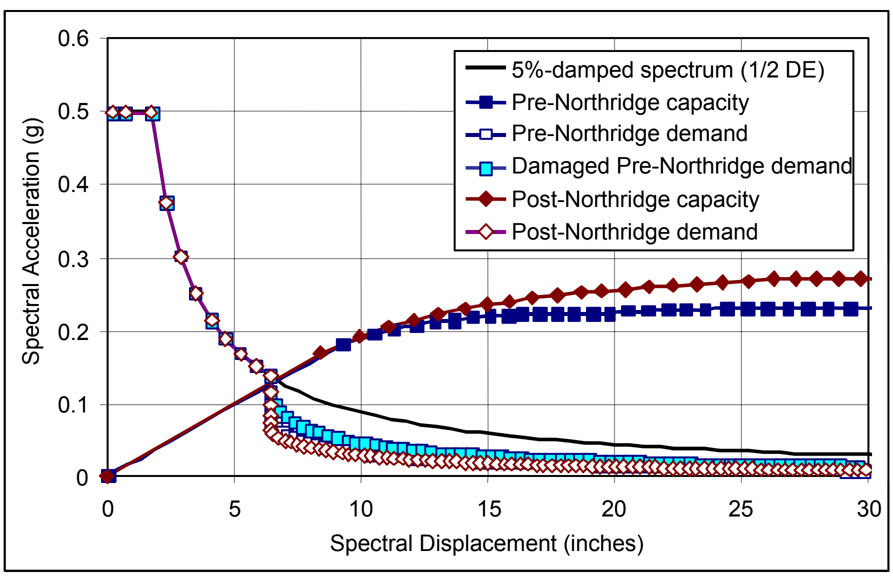


Figure B-4 Demand and Capacity of Typical 9-Story WSMF Buildings – Ground Shaking of ½ the Design Earthquake

The properties of the capacity curves are based on the yield and ultimate control points given in Table B-5. The demand spectra were constructed from the 5%-damped spectrum as described in Section B.5.2. The intersection points of demand and capacity curves indicate that spectral displacement of the building is about 6.5 inches for each building type. Figures B-5 and B-6 repeat the process and illustrate the determination of building spectral displacement for Design Earthquake (DE) and Maximum Considered Earthquake (MCE) ground shaking, respectively.

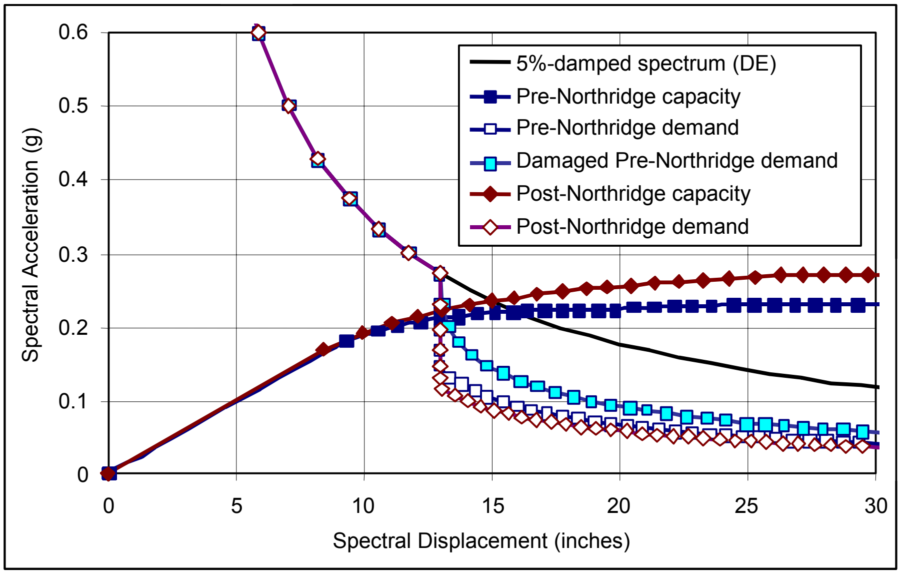


Figure B-5 Demand and Capacity of Typical 9-Story WSMF Buildings – Design Earthquake Ground Shaking

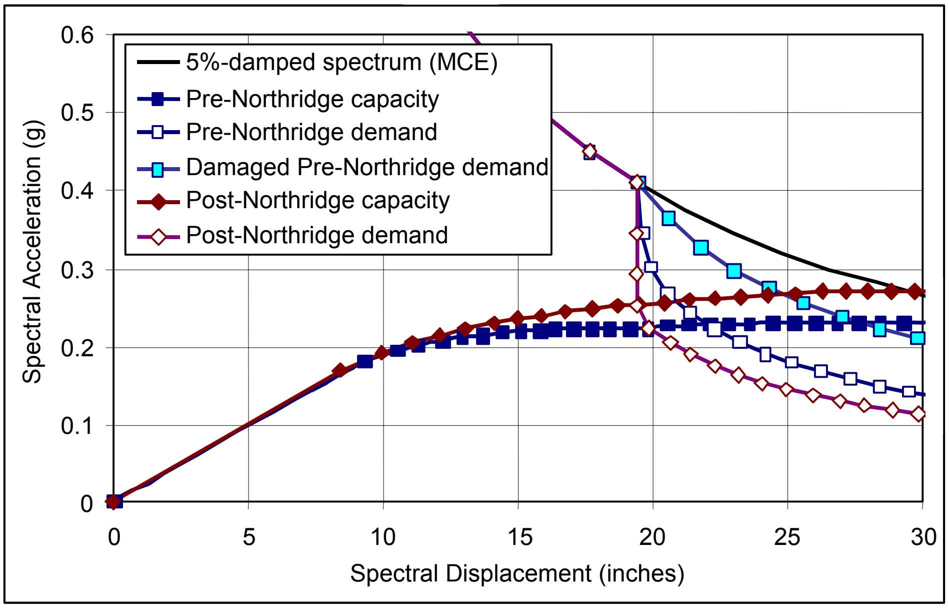


Figure B-6 Demand and Capacity of Typical 9-Story WSMF Buildings – Maximum Considered Earthquake Ground Shaking

