Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings

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

DISCLAIMER

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

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

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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* and the 1997 *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. ATC is a nonprofit corporation founded to develop structural engineering resources and applications to mitigate the effects of natural and other hazards on the built environment. Since its inception in the early 1970s, ATC has developed the technical basis for the current model national seismic design codes for buildings; the *de-facto* national standard for postearthquake safety evaluation of buildings; nationally applicable guidelines and procedures for the identification, evaluation, and rehabilitation of seismically hazardous buildings; and other widely used procedures and data to improve structural engineering practice. CUREe is a nonprofit organization formed to promote and conduct research and educational activities related to earthquake hazard mitigation. CUREe's eight institutional members are the California Institute of Technology, Stanford University, the University of California at Berkeley, the University of California at Davis, the University of California at Irvine, the University of California at Los Angeles, the University of California at San Diego, and the University of Southern California. These university earthquake research laboratory, library, computer and faculty resources are among the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by consultants and subcontractor universities and organizations from across the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of steel moment-frame structures.

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1. INTRODUCTION

1.1   Purpose

This report, *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings,* has been developed by the SAC Joint Venture under contract to the Federal Emergency Management Agency (FEMA) to provide communities and organizations developing programs for the assessment, occupancy status, and repair of welded steel moment-frame buildings that have been subjected to the effects of strong earthquake ground shaking. 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 postearthquake damage assessment and repair criteria, hereinafter referred to as *Recommended Criteria,* is presented in the form of specific damage assessment, safety evaluation and repair procedures together with supporting commentary explaining part of the basis for these recommendations. Detailed derivations and explanations of the basis for these engineering recommendations may be found in a series of State of the Art Reports prepared in parallel with these *Recommended Criteria*. These reports include:

- *FEMA-355A – State of the Art Report on Base Metals and Fracture.* This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties.

- *FEMA-355B – State of the Art Report on Welding and Inspection.* This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction.

- *FEMA-355C – State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking.* This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behavior of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites.

- *FEMA-355D – State of the Art Report on Connection Performance.* This report summarizes the current state of knowledge of the performance of different types of moment-resisting connections under large inelastic deformation demands. It includes information on fully restrained, partially restrained, and partial strength connections, both welded and bolted, based on laboratory and analytical investigations.

- *FEMA-355E – State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes.* This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events.

- *FEMA-355F – State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings.* This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria documents, *FEMA-350, FEMA-351*, and *FEMA-352*.

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

1.2   Intent

These *Recommended Criteria* are primarily intended as a resource document for communities developing formal programs for the assessment, occupancy status, and repair of buildings that have

been subjected to the effects of strong earthquake ground shaking. They are also intended for direct use by engineers and building officials in communities without such formal programs. 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 buildings. 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.

The fundamental goal of the information presented in these *Recommended Criteria* is to assist the technical community in implementing effective programs for:

- evaluation of steel moment-frame buildings affected by strong earthquake ground shaking to determine if they have been damaged, and to what extent,

- identification of those buildings that have been so severely damaged that they constitute a significant safety hazard, and

- repair of damaged structures such that they may safely be restored to long term occupancy.

> *Commentary: When a severe earthquake effects a community, many buildings are likely to become damaged and some, as a result of this damage, may pose a significant safety hazard. In the past, building officials in such communities, in fulfillment of their charge to protect the public safety through regulation of building occupancy, have instituted programs of building inspection and posting to provide guidance to the public on the condition of affected structures and whether they should be entered. Depending on the individual community and its resources, the task of inspection and posting may be conducted by the building department staff, by volunteer engineers and architects, by private consultants retained by individual building owners, or by a combination of these. Due to the limited resources available, it is usually necessary to limit these postearthquake inspections to those structures most likely to have been severely damaged and to make a rapid assessment of the severity of damage.*
>
> *Following initial postearthquake assessment, buildings are typically tagged with a placard to inform the owner and public of the assessed condition. "Green tags" are typically used to indicate that the building has been subjected to a rapid inspection and does not appear to have sustained damage that impairs its safety for occupancy. "Yellow tags" are typically used to indicate a condition of limited, or perhaps unknown, impairment of building safety. "Red tags" are commonly used to indicate that a building has been assessed as unsafe for further occupancy. Once a building has been posted with either a yellow or red tag, the building owner must take action to clear this posting. Typically the owner must retain a consultant to perform more detailed inspections and evaluations, and*

either report back to the building official that the building was not seriously damaged, or to prepare recommendations for repair of the structure and to have the posting removed. Note that only the building official is authorized to allow the posting to be altered or removed.

These Recommended Criteria provide guidelines for performing the rapid post-earthquake assessments, typically conducted by the building official; for performing the more detailed assessments, typically performed by a private consultant under contract to the building owner; and for developing repair programs. These repair programs are intended to restore the structure to the approximate condition and level of safety that existed prior to the onset of damage in this particular earthquake event. These Recommended Criteria do not specifically provide recommendations for upgrade of a building, to improve its performance in the event of future earthquake ground shaking.

In many cases, when a building experiences severe damage in a relatively moderate event, this damage is an indication that the building is vulnerable and could experience more extensive and severe damage in future events. In recognition of this, many locally adopted building codes contain provisions that require upgrade of structures, as well as repair, when they have been damaged beyond a certain level. This "trigger" level for upgrade varies widely from community to community. Regardless of whether or not the local building code requires upgrade as well as repair, an upgrade should be considered by the owner at the time structural repairs are conducted. For technical criteria for evaluating the advisability of upgrades, and methods of designing such upgrades, refer to FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.

When a decision is made to repair a structure, without upgrade, the engineer is cautioned to alert the owner that similar or perhaps more severe damage could be anticipated in future events. Further, the engineer should take care that in the process of conducting repairs, conditions of structural irregularity, discontinuity, or strength or stiffness deficiency are not introduced into the structure, and that existing such conditions are not made more severe.

1.3   Background

For many years, the basic intent of the building code seismic provisions has been to provide buildings with an ability to withstand intense ground shaking without collapse, but potentially with some significant structural damage. In order to accomplish this, one of the basic principles inherent in modern code provisions is to encourage the use of building configurations, structural systems, materials and details that are capable of ductile behavior. A structure is said to behave in a ductile manner if it is capable of withstanding large inelastic deformations without significant degradation in strength, and without the development of instability and collapse. The design forces specified by building codes for particular structural systems are related to the

amount of ductility the system is deemed to possess. Generally, structural systems with more ductility are designed for lower forces than less ductile systems, as ductile systems are deemed capable of resisting demands that are significantly greater than their elastic strength limit. Starting in the 1960s, engineers began to regard welded steel moment-frame buildings as being among the most ductile systems contained in the building code. Many engineers believed that steel moment-frame buildings were essentially invulnerable to earthquake-induced structural damage and thought that should such damage occur, it would be limited to ductile yielding of members and connections. Earthquake-induced collapse was not believed possible. Partly as a result of this belief, many large industrial, commercial and institutional structures employing steel moment-frame systems were constructed, particularly in the western United States.

The Northridge earthquake of January 17, 1994 challenged this paradigm. Following that earthquake, a number of steel moment-frame buildings were found to have experienced brittle fractures of beam-to-column connections. The damaged buildings had heights ranging from one story to 26 stories, and a range of ages spanning from buildings as old as 30 years to structures being erected at the time of the earthquake. The damaged buildings were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few buildings were located on sites that experienced the strongest ground shaking, damage to buildings on these sites was extensive. Discovery of these unanticipated brittle fractures of framing connections, often with little associated architectural damage 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, 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.