Figure B-6 shows different Maximum Considered Earthquake (MCE) intersection points (i.e., different values of building spectral displacement) for the three building types. In particular, buildings with damaged pre-Northridge connection conditions are expected to degrade more than buildings with undamaged connections during the long duration of (post-yield) ground shaking associated with the MCE ($\kappa = 0.1$, Table B-6) and hence are expected to displace farther.

Table B-11 provides a summary of the predicted peak building response parameters for each of the three earthquake ground shaking levels. Spectral displacement is used later in this section to estimate structural damage and loss. Table B-11 shows spectral displacement values converted to corresponding estimates of average interstory drift, 1st-mode only, average interstory drift including higher modes, and maximum interstory drift including higher modes. Average interstory drift applies to all stories over the height of the building; maximum interstory drift applies to the story experiencing the most displacement. Estimates of drift are based on the height of the building (H = 122 feet) and the factors α_2, α_3 and $\alpha_{4,ds}$, defined in Section B.5.3.

Table B-11 Summary of Peak Response – Typical 9-Story WSMF Buildings

	Ground Shaking Level – Connection Condition				
	½ DE	DE	MCE – Long Duration		
Peak Response Parameter	**All**	**All**	**Post-NR**	**Pre-NR**	**Damaged**
Spectral Displacement (in.) SD	6.5	13	19.5	22	27.5
Average Interstory Drift - 1st-Mode (SD/H) x $1/\alpha_2$	0.006	0.012	0.018	0.020	0.026
Average Interstory Drift - All Modes (SD/H) x $1/\alpha_2$ x α_3	0.008	0.016	0.025	0.028	0.035
Maximum Interstory Drift – All Modes (SD/H) x $1/\alpha_2$ x α_3 x $\alpha_{4,S}$	0.010	0.021	0.031	0.035	0.043

Figure B-7 illustrates structural fragility curves for the example 9-story steel moment-frame (WSMF) Los Angeles buildings with post-Northridge connection conditions. These curves are constructed using Equation B-12 and the fragility parameters defined in Section B.5.3. Figure B-8 illustrates discrete damage-state probabilities for the same buildings. These curves are calculated as the difference in probability between adjacent damage-state fragility curves shown in Figure B-7. At each value of spectral displacement, the sum of discrete damage-state probabilities is equal to the probability of Slight or greater structural damage and the compliment of Slight or greater damage is the probability of no structural damage. The considerable overlap of discrete damage-state curves shown in Figure B-8 is a measure of the relatively large uncertainty in the prediction of damage and is due primarily to the inherent uncertainty in the prediction of ground shaking.

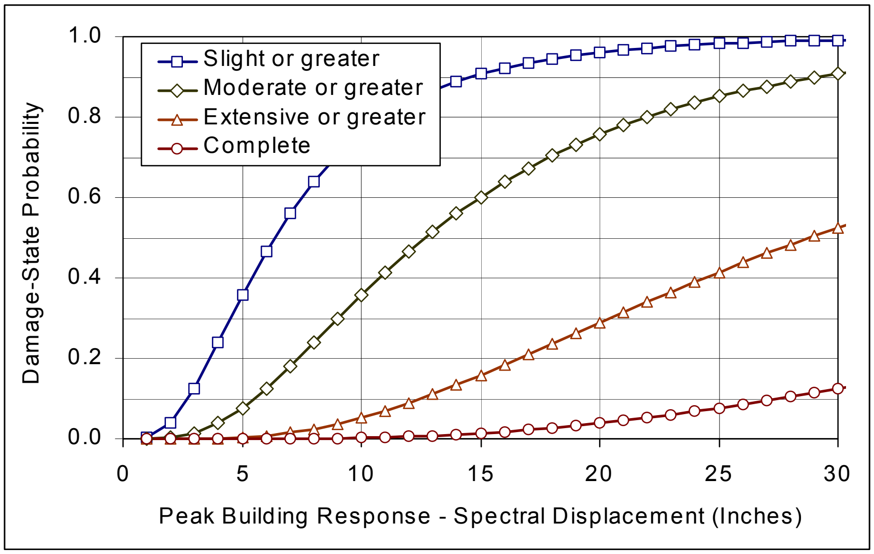


Figure B-7 Structural Fragility Curves – Typical 9-Story Los Angeles Buildings with Post-Northridge Connection Conditions

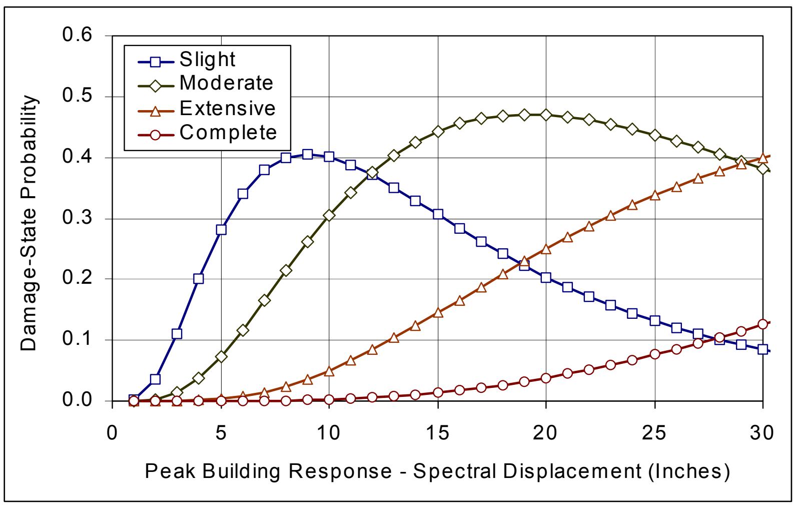


Figure B-8 Discrete Damage-State Probability Curves – Typical 9-Story Los Angeles Buildings with Post-Northridge Connection Conditions

Figure B-9 illustrates mean structural loss rates for the structural system of typical 9-story steel moment-frame (WSMF) Los Angeles buildings, expressed as a function of building spectral displacement. Structural loss ratios are shown for buildings with pre-Northridge and post-Northridge connection conditions to compare the typical reduction in postearthquake repair cost that would be expected for buildings with improved connections. Structural loss rates are the same for WSMF buildings with pre-Northridge connection conditions, with or without damage to connections, although buildings with damaged connections could, depending on the level and duration of ground shaking, experience larger spectral displacement and hence greater loss.

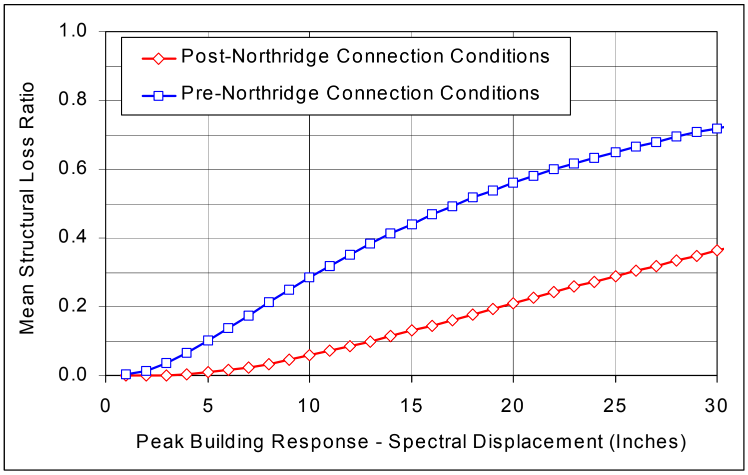


Figure B-9 Mean Structural Loss Ratio Curves – Typical 9-Story WSMF Los Angeles Buildings

Mean structural loss rate curves are constructed by first multiplying discrete damage-state probabilities, shown in Figure B-8, by the mean structural loss rates given in Table B-9, and then by summing the products over all damage states. Multiplying mean loss rates by the cost of the structural system produces mean estimates of the repair cost (including inspection cost for buildings with pre-Northridge connection conditions). For the typical 9-story buildings, the cost off the structural system is assumed to be about \$5 million (i.e., 20% x \$125/sq. ft. x 200,000 sq. ft.). Estimates of mean structural loss are made by finding the loss rate corresponding to the spectral displacement of the earthquake of interest (e.g., spectral displacement values given in Table B-11).

The results represent mean (or best) estimates of loss rates (rather than a complete distribution of loss), since loss rates represent mean (point estimates) of loss, given damage. Considering the rather large variability associated with damage estimates (which would only be made larger by considering loss uncertainty), actual loss for any given building could be significantly different than the mean estimate. The large uncertainty inherent in the fragility curves is reflected in the moderate slope of the curve for structural loss. At lower levels of loss, loss function tapers to zero gradually with decrease in building spectral displacement. Fragility uncertainty is primarily due to the uncertainty associated with median estimates of ground shaking. Actual ground shaking could be significantly higher (or lower) than the median and this uncertainty tends to broaden the loss functions, increasing estimates of loss at the low end and decreasing estimates of loss at the high end (which is typically beyond Maximum Considered Earthquake (MCE) demand).

The mean structural loss rate curves shown in Figure B-9 are plotted to spectral displacements of 30 inches, a displacement corresponding to an extremely rare level of earthquake ground shaking. Peak building spectral displacements likely to occur during the life of the building would not be expected to exceed the ½ DE level of ground shaking (i.e., about 6 inches, or less, of spectral displacement). Figure B-10 is a re-plot of Figure B-9 data to a spectral displacement of 10 inches. This figure shows that structural repair (and inspection) costs are likely not to exceed 13% of the cost of the structural system ($650,000 loss) on average during the life of the building. A loss of 13% is consistent with structural repair (and inspection) costs for steel moment-frame (WSMF) buildings damaged in the 1994 Northridge earthquake. The figure also shows that repair costs would likely not exceed 2% ($100,000 loss) on average for WSMF buildings with post-Northridge connection conditions. For comparison, a typical real estate transaction fee for a 200,000 square foot building, based only on the replacement value of the building (i.e., excluding the value of the land), would be in excess of $1 million each time the building is sold.

Figure B-11 illustrates mean functional loss (in days) due to damage of the structural system of typical 9-story WSMF Los Angeles buildings, expressed as a function of building spectral displacement. Functional loss is shown for buildings with pre-Northridge and post-Northridge connection conditions to compare the typical reduction in "downtime" for buildings with improved connections. Functional loss is the same for WSMF buildings with pre-Northridge connection conditions, regardless of connection damage, although buildings with damage connections could, depending on the level and duration of ground shaking, experience larger spectral displacement and hence greater loss.