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

Steel moment-frame buildings are anticipated to develop their ductility through the development of yielding in beam-column assemblies at the beam-column connections. This

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

Observation of damage sustained by buildings in the 1994 Northridge earthquake indicated that contrary to the intended behavior, in many cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-2). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.
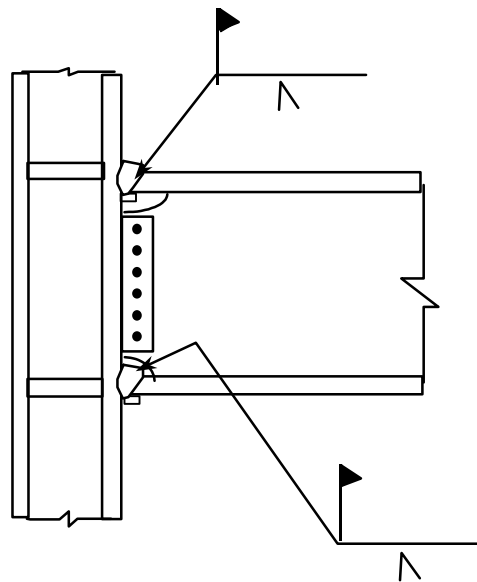
Figure 1-1   Typical Welded Moment-Resisting Connection Prior to 1994

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

A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone (Figure 1-4b). Investigators have reported some instances where columns fractured entirely across the section.
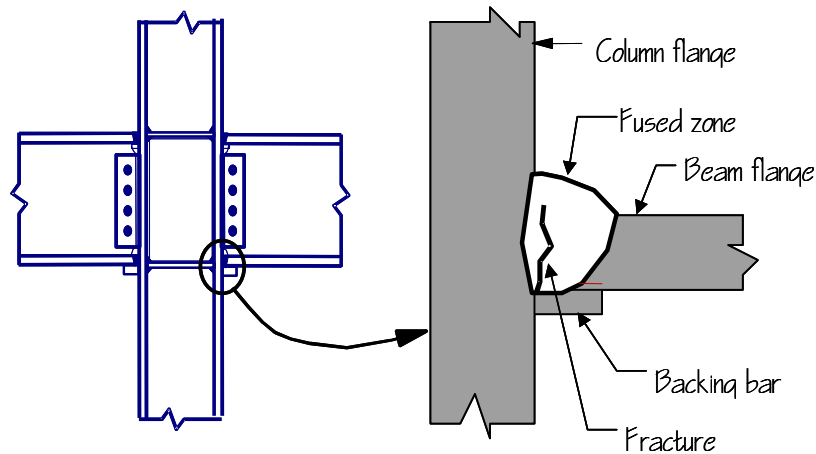
Figure 1-2   Common Zone of Fracture Initiation in Beam-Column Connection

a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

Figure 1-3   Fractures of Beam to Column Joints

a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-4   Column Fractures

Once such fractures have occurred, the beam-column connection has experienced a
significant loss of flexural rigidity and strength to resist those loads that tend to open the crack.
Residual flexural strength and rigidity must be developed through a couple consisting of forces
transmitted through the remaining top flange connection and the web bolts. However, in

providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures. These include fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or damage to architectural elements, making reliable postearthquake damage evaluations difficult. In order to determine 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 steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.
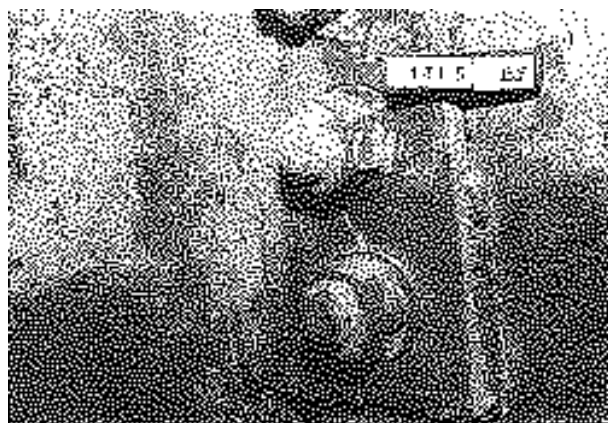
Figure 1-5   Vertical Fracture through Beam Shear Plate Connection

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

In September, 1994, the SAC Joint Venture, AISC, the American Iron and Steel Institute and National Institute of Standards and Technology jointly convened an international workshop (SAC, 1994) in Los Angeles to coordinate the efforts of the various participants and to lay the foundation for systematic investigation and resolution of the problem. Following this workshop, FEMA entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused studies of the seismic performance of steel moment-frame buildings and to develop

recommendations for professional practice (Phase I of SAC Steel Project). Specifically, these recommendations were intended to address the following: the inspection of earthquake-affected buildings to determine if they had sustained significant damage; the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide reliable seismic performance.

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

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

- The most severe stresses in the connection assembly occur where the beam joins to the column. Unfortunately, this is also the weakest location in the assembly. At this location, bending moments and shear forces in the beam must be transferred to the column through the combined action of the welded joints between the beam flanges and column flanges and the shear tab. The combined section properties of these elements, for example the cross sectional area and section modulus, are typically less than those of the connected beam. As a result, stresses are locally intensified at this location.

- The joint between the bottom beam flange and the column flange is typically made as a downhand field weld, often by a welder sitting on top of the beam top flange, in a so-called "wildcat" position. To make the weld from this position each pass must be interrupted at the beam web, with either a start or stop of the weld at this location. This welding technique often results in poor quality welding at this critical location, with slag inclusions, lack of fusion and other defects. These defects can serve as crack initiators, when the connection is subjected to severe stress and strain demands.

- The basic configuration of the connection makes it difficult to detect hidden defects at the root of the welded beam-flange-to-column-flange joints. The backing bar, which was typically left in place following weld completion, restricts visual observation of the weld root. Therefore, the primary method of detecting defects in these joints is through the use of ultrasonic testing (UT). However, the geometry of the connection also makes it very difficult for UT to detect flaws reliably at the bottom beam flange weld root, particularly at the center of the joint, at the beam web. As a result, many of these welded joints have undetected significant defects that can serve as crack initiators.

- Although typical design models for this connection assume that nearly all beam flexural stresses are transmitted by the flanges and all beam shear forces by the web, in reality, due to boundary conditions imposed by column deformations, the beam flanges at the connection carry a significant amount of the beam shear. This results in significant flexural stresses on the beam flange at the face of the column, and also induces large secondary stresses in the welded joint. Some of the earliest investigations of these stress concentration effects in the welded joint were conducted by Richard, et al. (1995). The stress concentrations resulting from this effect resulted in severe strength demands at the root of the complete joint penetration welds between the beam flanges and column flanges, a region that often includes significant discontinuities and slag inclusions, which are ready crack initiators.

- In order that the welding of the beam flanges to the column flanges be continuous across the thickness of the beam web, this detail incorporates weld access holes in the beam web, at the beam flanges. Depending on their geometry, severe strain concentrations can occur in the beam flange at the toe of these weld access holes. These strain concentrations can result in low-cycle fatigue and the initiation of ductile tearing of the beam flanges after only a few cycles of moderate plastic deformation. Under large plastic flexural demands, these ductile tears can quickly become unstable and propagate across the beam flange.

- Steel material at the center of the beam-flange-to-column-flange joint is restrained from movement, particularly in connections of heavy sections with thick column flanges. This condition of restraint inhibits the development of yielding at this location, resulting in locally high stresses on the welded joint, which exacerbates the tendency to initiate fractures at defects in the welded joints.

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

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

- In the mid-1960s, the construction industry moved to the use of the semi-automatic, self-shielded, flux-cored arc welding process (FCAW-S) for making the joints of these connections. The welding consumables that building erectors most commonly used

inherently produced welds with very low toughness. The toughness of this material could be further compromised by excessive deposition rates, which unfortunately were commonly employed by welders. As a result, brittle fractures could initiate in welds with large defects, at stresses approximating the yield strength of the beam steel, precluding the development of ductile behavior.

- Early steel moment frames tended to be highly redundant and nearly every beam-column joint was constructed to behave as part of the lateral-force-resisting system. As a result, member sizes in these early frames were small and much of the early acceptance testing of this typical detail were conducted with specimens constructed of small framing members. As the cost of construction labor increased, the industry found that it was more economical to construct steel moment-frame buildings by moment-connecting a relatively small percentage of the beams and columns and by using larger members for these few moment-connected elements. The amount of strain demand placed on the connection elements of a steel moment frame is related to the span-to-depth ratio of the member. Therefore, as member sizes increased, strain demands on the welded connections also increased, making the connections more susceptible to brittle behavior.

- In the 1960s and 1970s, when much of the initial research on steel moment-frame construction was performed, beams were commonly fabricated using A36 material. In the 1980s, many steel mills adopted more modern production processes, including the use of scrap-based production. Steels produced by these more modern processes tended to include micro-alloying elements that increased the strength of the materials so that despite the common specification of A36 material for beams, many beams actually had yield strengths that approximated or exceeded that required for grade 50 material. As a result of this increase in base metal yield strength, the weld metal in the beam-flange-to-column-flange joints became under-matched, potentially contributing to its vulnerability.

At this time, it is clear that in order to obtain reliable ductile behavior of steel moment-frame construction a number of changes to past practices in design, materials, fabrication, erection and quality assurance are necessary. The recommended criteria contained in this document, and the companion publications, are based on an extensive program of research into materials, welding technology, inspection methods, frame system behavior, and laboratory and analytical investigations of different connection details. The recommended criteria presented herein are believed to be capable of addressing the vulnerabilities identified above and providing for frames capable of more reliable performance in response to earthquake ground shaking.

Although many of the above conditions developed incrementally, over a period of twenty or more years, most steel moment-frame buildings constructed during the period 1960-1994 employed connections of a type that is subject to these vulnerabilities. Therefore, all steel moment-frame buildings constructed during this period should be considered vulnerable to brittle, earthquake-induced, connection fractures, unless specific evidence is available that indicates these vulnerabilities are not present.

Commentary: The typical moment connection detail employed in most welded steel moment-frames constructed during the period 1960-1994 is that shown in Figure 1-1. Although the properties of structural steels and weld metals employed in fabricating and constructing these connections varied somewhat over the years, the basic configuration was almost universally applied in this construction type during this time period. It is now known that almost all such connections can be subject to fracture at levels of inelastic demand that are significantly below those currently believed to be appropriate. Therefore, following strong earthquake ground shaking, unless suitable evidence is available to indicate that a building does not have vulnerable connections, or if evaluations conducted in accordance with these recommendations indicate that significant damage is unlikely to have occurred, all steel moment-frame buildings constructed during the period 1960-1994 should be considered to be potentially damaged. Suitable evidence that a building does not have vulnerable connections could include original construction documents that portray connection details that are substantially different from those indicated in Figure 1-1.

1.4   Application

These *Recommended Criteria* supersede the postearthquake evaluation and repair guidelines for existing steel moment-frame 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)*. This document has been prepared in coordination with *FEMA-302 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, the 1997 *AISC Seismic Specification* (AISC, 1997) and the 1998 *AWS D1.1 Structural Welding Code – Steel* (AWS, 1998). Users are cautioned to consider carefully any differences between the aforementioned documents and those actually enforced by the building department having jurisdiction for a specific project and to adjust the recommendations contained here accordingly.

1.5   Postearthquake Evaluation and Repair Process

Postearthquake evaluation of a welded steel moment-frame is a multi-step process (Figure 1-6). The intent is to identify buildings that have sustained sufficient structural damage to compromise future performance, determine the extent and severity of this damage, assess the general implications of the damage with regard to building safety and determine appropriate actions regarding building occupancy and repair. Once a determination is made that a building has sustained significant damage the structural engineer should conduct a more detailed evaluation of the structure's residual structural integrity and safety and develop a detailed plan for repair, upgrade, demolition, or other action, as appropriate.

Currently, most building codes only require repair of damaged structures, not upgrade. As such, the focus of this document is the identification and repair of damage. However, the extent, severity or characteristics of damage may be sufficiently severe that the owner may wish to consider upgrading or modifying the structure to improve probable performance in future events. Such action may be particularly appropriate when a building has sustained severe damage as a
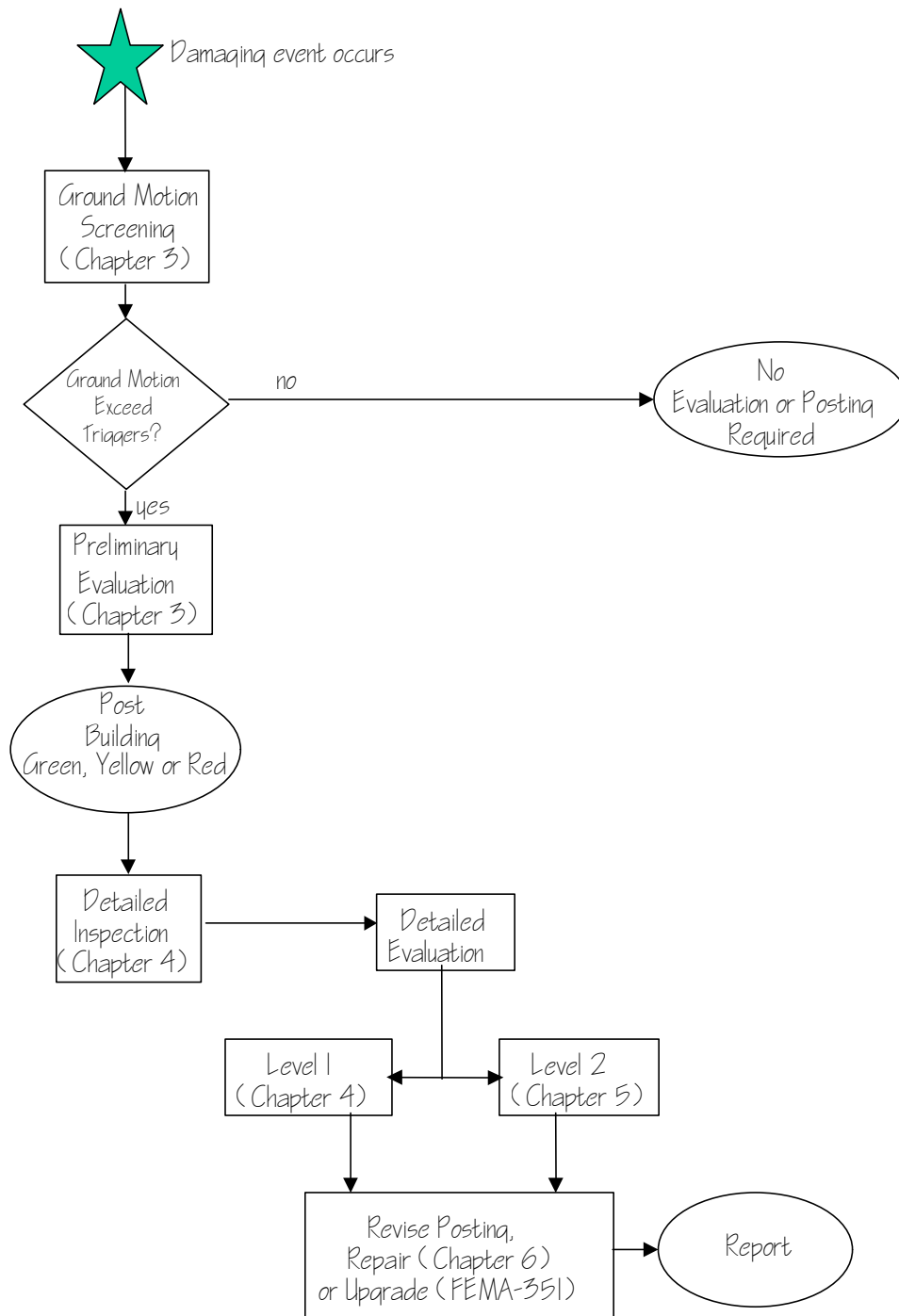
Figure 1-6   Flow Chart for Postearthquake Actions

result of moderate ground shaking, as this may indicate an inability to reliably resist failure in stronger events. Prediction of structural performance during future earthquakes and selection of appropriate upgrades to achieve desired performance is the subject of a companion document,

FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Welded Steel Moment-Frame Buildings.

The first step in the evaluation process is a screening to identify those buildings unlikely to have experienced ground motion of sufficient intensity to cause significant damage. Since strong ground motion instruments are installed in relatively few buildings, it is typically necessary to estimate the regional distribution of ground motion intensity using available instrumental recordings and observed patterns of damage. Those buildings suspected of having experienced ground motion of sufficient intensity to cause damage should be subjected to a rapid on-site evaluation, to determine if there are obvious indications of potentially life threatening conditions.

Following this rapid evaluation, the building should be posted, to indicate whether such conditions were found. Criteria for performing the initial screening and rapid on-site evaluations are presented in Chapter 3.

Often, damage to steel moment-frame buildings cannot be detected by rapid evaluations like those presented in Chapter 3. Therefore, buildings suspected of having experienced potentially damaging ground motion should also be subjected to more detailed inspections and evaluation. Chapter 4 outlines a simplified method for such evaluations, similar to that contained in *FEMA-267*. Chapter 5 presents an alternative, more rigorous procedure consistent with that used for structural performance assessments in other documents prepared by the FEMA/SAC project. Both of these procedures contain recommendations for inspection of some or all steel moment-frame connections in the building; classification of the damage found (in accordance with a system presented in Chapter 2); assessment of the safety of the building, and development of recommendations for repair or other remedial action. Methods of conducting repair and criteria for specifying these methods are presented in Chapter 6. These recommendations do not cover routine correction of non-conforming conditions resulting from deficiencies in the original construction. Industry standard practices are acceptable for such repairs. Recommended criteria for the assessment of seismic performance of the repaired building and recommendations for improved performance may be found in the companion publication, *FEMA-351*.