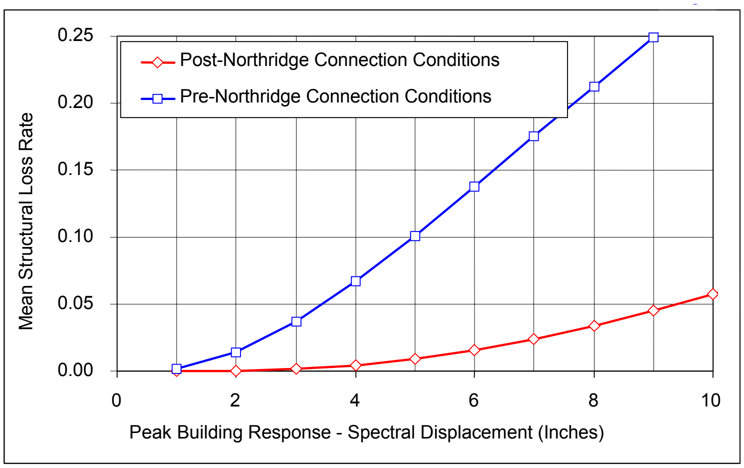


Figure B-10 Mean Structural Loss Rate Curves – Typical 9-Story WSMF Los Angeles Buildings

Mean loss of function curves are constructed by first multiplying discrete damage-state probabilities, shown in Figure B-8, by the product of the cleanup and construction time and the loss-of-function multipliers of Table B-10. Estimates of mean loss of function are made by finding the loss of time corresponding to the spectral displacement of the earthquake of interest (e.g., spectral displacement values given in Table B-11).

Mean loss of function (in days) is the probabilistic combination of short downtime due to Slight or Moderate structural damage, and long downtime due to Extensive or Complete structural damage. Complete damage is assumed to close the building for about the time it would take to build a new one (360 days for a 9-story WSMF building). Since the loss-of-function multipliers are very small for Slight or Moderate damage (repairs can usually be made while the building is in operation), loss function is dominated by the probability of Extensive or

Complete structural damage that would likely close the building for an extended period of time. While mean estimates of loss of function are valid as the average of many buildings, actual downtime of specific building could range from no loss of function to long-term building closure. It may make more sense for users to convert mean loss of function (in days) to a probability of long-term building closure by dividing the mean days of downtime by maximum down time associated with Complete structural damage. For example, a building with post-Northridge connection conditions is expected to have about 18 days of downtime due to Design Earthquake (DE) ground shaking. Actual downtime would likely be considerably less, provided the building did not sustain damage sufficient to warrant long-term closure (e.g., a red tag). In this case, the probability of long-term closure is about 5% (i.e., mean loss estimate of 18 days divided by 360 days of loss associated with Complete damage).

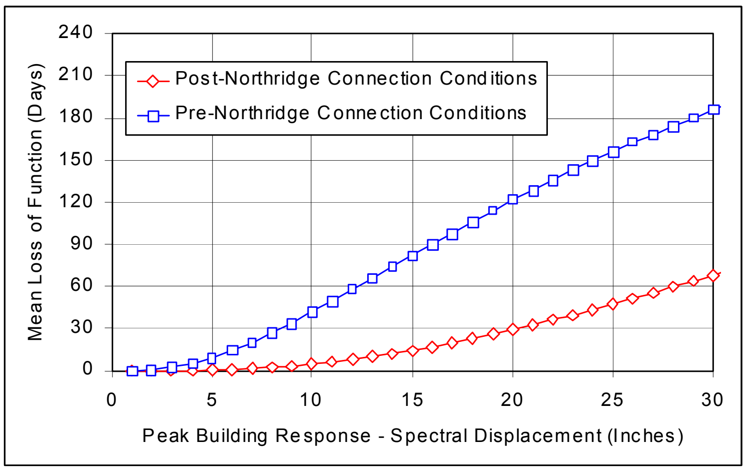


Figure B-11 Mean Loss of Function Curves – Typical 9-Story WSMF Los Angeles Buildings

REFERENCES, BIBLIOGRAPHY, AND ACRONYMS

This section contains references, additional bibliography and acronyms that are generally common to the set of reports, FEMA-350, FEMA-351, FEMA-352, and FEMA-353. Following the regular references are three sections containing ASTM Standards published by the American Society for Testing and Materials, West Conshohocken, Pennsylvania and listed numerically, AWS Specifications published by the American Welding Society, Miami, Florida, and listed numerically, FEMA Reports published by the Federal Emergency Management Agency, Washington, DC, and listed by report number, and SAC Reports published by the SAC Joint Venture, Sacramento, California, and listed by report number.

References and Additional Bibliography.

AISC, 1985, *Specification for Structural Joints using ASTM A325 or A490 Bolts,* American Institute of Steel Construction, Chicago, Illinois.

AISC, 1989, *Manual of Steel Construction*, ASD, Ninth Edition, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1993, 1997, *Load and Resistance Factor Design Specifications for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994a, *Proceedings of the AISC Special Task Committee on the Northridge Earthquake Meeting*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1994b, *Northridge Steel Update 1*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1997, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois.

AISC, 1998a, *Load and Resistance Factor Design Specifications for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, Illinois.

AISC, 1998b, *LRFD Manual of Steel Construction, 2nd Edition,* American Institute of Steel Construction, Chicago, Illinois.

AISC, 1999, *Supplement No. 1 to the 1997 Seismic Provisions for Structural Steel Buildings,* American Institute of Steel Construction, Chicago, Illinois.

Allen, J., Partridge, J.E., Richard, R.M., and Radau, S., 1995, "Ductile Connection Designs for Welded Steel Moment Frames," *Proceedings, 64th Annual Convention,* Structural Engineers Association of California, Sacramento, California.

Anderson, J, Duan, J., Xiao, Y., and Maranian, P., 2000, *Improvement of Welded Connections Using Fracture Tough Overlays*, Report No. SAC/BD-00/20, SAC Joint Venture, Sacramento, California.

ASCE, 1998, ASCE-7 maps, American Society of Civil Engineers, Reston, Virginia.

ASTM citations: see the list of ASTM Standards on page R-4.

ATC, 1985, *Earthquake Damage Evaluation Data for California,* Report ATC-13 , Applied Technology Council, Redwood City, California.

ATC, 1987, *Evaluating the Seismic Resistance of Existing Buildings*, Report ATC-14, Applied Technology Council, Redwood City, California.

ATC, 1989, *Procedures for Postearthquake Safety Evaluations of Buildings*, Report ATC-20, Applied Technology Council, Redwood City, California.

ATC, 1992, *Guidelines for Cyclic Seismic Testing of Components of Steel Structures*, Report ATC-24, Applied Technology Council, Redwood City, California.

ATC, 1995, *Addendum to the ATC-20 Postearthquake Building Safety Evaluation Procedures*, Report ATC-20-2, Applied Technology Council, Redwood City, California.

ATC, 1997a, *Seismic Evaluation and Retrofit of Concrete Buildings,* prepared by the Applied Technology Council (Report No. ATC-40), for the California Seismic Safety Commission (Report No. SSC 96-01), Sacramento, California.

ATC, 1997b, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* Report No. FEMA-273, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

ATC, 1997c, *Commentary to NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Report No. FEMA-274, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

Avent, R., 1992, "Designing Heat-Straightening Repairs," *National Steel Construction Conference Proceedings*, Las Vegas, Nevada.

AWS citations: see the list of AWS reports, specifications and codes on page R-5.

Barsom, J.M., 1996, "Steel Properties — Effects of Constraint, Temperature, and Rate of Loading," *Proceedings of the 2nd US Seminar, Seismic Design, Evaluation and Retrofit of Steel Bridges,* San Francisco, Report No. UCB/CEE STEEL-96/09, Dept. of Civil and Environmental Engineering, UC Berkeley, pp.115-143.

Boore, D.M., and Joyner, W.B., 1994, *Proceedings of Seminar on New Developments in Earthquake Ground-Motion Estimation and Implications for Engineering Design Practice*, Report ATC-35-1, Applied Technology Council, Redwood City, California, pp 6-1 to 6-41.

BSSC, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings,* developed by the Building Seismic Safety Council for the Federal Emergency Management Agency, Report FEMA-178, Washington, D.C.

BSSC, 1997a, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 – Provisions*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency (Report No. FEMA-302), Washington, DC.

BSSC, 1997b, *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 – Commentary*, prepared by the Building Seismic

Safety Council for the Federal Emergency Management Agency (Report No. FEMA-303), Washington, DC.

Campbell, K.W., and Bozorgnia, Y., 1994, "Near-Source Attenuation of Peak Horizontal Acceleration from Worldwide Accelerograms Recorded from 1957 to 1993*," Fifth U.S. National Conference on Earthquake Engineering, Proceedings*, Vol. III, pp 283-292, Earthquake Engineering Research Institute, Oakland, California.

Chi, W.M., Deierlein, G., and Ingraffea, A., 1997, "Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections," SAC Joint Venture, Report No. SAC/BD-97/05.

FEMA citations: see the list of FEMA reports on page R-6.

Goel, R.K., and Chopra, A.K., 1997, "Period Formulas for Moment-Resisting Frame Buildings," *Journal of Structural Engineering*, Vol. 123, No. 11, pp. 1454-1461.

Gross, J.L., Engelhardt, M.D., Uang, C.M., Kasai, K. and Iwankiw, N.R., 1999, *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance*, *AISC Design Guide Series 12*, American Institute of Steel Construction, Chicago, Illinois.

Grubbs, K., 1997, "The Effect of the Dogbone Connection on the Elastic Stiffness of Steel Moment Frames" Masters Thesis, Department of Civil Engineering, University of Texas at Austin.

ICBO, 1988, 1991, and 1997, *Uniform Building Code, indicated edition,* International Conference of Building Officials, Whittier, California.

ICC, 2000, *International Building Code,* International Code Council, Falls Church, Virginia.

Kircher, C.A., Nassar, A.A., Kustu, O. and Holmes, W.T., 1997, "Development of Building Damage Functions for Earthquake Loss Estimation," *Earthquake Spectra,* Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 663-682.

Kircher, C.A., Reitherman, R.K., Whitman, R.V., and Arnold, C., 1997, "Estimation of Earthquake Losses to Buildings," *Earthquake Spectra*, Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 703-720.

Kircher, C.A., 1999, *Procedures for Development of HAZUS-Compatible Building-Specific Damage and Loss Functions*, National Institute of Building Sciences, Washington, D.C.

Krawinkler, H., Gupta, A., Medina, R. and Luco, N., 2000, *Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies*, Report No. SAC/BD-00/10, SAC Joint Venture, Sacramento, California.

NIBS, 1997a, *HAZUS Earthquake Loss Estimation Methodology, Users Manual,* National Institute of Building Sciences, Washington, DC.

NIBS, 1997b, *HAZUS Earthquake Loss Estimation Methodology, Technical Manual*, 3 Volumes. National Institute of Building Sciences, Washington, DC.

RCSC, 1996, *Load and Resistance Factor Design: Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Research Council on Structural Connections.

Richard, R., Partridge, J.E., Allen, J., and Radau, S., 1995, "Finite Element Analysis and Tests of Beam-to-Column Connections," *Modern Steel Construction*, Vol. 35, No. 10, pp. 44-47, American Institute of Steel Construction, Chicago, Illinois.