1.6   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: Inspection and Classification of Damage**. This chapter provides an overview of the different types of structural damage that may be anticipated to occur in welded steel moment-frame buildings, together with a discussion of their significance. This chapter also introduces a damage classification system that is referenced throughout the remaining chapters.

- **Chapter 3: Preliminary Postearthquake Assessment**. This chapter provides screening criteria that can be used to determine if there is sufficient likelihood that a welded steel moment-resisting frame structure has experienced significant damage to warrant further investigation. This Chapter also provides a preliminary evaluation procedure that may be

rapidly performed to determine if the building presents imminent safety hazards. Building officials may use the screening criteria to determine which buildings should be subjected to inspections by the Building Department using the Preliminary Evaluation Procedures. While these preliminary evaluation procedures should permit the identification of structures with damage so severe that imminent hazards have been created, they will typically not be sufficient to determine if more moderate levels of damage have occurred. Chapters 4 and 5 provide procedures for more detailed evaluations, necessary to make such determination.

- **Chapter 4: Level 1 Detailed Postearthquake Evaluations**. Except for those structures that have experienced partial or total collapse, or that exhibit significant permanent interstory drift, the results of a preliminary evaluation conducted in accordance with Chapter 3 are likely to be inconclusive with regard to the postearthquake condition of the structure. This chapter provides procedures for conducting more detailed evaluations of the building to confirm its postearthquake condition and develop recommendations for occupancy and repair of the structure as appropriate. It includes performing inspections of the fracture-susceptible connections in the structure, to determine their condition, and calculation of a damage index. Recommendations for occupancy restriction and repair are provided, based on the calculated value of the damage index. This level of evaluation is too lengthy to be conducted as part of the rapid postearthquake assessments typically conducted by building departments and is anticipated to be implemented by engineers engaged by the building owner.

- **Chapter 5: Level 2 Detailed Postearthquake Evaluations**. If a building has experienced many connection fractures, and other types of structural damage, as revealed by a level 1, detailed evaluation, then it may be advisable to restrict occupancy of the building until it can be repaired. Decisions to restrict occupancy can result in a large economic burden, both for the building owner and the tenants and some engineers may be reluctant to advise such action unless analytical evaluation indicates the presence of significant safety hazards. This chapter provides an analytical methodology for estimating the probability of earthquake-induced collapse of a damaged building that can be used to supplement occupancy decisions suggested by the evaluation procedures of Chapter 4.

- **Chapter 6: Postearthquake Repair**. This chapter provides recommendations for repair of the most common types of damage encountered in welded steel moment-frame construction. It does not include guidelines for structural upgrade. Often, the most logical time to conduct a structural upgrade is during the time that earthquake damage is being repaired. In addition, some jurisdictions require upgrade of buildings that have sustained extensive damage as a matter of policy. Criteria for performing structural upgrade may be found in a companion publication, *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*.

- **Appendix A: Detailed Procedures for Performance Evaluation.** This appendix describes in detail the basis of the reliability-based evaluation methods presented in Chapter 5. It may be used to obtain more certain estimates of structural capacity and must be used for that purpose, instead of the procedures of Chapter 5, for irregular structures.

- **Appendix B: Sample Placards.** This appendix contains sample placards that may be used to post buildings following preliminary postearthquake evaluations conducted in accordance with Chapter 3 (from ATC, 1995).

- **Appendix C: Sample Inspection Forms.** This appendix contains a series of forms that may be used to record damage detected in beam-column connections as part of a detailed postearthquake inspection program conducted in accordance with Chapter 4.

- **References, Bibliography, and Acronyms.**

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2. INSPECTION AND CLASSIFICATION OF DAMAGE

2.1   Introduction

This chapter defines a uniform system for classification and reporting of damage to steel moment-frame structures that have been subjected to strong earthquake ground shaking.

Structural damage observed in steel moment-frame buildings following strong ground shaking can include yielding, buckling and fracturing of the steel framing elements (beams and columns) and their connections, as well as permanent lateral drift. Damaged elements can include girders, columns, column panel zones (including girder flange continuity plates and column web doubler plates), the welds of the beam to column flanges, the shear tabs which connect the girder webs to column flanges, column splices and base plates. Figure 2-1 illustrates the location of these elements.
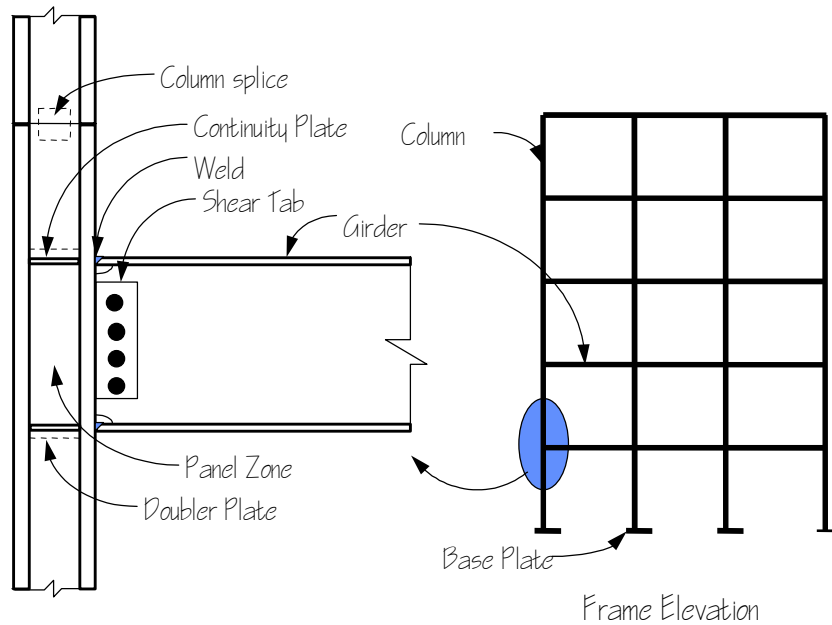
Figure 2-1   Elements of Welded Steel Moment Frame

2.2   Damage Types

Damage to framing elements of steel moment-frame buildings may be categorized as belonging to the weld (W), girder (G), column (C), panel zone (P) or shear tab (S) categories. This section defines a uniform system for classification and reporting of damage to elements of steel moment-frame structures that is utilized throughout these *Recommended Criteria*. The damage types indicated below are not mutually exclusive. A given girder-column connection, for example, may exhibit several different types of damage. In addition to the individual element damage types, a damaged steel moment-frame may also exhibit global effects, such as permanent interstory drifts.

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Following a detailed postearthquake inspection, classification of the damage found, as to its type and degree of severity, is the first step in performing an assessment of the condition and safety of a damaged steel moment-frame structure. In a level 1 evaluation, conducted in accordance with Chapter 4 of these *Recommended Criteria,* the classifications of this section are used for the assignment of damage indices. These damage indices are statistically combined and extrapolated to provide an indication of the severity of damage to a structure's lateral force resisting system and are used as a basis for selecting building repair strategies. For a level 2 evaluation, conducted in accordance with Chapter 5 of these *Recommended Criteria*, these damage classifications are keyed to specific modeling recommendations for analysis of damaged buildings to determine their response to likely ground shaking in the immediate postearthquake period. Chapter 6 addresses specific techniques and design criteria recommended for the repair and modification of the different types of damage, keyed to these same damage classifications.

Commentary: The damage types contained in this chapter are based on a system first defined in a statistical study of damage reported in NISTR-5625 (Youssef et al., 1995). The original classes contained in that study have been expanded somewhat to include some conditions not previously identified.

2.2.1   Girder Damage

Girder damage may consist of yielding, buckling or fracturing of the flanges of girders at or near the girder-column connection. Seven separate types are defined in Table 2-1. Figure 2-2 illustrates these various types of damage. See Section 2.2.3 and 2.2.4 for damage to adjacent welds and shear tabs, respectively.

Commentary: 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. Removal of finishes, by scraping, may often obscure the detection of this type of damage. Girder flange yielding, without local buckling or fracture, results in negligible degradation of structural strength and typically need not be repaired.

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 (top or bottom)
G4	Flange fracture outside Heat Affected Zone (top or bottom)
G5	Not used
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsion buckling of section

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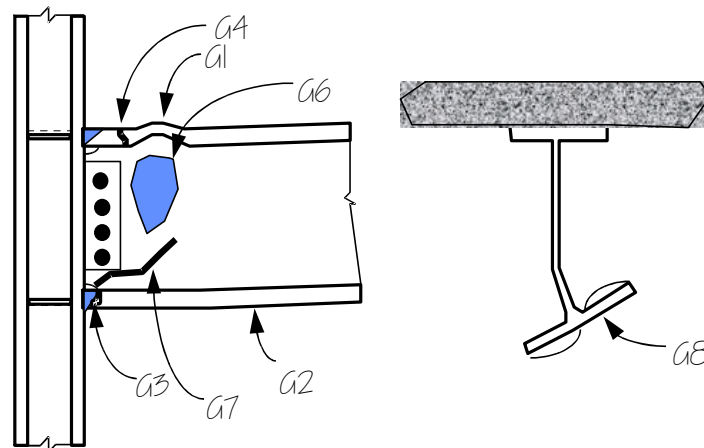
Figure 2-2   Types of Girder Damage

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength, particularly when accompanied by girder web buckling (type G6). 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, compared to 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. Visually evident girder flange buckling should be repaired.

In structures with weld material with low notch-toughnesss, girder flange cracking within the Heat Affected Zone (HAZ) (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 HAZ. However, these fractures can also occur in connections with welded joints made with notch-tough weld metal and following appropriate procedures, as a result of low-cycle fatigue, exacerbated by the very high strain demands 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 being less restrained can experience buckling relatively easily.
Finally, much of the damage found in girders initiates as a result of defects at the
root of the beam flange to column flange weld. Due to its position, the weld of the
bottom beam flange to column flange is more difficult to make than that at the top
flange, and therefore, is more likely to have defects that can initiate such damage.*

2.2.2   Column Flange Damage

Seven types of column flange damage are defined in Table 2-2 and illustrated in Figure 2-3.
Column flange damage typically results in degradation of a structure's gravity-load-carrying
strength as well as lateral-load resistance. For related damage to column panel zones, refer to
Section 2.2.5.

Table 2-2   Types of Column Damage

Type	Description
C1	Minor column flange surface 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
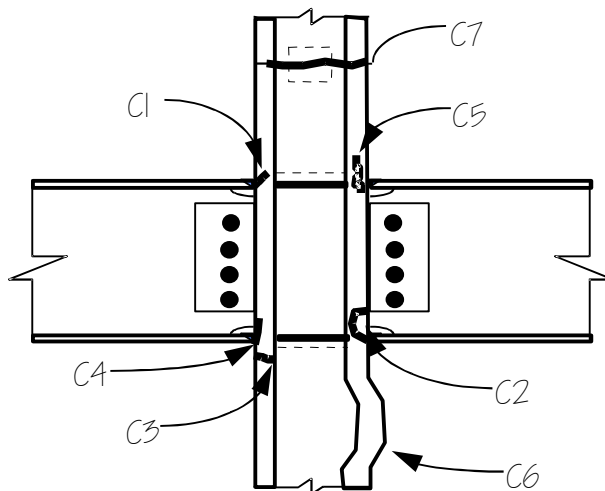
Figure 2-3   Types of Column Damage

*Commentary: Column flange damage includes types C1 through C7. Type C1
damage consists of a small crack at the surface of the column flange and
extending into its thickness, typically at the location of the adjoining girder*

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flange. C1 damage does not go through the thickness of the column flange and can often be detected only by nondestructive testing (NDT). 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 into the column flange. In some cases this 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 may result in both a loss of effective attachment of the girder flange to the column for tensile demands and could cause 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 damage may occur as a result of non-metallic inclusions within the column flange. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands, such as the residual stresses induced by welding, has been known for a number of years, and is termed lamellar tearing. There is no evidence that lamellar tearing actually occurred in buildings as a result of earthquake ground shaking and it is currently thought that when type C5 damage did occur, it was an extension of fracturing that initiated in the weld root. This damage has sometimes been identified as a potential contributing mechanism for type C2 column flange through-thickness failures. Note that in many cases, type C2 damage may be practically indistinguishable from type W3 fractures (see Section 2.2.3). The primary difference is that in type W3, the fracture surface generally remains with in the heat affected zone of the column flange material while in C2 damage, the fracture surface progresses deeper into the column flange material.

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,

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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 very large interstory drifts have occurred.

Type C7 damage, fracturing of welded column splices, also was not observed following the Northridge earthquake. However, the partial joint penetration groove welds commonly used in these splices are very 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.

As a result of the potential safety consequences of complete column failure, all column damage should be considered as significant and repaired expeditiously.

2.2.3 Weld Damage

Three types of weld damage are defined in Table 2-3 and illustrated in Figure 2-4. All apply to the complete joint penetration welds between the girder flanges and the column flanges.

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

Type	Description
W1, W1a, W1b	Not Used (see commentary)
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	Not Used (see commentary)
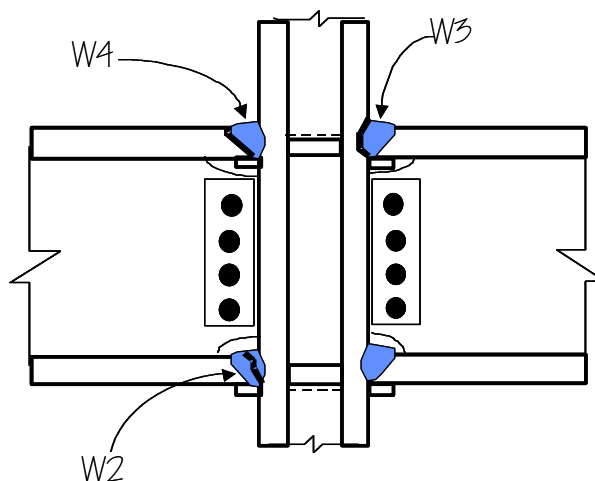
Figure 2-4   Types of Weld Damage

Commentary: In addition to the W2, W3, and W4 types of damage indicated in Table 2-3 and Figure 2-4, the damage classification system presented in FEMA-267 included conditions at the root of the complete joint penetration weld that did

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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 (NDT). 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 such conditions could be reported in the event they were detected in the course of the ultrasonic testing (UT) that FEMA-267 required. There was no requirement to repair such conditions. Since these Recommended Criteria do not require UT as a routine part of the inspection protocol, W5 conditions are unlikely to be detected and have been omitted as a damage classification.

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 conditions are pre-existing defects, rather than earthquake damage. This research also demonstrated that W1 conditions are difficult to detect reliably unless the weld backing is removed. In a number of case studies, it has been demonstrated that when W1 conditions are indicated by UT, they are often found not to exist when weld backing is removed. Similarly, in other cases, upon removal of backing, W1 conditions were found to exist where none had been detected by UT. For these reasons, in the development of these recommendations, it has been decided to de-classify W1 conditions as damage and to eliminate the need for routine use of UT in the performance of detailed connection inspections.

Notwithstanding the above, it is important to recognize that a very significant amount of the "damage" reported following the Northridge earthquake was type W1 conditions. Studies of 209 buildings in the city of Los Angeles have shown that approximately 2/3 of all reported "damage" conditions were type W1. Although these Recommended Criteria do not classify W1 conditions as damage, their presence in a connection can lead to a significant increase in the vulnerability of the building to earthquake induced connection fracture. If, in the performance of connection inspections or repairs it is determined that rejectable

discontinuities, lack of fusion, slag inclusions or cracks exist at the root of a weld, they should be reported and consideration should be given to their repair, as a correction of an undesirable, pre-existing condition.

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 and should be repaired.

2.2.4 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 2-5. Severe damage to shear tabs is unlikely to occur unless other damage has also occurred to the connection, i.e., column, girder, panel zone, or weld damage, as previously defined.