SAC citations: see the list of SAC Joint Venture reports on page R-7.

Shonafelt, G.O., and Horn, W.B, 1984, *Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*, NCHRP Report 271, prepared by the National Cooperative Highway Research Program, for the Transportation Research Board, Washington, DC.

Wald, D.J., Quitoriano, T.H., Kanamori, H. and Scrivner, C.W., 1998, "Trinet Shakemaps – Rapid Generation of Peak Ground Motion and Intensity Maps for Earthquakes in Southern California", *SMIP98 Proceedings*, California Division of Mines and Geology, Sacramento, California.

Whitman, R., Anagnos, T., Kircher, C., Lagorio, H.J., Lawson, R.S., and Schneider, P., 1997, "Development of a National Earthquake Loss-Estimation Methodology," *Earthquake Spectra*, Vol. 13, No. 4, Earthquake Engineering Research Institute, Oakland, California, pp. 643-661.

Youssef, N.F.G, Bonowitz, D., and Gross, J.L., 1995, *A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake*, Report No. NISTR 56254, National Institute for Science and Technology, Gaithersburg, Maryland.

ASTM Standards.

ASTM Standards are published by the American Society for Testing and Materials, West Conshohocken, Pennsylvania, and are listed alphanumerically.

ASTM, 1997, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A6, *Supplementary Requirement S5*

A36, *Specification for Carbon Structural Steel*

A307-97, *Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength*

A325, *Specification for Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength*

A435, *Straight Beam Ultrasonic Examination of Steel Plates*

A490, *Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A563, *Specification for Carbon and Alloy Steel Nuts*

A572, *Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A370-97a, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A898, *Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes*

A913, *Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process*

A992, *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

E329, *Standard Specification for Agencies Engaged in the Testing and/or Inspection of Material Used in Construction*

E543, *Standard Practice for Agencies Performing Nondestructive Testing*

E548, *Standard Guide for General Criteria Used for Evaluating Laboratory Competence*

E994, *Standard Guide for Laboratory Accreditation Systems*

E1212, *Standard Practice for Establishment and Maintenance of Quality Control Systems for Nondestructive Testing Agencies*

E1359*, Standard Guide for Surveying Nondestructive Testing Agencies*

F436, *Specification for Hardened Steel Washers*

F606-98, *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets*

F959, *Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F1554, *Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

F1852, *Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

AWS Reports, Specifications, and Codes.

AWS reports are published by the American Welding Society, Miami, Florida, and are listed alphanumerically.

AWS A2.4, *Standard Symbols for Welding, Brazing, and Nondestructive Testing*

AWS A4.3, *Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding*

ANSI/AWS A5.1-91, *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

ANSI/AWS A5.18-93, *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

ANSI/AWS A5.20-95, *Specification for Carbon Steel Electrodes for Flux-Cored Arc Welding*

AWS, 1995, *Presidential Task Group Report*

ANSI/AWS A5.5-96*, Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

ANSI/AWS A5.28-96, *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

ANSI/AWS A5.23/A5.23M-97, *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

ANSI/AWS A5.25/A5.25M-97, *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*

ANSI/AWS A5.26/A5.26M-97, *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*

ANSI/AWS A5.32/A5.32M-97, *Specification for Welding Shielding Gases*

ANSI/AWS A5.17/A5.17M-97, *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

ANSI/AWS A5.29-98, *Specification for Low-Alloy Steel Electrodes for Flux-Cored Arc Welding*

AWS D1.1-1998, 2000, *Structural Welding Code – Steel*

AWS D1.3, *Structural Welding Code*

AWS D1.4, *Structural Welding Code*

AWS QC1, *Standard for AWS Certification of Welding Inspectors*

FEMA Reports.

FEMA reports are listed by report number.

FEMA-178, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-267, 1995, *Interim Guidelines, Inspection, Evaluation, Repair, Upgrade and Design of Welded Moment Resisting Steel Structures,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-267A, 1996, *Interim Guidelines Advisory No. 1,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-267B, 1999, *Interim Guidelines Advisory No. 2,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-274, 1997, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-302, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 – Provisions*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-303, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 – Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-310, 1998, *Handbook for the Seismic Evaluation of Buildings – A Prestandard*, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, DC.

FEMA-350, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-351, 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-352, 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-353, 2000, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-354, 2000, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355A, 2000, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355B, 2000, *State of the Art Report on Welding and Inspection,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355C, 2000, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355D, 2000, *State of the Art Report on Connection Performance,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355E, 2000, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355F, 2000, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC Joint Venture Reports.

SAC Joint Venture reports are listed by report number, except for SAC 2000a through 2000k; those entries that do not include a FEMA report number are published by the SAC Joint Venture.

SAC 94-01, 1994*, Proceedings of the Invitational Workshop on Steel Seismic Issues, Los Angeles,* September 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 94-01, 1994b, *Proceedings of the International Workshop on Steel Moment Frames*, *Sacramento,* December, 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-01, 1995, *Steel Moment Frame Connection Advisory No. 3*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-02, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267, Washington, DC.

SAC 95-03, 1995, *Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-04, 1995, *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-05, 1995, *Parametric Analytic Investigations of Ground Motion and Structural Response, Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-06, 1995, *Technical Report: Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-07, 1995, *Technical Report: Case Studies of Steel Moment-Frame Building Performance in the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-08, 1995, *Experimental Investigations of Materials, Weldments and Nondestructive Examination Techniques*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-09, 1995, *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-288, Washington, DC.

SAC 96-01, 1996, *Experimental Investigations of Beam-Column Subassemblages, Part 1 and 2*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 96-02, 1996, *Connection Test Summaries*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-289, Washington, DC.

SAC 96-03, 1997, *Interim Guidelines Advisory No. 1 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267A, Washington, DC.

SAC 98-PG, *Update on the Seismic Safety of Steel Buildings – A Guide for Policy Makers,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 99-01, 1999, *Interim Guidelines Advisory No. 2 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture, for the Federal Emergency Management Agency, Report No. FEMA-267B, Washington, DC.

SAC, 2000a, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-350, Washington, D.C.

SAC, 2000b, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-351, Washington, D.C.

SAC, 2000c, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-352, Washington, D.C.

SAC, 2000d, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-353, Washington, D.C.

SAC, 2000e, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-354, Washington, D.C.

SAC, 2000f, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355A, Washington, D.C.

SAC, 2000g, *State of the Art Report on Welding and Inspection*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355B, Washington, D.C.

SAC, 2000h, *State of the Art Report on Systems Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355C, Washington, D.C.

SAC, 2000i, *State of the Art Report on Connection Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355D, Washington, D.C.

SAC, 2000j, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355E, Washington, D.C.

SAC, 2000k, *State of the Art Report on Performance Prediction and Evaluation*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355F, Washington, D.C.

SAC/BD-96/01, *Selected Results from the SAC Phase 1 Beam-Column Connection Pre-Test Analyses*, submissions from B. Maison, K. Kasai, and R. Dexter; and A. Ingraffea and G. Deierlein.

SAC/BD-96/02, *Summary Report on SAC Phase 1 - Task 7 Experimental Studies*, by C. Roeder (a revised version of this document is published in Report No. SAC 96-01; the original is no longer available).

SAC/BD-96/03, *Selected Documents from the U.S.-Japan Workshop on Steel Fracture Issues*.

SAC/BD-96/04, *Survey of Computer Programs for the Nonlinear Analysis of Steel Moment Frame Structures*.

SAC/BD-97/01, *Through-Thickness Properties of Structural Steels*, by J. Barsom and S. Korvink.

SAC/BD-97/02, *Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens*, by P. Clark, K. Frank, H. Krawinkler, and R. Shaw.

SAC/BD-97/03, *Proposed Statistical and Reliability Framework for Comparing and Evaluating Predictive Models for Evaluation and Design*, by Y.-K. Wen.

SAC/BD-97/04, *Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project*, by. P. Somerville, N. Smith, S. Punyamurthula, and J. Sun.

SAC/BD-97/05, *Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections*, by W.-M. Chi, G. Deierlein, and A. Ingraffea.

SAC/BD-98/01, *Strength and Ductility of FR Welded-Bolted Connections*, by S. El-Tawil, T. Mikesell, E. Vidarsson, and S. K. Kunnath.

SAC/BD-98/02, *Effects of Strain Hardening and Strain Aging on the K-Region of Structural Shapes*, by J. Barsom and S. Korvink

SAC/BD-98/03, *Implementation Issues for Improved Seismic Design Criteria: Report on the Social, Economic, Policy and Political Issues Workshop* by L.T. Tobin.

SAC/BD-99/01, *Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae

SAC/BD-99/01A, *Appendix to: Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae

SAC/BD-99/02, *Through-Thickness Strength and Ductility of Column Flange in Moment Connections* by R. Dexter and M. Melendrez.

SAC/BD-99/03, *The Effects of Connection Fractures on Steel Moment Resisting Frame Seismic Demands and Safety* by C. A. Cornell and N. Luco

SAC/BD-99/04, *Effects of Strength/Toughness Mismatch on Structural and Fracture Behaviors in Weldments* by P. Dong, T. Kilinski, J. Zhang and F.W. Brust

SAC/BD-99/05, *Assessment of the Reliability of Available NDE Methods for Welded Joint and the Development of Improved UT Procedures* by G. Gruber and G. Light

SAC/BD-99/06*, Prediction of Seismic Demands for SMRFs with Ductile Connections and Elements* by A. Gupta and H. Krawinkler

SAC/BD-99/07, *Characterization of the Material Properties of Rolled Sections* by T. K. Jaquess and K. Frank

SAC/BD-99/08, *Study of the Material Properties of the Web-Flange Intersection of Rolled Shapes* by K. R. Miller and K. Frank

SAC/BD-99/09, *Investigation of Damage to WSMF Earthquakes other than Northridge* by M. Phipps

SAC/BD-99/10, *Clarifying the Extent of Northridge Induced Weld Fracturing and Examining the Related Issue of UT Reliability* by T. Paret

SAC/BD-99/11, *The Impact of Earthquakes on Welded steel Moment Frame Buildings: Experience in Past Earthquakes* by P. Weinburg and J. Goltz

SAC/BD-99/12, *Assessment of the Benefits of Implementing the New Seismic Design Criteria and Inspection Procedures* by H. A. Seligson and R. Eguchi

SAC/BD-99/13, *Earthquake Loss Estimation for WSMF Buildings*, by C. A. Kircher

SAC/BD-99/14, *Simplified Loss Estimation for Pre-Northridge WSMF Buildings*, by B. F. Maison and D. Bonowitz

SAC/BD-99/15, *Integrative Analytical Investigations on the Fracture Behavior of Welded Moment Resisting Connections*, by G. G. Deierlein and W.-M. Chi

SAC/BD-99/16, *Seismic Performance of 3 and 9 Story Partially Restrained Moment Frame Buildings*, by B. F. Maison and K. Kasai

SAC/BD-99/17, *Effects of Partially-Restrained Connection Stiffness and Strength on Frame Seismic Performance*, by K. Kasai, B. F. Maison, and A. Mayangarum

SAC/BD-99/18, *Effects of Hysteretic Deterioration Characteristics on Seismic Response of Moment Resisting Steel Structures*, by F. Naeim, K. Skliros, A. M. Reinhorn and M.V. Sivaselvan