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
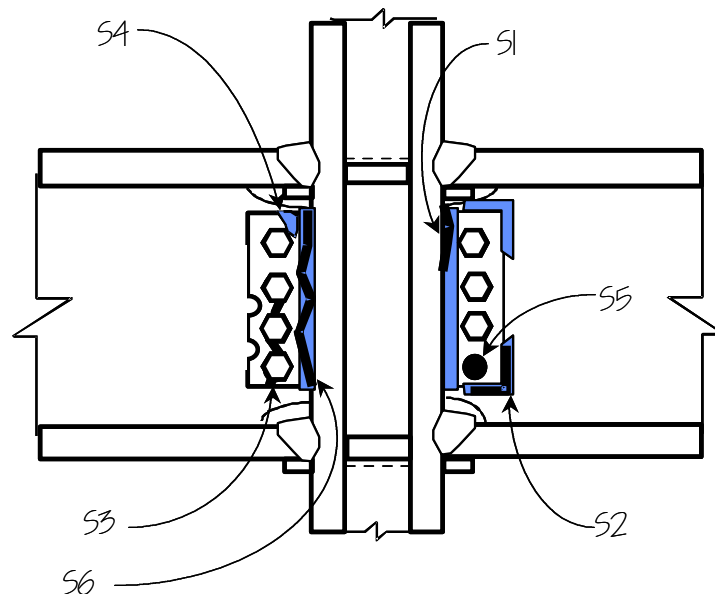
Figure 2-5   Types of Shear Tab Damage

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Commentary: 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, then failure of the shear tab is likely to trigger distress in the welded joints of the girder flanges.

2.2.5 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 2-6. 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. In addition, the difficult access to the column panel zone and the difficulty of removing sections of the column for repair, without jeopardizing gravity load support, make this damage among the most costly to repair.

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
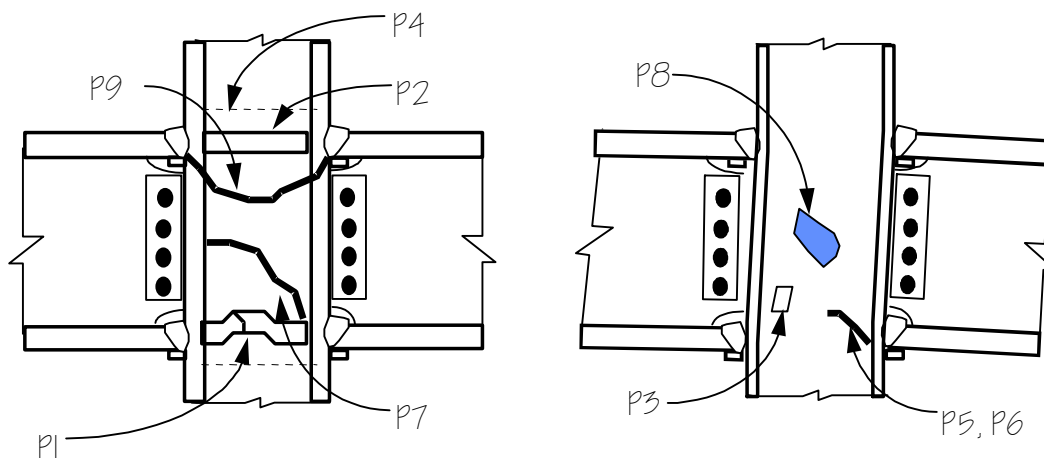
Figure 2-6   Types of Panel Zone Damage

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Commentary: 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.

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.

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.

2.2.6 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 that may experience damage include: (1) column base plates, beams, columns, and their connections that were not considered 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 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. When such damage is discovered in a building, it should be reported and repaired, as suggested by later sections of these *Recommended Criteria*.

3. PRELIMINARY POSTEARTHQUAKE ASSESSMENT

3.1   Introduction

3.1.1   General

Following a potentially damaging earthquake, an assessment should be performed for each steel moment-frame building to determine the likelihood of significant structural damage, the implications of this damage with regard to building safety and occupancy and the need for repair. A three-step process is recommended. These steps include:

Screening. In this step, an estimate is made of the probable ground motion experienced at the building site. If this estimated ground motion falls below certain trigger values, further evaluation is not required. Section 3.2 provides recommended criteria for screening.

Preliminary Evaluation. In this step, a site visit is made to the building and the condition of the building is observed to determine if there are obvious indications of structural or nonstructural damage that pose a potential risk to life safety. The building is typically posted with a placard, based on the findings of this evaluation. Section 3.3 provides recommended criteria for preliminary evaluation.

Detailed Evaluation. In this step, detailed inspections of building framing and connections are performed to determine the condition of the structure. If structural damage is detected in the course of these inspections, further evaluations are performed to determine the significance of this damage and the appropriate repair and occupancy actions. Revision of the posting status of the building may be appropriate following such evaluation. Chapters 4 and 5 provide procedures for detailed evaluation.

As indicated in Section 1.3 and Chapter 2, following the 1994 Northridge and other recent earthquakes, structural damage was detected in many steel moment-frame buildings that had little outward signs of structural distress. Detailed postearthquake evaluations are necessary to find such damage, but involve rigorous inspection of structural condition. These more detailed evaluations can be quite costly and may be unnecessary for buildings that have not sustained significant structural damage. The initial screening process presented in this chapter is intended to provide rapid identification of those buildings that likely did not experience sufficient ground shaking to cause significant damage and which therefore need not be subjected to further evaluations. The preliminary evaluation procedures of this chapter are intended to identify those buildings that present obvious signs of severe damage so that immediate restrictions on occupancy may be placed. Following preliminary evaluation, a report of pertinent findings should be made to the Owner. If the evaluation was ordered by the Building Official, these findings should also be reported to the Building Department and the building should be posted with an appropriate placard in accordance with Section 3.3.

> *Commentary: Screening is intended to identify those buildings that experienced sufficient ground shaking that they may have sustained significant damage. If a building is not identified as likely to have experienced such ground shaking, no*

evaluation need be performed. However, if a building is identified as having experienced such ground motion both a preliminary and detailed evaluation should be performed. The preliminary evaluation is intended to provide a rapid basis for making recommendations regarding immediate postearthquake occupancy. Detailed evaluations, in accordance with Chapters 4 and 5 are used to confirm the extent and severity of any damage present and to serve as the basis for repair programs, should these be necessary.

The procedures contained in this chapter and in Chapters 4 and 5 are specifically intended to identify if earthquake ground shaking has damaged a building and thereby, impaired its safety. These procedures are not intended to determine if a building had adequate structural characteristics prior to the onset of damage or how the structure may perform in future earthquakes. Some owners may wish to assess the likely performance of their building when subjected to a future earthquake, irrespective of any damage that has occurred in the present event. Readers are referred to the companion publication, FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings for performance evaluation and upgrade recommendations for such structures. It is recommended that such performance evaluations be performed when a building has sustained substantial damage as a result of ground shaking that is significantly less intense than the shaking specified by the current building code for design of a new structure at that site.

3.1.2 Evaluator Qualifications

Postearthquake evaluations entail the observation of different conditions within a building, making judgments as to whether they are indicative of structural damage and the likely effect of such damage with regard to the ability of the structure to withstand additional loading. This requires the application of considerable structural engineering knowledge and judgment. In order to perform these tasks properly, the evaluator should possess at least the same levels of knowledge, experience and training necessary to act as the design professional of record for the structure, and in some cases, more detailed knowledge, experience and training may be necessary. Persons possessing such knowledge, experience and training are referred to in these *Recommended Criteria* as the structural engineer. References to the structural engineer throughout these *Recommended Criteria* indicate that the work is to be performed either directly by persons possessing these qualifications, or by persons acting under the direct supervision of such a person.

3.2 Screening

Prior to performing preliminary or detailed postearthquake evaluations, it is recommended that screening be performed to determine if a building has likely experienced ground shaking of sufficient intensity to cause significant damage. Buildings need to be subjected to evaluations only if any of the following apply:

- estimated ground-motion acceleration or intensity (MMI) at the site exceeds the limits indicated in Table 3-1;

- significant structural damage is observed in one or more steel moment-frame structures located within 1 kilometer of the building on sites with similar or more firm soil profiles;

- significant structural damage is observed to one or more modern, apparently well-designed structures (of any structural system) within 1 kilometer of the building and on sites with similar or more firm soil profiles;

- damage to the general building stock within 1 kilometer of the building and on sites with similar or more firm soil profiles corresponds with the categories indicated in Table 3-1;

- for an earthquake having a magnitude of 6.5 or greater, the structure is either within 5 kilometers of the trace of a surface rupture or within 5 kilometers of the ruptured area of the fault plane when no surface rupture has occurred;

- significant architectural or structural damage is observed in the building; or

- entry to the building has been limited by the building official because of earthquake damage, regardless of the type or nature of the damage.

Table 3-1   Ground Motion Indicators of Potential Damage

1997 NEHRP MCE Map* Short-Period Contour Area	Estimated Peak Ground Acceleration	Level of Damage to Buildings Within 1 Kilometer	Estimated Modified Mercali Intensity, MMI
$S_S \geq 0.50$	$\geq 0.25g$	Prevalent partial collapse of unreinforced masonry buildings. High levels of nonstructural damage. Considerable damage to ordinary buildings.	VIII
$0 < S_S < 0.50$	$\geq 0.15g$	Considerable damage to unreinforced masonry buildings. Slight damage to well-designed buildings. Prevalent nonstructural damage.	VII

* FEMA-302, ASCE (1998) and *IBC* (ICC, 2000) maps.

If none of the above conditions apply to a building, it may be classed as unlikely to have experienced significant damage and need not be subjected to evaluation.

Commentary: Preliminary screening is typically performed by the Building Official in order to identify those areas of a community in which post earthquake evaluations should be performed. The screening criteria presented in this section can typically be applied on a regional basis, after preliminary reconnaissance has been performed to determine the general patterns and distribution of damage that has occurred in the affected region. Building departments will typically perform such surveys in the hours immediately following an earthquake, in coordination with emergency response agencies in order to coordinate emergency response

activities. The data obtained from such surveys can be used to develop preliminary isoseismal maps (maps with contours indicating probable intensities of ground shaking). These isoseismal maps can then be used to identify geographic areas, within which evaluations should be performed or ordered.

Typically, initial determination of the distribution of ground motion intensity from an earthquake and the geographic areas in which building evaluations should be performed will be subject to revision, over time, as more detailed data becomes available. A number of techniques and sources of information are available for developing these more accurate estimates of ground motion intensity. Frequently, the United States Geologic Survey (USGS) or other government agencies will develop maps of ground motion intensity, shortly after an earthquake occurs. In regions with a large number of strong-motion accelerographs present, actual ground motion recordings provide the best method of mapping contours of ground motion. These should be used if located near the building, and are located on sites having similar characteristics.

In other regions, empirical techniques, such as the use of standard ground-motion attenuation relationships (e.g., Boore and Joyner, 1994; Campbell and Bozorgnia, 1994) may be required. These can be supplemented with analytically derived estimates such as those obtained by direct simulation of the fault rupture and ground wave propagation. It may be desirable to retain a qualified geotechnical engineer or earth science consultant to make these estimates. It should be noted, however, that lacking direct instrumental evidence, site-specific ground motion estimates are, at best, uncertain and subject to wide variations depending on the assumptions made. Therefore, the best indicator of the severity of ground motion at a site is often the performance of adjacent construction. The criteria of Table 3-1 are provided to help assure that sites that experienced relatively strong ground motion are not overlooked as a result of inaccurate estimates of the ground motion severity.

3.3 Preliminary Evaluation

3.3.1 General

The objective of preliminary evaluation is to determine, on a rapid, preliminary basis, whether a building has sustained either structural or nonstructural damage that results in a hazardous condition. Preliminary evaluation includes:

- a general review of the building's construction characteristics to determine its structural system and vulnerable features (Section 3.3.2),

- a visit to the building site to observe its overall condition and note obvious signs of damage (Section 3.3.3),

- a determination of an appropriate posting category for the building, on the basis of the preceding results and engineering judgment (Section 3.3.4).

The condition ratings presented in Table 3-2 are recommended as posting categories. Section 3.3.4 provides recommended criteria for assignment of a building to the various posting categories.

Table 3-2   Postearthquake Condition Designations

Condition		Finding	Description
G R E E N	1	Inspected	The building does not appear to have experienced significant damage either to structural or nonstructural components. Occupancy may continue, pending completion of detailed evaluations.
	2	Minor nonstructural damage	The building does not appear to have experienced significant damage to structural elements, but has experienced some damage to nonstructural components. Occupancy may continue, pending completion of detailed evaluations. Repair of nonstructural damage may be conducted at convenience.
	3	Minor damage	The building appears to have sustained limited damage to structural and nonstructural elements. Occupancy may continue, pending completion of detailed evaluations. Repair of damage may be conducted at convenience.
Y E L L O W	1	Damaged – nonstructural	The building does not appear to have experienced significant damage to structural elements; however, it has sustained damage to nonstructural components that pose a limited safety hazard. Occupancy of the building in areas subject to these hazards should be limited until repairs are instituted. Occupancy of other portions of the building may continue, pending completion of detailed evaluations.
	2	Damaged – structural	The building appears to have experienced significant damage to structural elements. Although it does not appear that the building is an imminent collapse risk, localized safety hazards may exist. Occupancy of the building in areas subject to these hazards should be limited until repairs or stabilization can be implemented, or a more reliable assessment of the building's condition can be made to demonstrate that hazards do not exist. Occupancy of other portions of the building may continue, pending completion of detailed evaluations.
R E D	1	Unsafe – repairable	The building appears to have sustained significant damage to structural elements that has substantially impaired its ability to resist additional loading or to nonstructural elements that pose a significant hazard to occupants. It should not be occupied until repair or stabilization work has been performed or a more detailed evaluation of its condition can be made to demonstrate that hazards do not exist.
	2	Unsafe	The building appears to have sustained significant damage to structural elements, substantially impairing its ability to resist additional loading. It appears to be a potential collapse hazard and should not be occupied.

Commentary: The condition assessment categories indicated in Table 3-1 should be assigned on the basis of the preliminary(rapid) evaluation. However, the assignment should be subject to change on the basis of detailed evaluations conducted in accordance with Chapters 4 and 5.

It is not uncommon during the postearthquake evaluation process to discover that although a building has relatively little damage, it has severe structural deficiencies relative to current building code requirements and may as a result be

*structurally unsafe. The condition assessments indicated in Table 3-2 are
intended to be applied only to those conditions resulting from earthquake damage
and should not be used to rate a building that is otherwise structurally deficient.
However, when such deficiencies are identified in a building during the course of
a postearthquake evaluation, the engineer should notify the Owner and Building
Official of these conditions.*

3.3.2 Building Construction Characteristics

In order to make a meaningful assessment of a building's postearthquake condition it is
necessary to develop an understanding of its structural system and basic details of the building's
construction and to clearly establish the seismic load path. Whenever the structural and
architectural drawings for the building are available, they should be reviewed as part of the
preliminary evaluation. The review should include the following:

- confirmation that the building is a steel moment-frame structure,

- determination of the year of design and construction and code used as a basis; this may
  provide information on particular vulnerabilities, such as the presence of weak stories, or use
  of particular weld metals,

- identification of materials and typical details of connections and elements for areas of
  particular vulnerability,

- identification of the location of steel moment frames,

- identification of locations of moment-resisting beam-column connections and column
  splices, to identify locations where potentially vulnerable conditions exist,

- identification of any structural irregularities in the vertical and horizontal load resisting
  systems, that could lead to potential concentrations of damage, and

- identification of architectural elements that could affect the behavior of the structural system
  or elements, or that may themselves be vulnerable to damage and be a threat to occupants,
  including, for example, precast concrete cladding systems and interior shaft walls.

3.3.3 Preliminary Site Inspection

Every steel moment-frame building situated on a site that has experienced strong ground
shaking, as identified in accordance with the screening criteria of Section 3.2, should be
subjected to a rapid postearthquake inspection to ascertain whether there is apparent damage and
to determine the apparent severity of such damage. When performing the inspection, the
structural engineer should attempt to determine if strong motion accelerometers are present in the
building. If so, the record should be accessed and reviewed for noticeable changes in behavior
during the building response that may be indicative of significant structural damage.

Preliminary site inspections should include the following:

1. Visual observation of the building exterior. Check for:

☑ obvious indications of large permanent interstory drift,

☑ indications of foundation settlement or distress as evidenced by sags in horizontal building fenestration or distress in base level slabs,

☑ loosened or damaged cladding or glazing systems,

☑ indications of discrete areas of the building where interstory drift demands may have concentrated as evidenced by apparent concentrations of architectural damage to fascia and cladding systems,

☑ pounding against adjacent buildings or portions of the building separated by expansion joints, and

☑ potential site instabilities such as landslides or lateral spreading that may have resulted in damage to the building foundations or structure.