SAC/BD-99/19, *Cyclic Instability of Steel Moment Connections with Reduced Beam Section*, by C.-M. Uang and C.-C. Fan

SAC/BD-99/20, *Local and Lateral-Torsion Buckling of Wide Flange Beams*, by L. Kwasniewski, B. Stojadinovic, and S. C. Goel

SAC/BD-99/21, *Elastic Models for Predicting Building Performance*, by X. Duan and J. C. Anderson

SAC/BD-99/22, *Reliability-Based Seismic Performance Evaluation of Steel Frame Buildings Using Nonlinear Static Analysis Methods*, by G. C. Hart and M. J. Skokan

SAC/BD-99/23, *Failure Analysis of Welded Beam to Column Connections*, by J. M. Barsom

SAC/BD-99/24, *Weld Acceptance Criteria for Seismically-Loaded Welded Connections*, by W. Mohr

SAC/BD-00/01, *Parametric Tests on Unreinforced Connections*, by K.-H. Lee, B. Stojadinovic, S. C. Goel, A. G. Margarian, J. Choi, A. Wongkaew, B. P. Reyher, and D.-Y, Lee

SAC/BD-00/02, *Parametric Tests on the Free Flange Connections*, by J. Choi, B. Stojadinovic, and S. C. Goel

SAC/BD-00/03, *Cyclic Tests on Simple Connections Including Effects of the Slab*, by J. Liu and A. Astaneh-Asl

SAC/BD-00/04, *Tests on Bolted Connections*, by J. Swanson, R. Leon and J. Smallridge

SAC/BD-00/05, *Bolted Flange Plate Connections*, by S. P. Schneider and I. Teeraparbwong

SAC/BD-00/06, *Round Robin Testing of Ultrasonic Testing Technicians*, by R. E. Shaw, Jr.

SAC/BD-00/07, *Dynamic Tension Tests of Simulated Welded Beam Flange Connections*, by J. M. Ricles, C. Mao, E. J. Kaufmann, L.-W. Lu, and J. W. Fisher

SAC/BD-00/08, *Design of Steel Moment Frame Model Buildings in Los Angeles, Seattle and Boston*, by P. Clark

SAC/BD-00/09, *Benchmarking of Analysis Programs for SMRF System Performance Studies*, by A. G. and H. Krawinkler

SAC/BD-00/10, *Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies*, by H. Krawinkler, A. Gupta, R. Medina and N. Luco

SAC/BD-00/11, *Development of Improved Post-Earthquake Inspection Procedures for Steel Moment Frame Buildings*, by P. Clark

SAC/BD-00/12, *Evaluation of the Effect of Welding Procedure on the Mechanical Properties of FCAW-S and SMAW Weld Metal Used in the Construction of Seismic Moment Frames*, by M. Q. Johnson

SAC/BD-00/13, *Preliminary Evaluation of Heat Affected Zone Toughness in Structural Shapes Used in the Construction of Seismic Moment Frames*, by M. Q. Johnson

SAC/BD-00/14, *Evaluation of Mechanical Properties in Full-Scale Connections and Recommended Minimum Weld Toughness for Moment Resisting Frames*, by M. Q. Johnson, W. Mohr, and J. Barsom

SAC/BD-00/15, *Simplified Design Models for Predicting the Seismic Performance of Steel Moment Frame Connections*, by C. Roeder, R.G. Coons, and M. Hoit

SAC/BD-00/16, *SAC Phase 2 Test Plan*, by C. Roeder

SAC/ BD-00/17, *Behavior and Design of Radius-Cut, Reduced Beam Section Connections*, by M. Engelhardt, G. Fry, S. Johns, M. Venti, and S. Holliday

SAC/BD-00/18, *Test of a Free Flange Connection with a Composite Floor Slab*, by M. Venti and M. Engelhardt

SAC/BD-00/19, *Cyclic Testing of a Free Flange Moment Connection* by C. Gilton, B. Chi, and C. M. Uang

SAC/BD-00/20, *Improvement of Welded Connections Using Fracture Tough Overlays*, by James Anderson, J. Duan, P. Maranian, and Y. Xiao

SAC/BD-00/21, *Cyclic Testing of Bolted Moment End-Plate Connections*, by T. Murray and E. Sumner

SAC/BD-00/22, *Cyclic Response of RBS Moment Connections: Loading Sequence and Lateral Bracing Effects*, by Q.S. Yu, C. Gilton, and C. M. Uang

SAC/BD-00/23, *Cyclic Response of RBS Moment Connections: Weak Axis Configuration and Deep Column Effects*, by C. Gilton, B. Chi, and C. M. Uang

SAC/BD-00/24, *Development and Evaluation of Improved Details for Ductile Welded Unreinforced Flange Connections*, by J.M. Ricles, C. Mao, L.W. Lu, and J. Fisher

SAC/BD-00/25, *Performance Prediction and Evaluation of Steel Special Moment Frames for Seismic Loads*, by K. Lee and D. A. Foutch

SAC/BD-00/26, *Performance Prediction and Evaluation of Low Ductility Steel Moment Frames for Seismic Loads*, by S. Yun and D. A. Foutch

SAC/BD-00/27, *Steel Moment Resisting Connections Reinforced with Cover and Flange Plates*, by T. Kim, A.S. Whittaker, V.V. Bertero, and A.S.J. Gilani

SAC/BD-00/28, *Failure of a Column K-Area Fracture*, by J.M. Barsom and J.V. Pellegrino

SAC/BD-00/29, *Inspection Technology Workshop*, by R. E. Shaw, Jr.

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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