2. Visual observation of the building interior. Check for:

☑ damage to nonstructural components, such as suspended ceilings, light fixtures, ducting, and masonry partitions, that could result in potential hazards,

☑ damage to floor slabs around columns, to finishes, and to partitions, that may suggest damage to adjacent beams and connections,

☑ indications of discrete areas of the building where interstory drift demands may have concentrated as evidenced by apparent concentrations of damage to architectural elements including interior partitions,

☑ damage to interior finishes on structural elements, such as columns, that could be indicative of damage to the underlying structure,

☑ damage to equipment or containers containing potentially hazardous substances, and

☑ damage to elevator counterweight and rail systems.

3. Evaluation of the building for permanent interstory drift.

Preliminary evaluation of the building for permanent interstory drift should be performed. This can be done by dropping a plumb bob through the elevator shaft and determining any offset between threshold plates in adjoining levels of the building. Multiple levels should be checked simultaneously, to minimize the effect of minor offsets resulting from within-tolerance variations in the original construction.

4. Perform preliminary visual inspection of selected moment frames for indications of damage. Refer to Sections 3.3.3.1 and 3.3.3.2 for preliminary inspection procedures for moment-resisting connections with and without fireproofing present, respectively.

☑ If visual observation of building exterior or interior indicates a zone or zones of large permanent interstory drift, perform selective removal of architectural finishes to expose framing. Observe for indications of yielding, buckling or other damage to framing, or connections. Exposures and observation should be made of at least one beam-column connection per line of framing per story within the zone or zones of large permanent

interstory drift. For highly redundant structures, with many lines of framing, exposures and observations may be limited to one beam-column connection on one line of framing in each direction of building response, on each side of the structure, with a minimum of two exposures per story.

☑ If visual observation of building exterior or interior indicates zones of concentrated interstory drift demand, perform selective removal of architectural finishes to expose framing. Observe for indications of fracture, yielding or buckling of framing, or damage to connections. Exposures and observation should be made of at least one beam-column connection per line of framing per story within the zone or zones of concentrated interstory drift demand. For highly redundant structures, with many lines of framing, exposures and observations may be limited to one beam-column connection on one line of framing in each direction of building response, on each side of the structure, with a minimum of four exposures per story.

☑ If visual observation of building exterior or interior indicates neither zones of large permanent interstory drift, nor of concentrated interstory drift demand, perform selective removal of architectural finishes to expose framing throughout structure. Observe for indications of yielding or buckling of framing, or damage to connections. Exposures and observation should be made of at least one beam-column connection per line of framing per story. For highly redundant structures, with many lines of framing per story, exposures and observations may be limited to one beam-column connection on one line of framing in each direction of building response, on each side of the structure, with a minimum of two exposures per story.

☑ If visual observation of the building exterior indicates zones of pounding against adjacent structures, expose framing in the area of pounding to identify damage to structural elements and connections.

Commentary: In most steel moment-frame buildings, structural steel will be obscured by fire protective coverings that are frequently difficult to remove. In many cases these coverings will be composed of asbestos-containing materials, if constructed before 1976, and must not be removed by anyone without proper training. Observation conducted as part of preliminary procedures is limited to observing the condition of the steel, if exposed to view, or the condition of the fire protective covering if the steel is not exposed, to observe tell-tale signs of structural damage including cracking or spalling of the covering material, or loosened and broken bolts.

The presence of one or more strong-motion instruments in a building can provide valuable evidence as to the extent of damage a building has experienced. Noticeable lengthening of the building period can be an indication of structural damage. However, even in the absence of instruments within a building, it may be possible to obtain indirect evidence of changes in a building's dynamic properties that are indicative of damage. This could include apparent lengthening of the building period, or increasing nonstructural damage in aftershocks.

3.3.3.1   Preliminary Connection Inspections when Fireproofing is Present

Perform the observations indicated in the checklist below. Figure 3-1 indicates the various zones of observation. Note that fireproofing need not be removed as part of the preliminary inspection, unless indications of potential damage are noted, at which point fireproofing should be removed to allow confirmation of the extent of any damage. If there is reason to believe the fireproofing is an asbestos-containing material, removal should be performed by appropriately trained personnel with proper personnel protection. The engineer should not personally attempt to remove fireproofing suspected of being an asbestos containing material unless he has been trained in the appropriate hazardous materials handling procedures and is wearing appropriate protective equipment.
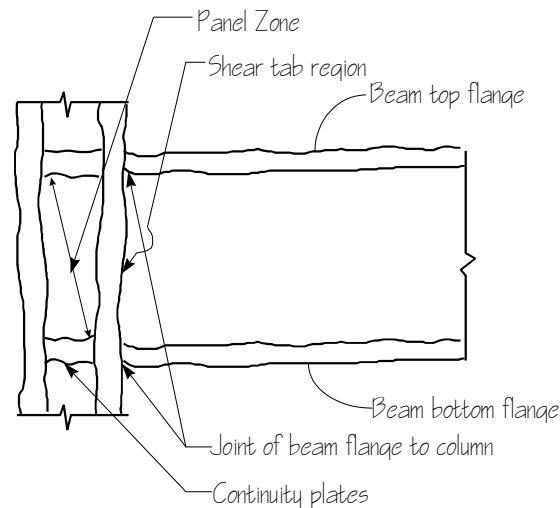
Figure 3-1   Observation Zones for Fire-Proofed Beam-Column Connections

☑ Observe beam framing into connection for trueness to line, and potential indications of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).

☑ Observe condition of fireproofing along beam within one beam depth of the column for cracking or spalling of the fireproofing material along the beam surface, indicating potential yielding or buckling of the beam flanges (damage types G1 and G2, Section 2.2.2).

☑ Observe the top and bottom surface of the bottom flange fireproofing and bottom surface of the top flange fireproofing at the locations where the beam flanges join the column flanges (or continuity plates for minor axis connections) for cracks or losses of material that could indicate cracking at the full penetration weld (damage types G3, Section 2.2.1; C1, C3 and C4, Section 2.2.2; W2, W3, W4, Section 2.2.3).

☑ Observe the condition of the fireproofing at the beam web, in the vicinity of the clip connection from the beam web to the column for loosened, cracked or spalled material indicative of potential damage to shear tabs (damage types S1 through S5, Section 2.2.4).

☑ Observe the condition of the fireproofing at the column panel zone for cracks, loosened or spalled material, indicative of damage to the panel zone or continuity plates (damage types P1 through P8, Section 2.2.5).

☑ Observe the flanges of the column at and beneath the joint with the beam flange for loosened, spalled or cracked material, indicative of fractures, buckled or yielded sections (damage types C1, C3, C4, C6, Section 2.2.6).

☑ Observe the column flange in the area immediately above the bottom beam flange for loosened, spalled or cracked material, indicative of a potential divot type fracture of the column material (damage type C2, Section 2.2.2).

> *Commentary: The presence of fireproofing will tend to obscure many types of damage, unless the damage is very severe. However, removal of fireproofing can be a difficult and time consuming process. For the purposes of preliminary inspection in buildings with fireproofing, inspection is limited to that readily observable with the fire proofing in place. Removal of fireproofing and more careful visual inspection in such buildings is limited to inspections performed as part of detailed evaluations, in accordance with Chapters 4 and 5 of this publication. An exception is the case when observation indicates that the fireproofing has noticeably cracked, spalled or loosened, indicating that damage has probably occurred to the steel framing beneath. In this case, removal of fireproofing is recommended as part of the preliminary inspection to determine the extent of damage.*
>
> *In many buildings constructed prior to 1976, the original fireproofing materials commonly contained friable asbestos fibers. Disturbing such material without wearing suitable breathing apparatus can result in a significant health hazard both to the person performing the work and also to others located in the area. For this reason, owners have been gradually addressing these hazards either by encapsulating such fireproofing, to prevent it from being disturbed, or replacing it with non-hazardous materials. In buildings constructed prior to 1976, the engineer should not permit fireproofing to be removed except by properly trained personnel using appropriate procedures unless the owner can present suitable evidence that the material does not contain friable asbestos.*

3.3.3.2   Bare Structural Steel

Preliminary inspection of framing connections in buildings that do not have fireproofing in place on the structural steel should include the complete joint penetration (CJP) groove welds connecting both top and bottom beam flanges to the column flange, the backing bars and the weld access holes in the beam web; the shear tab connection, including the bolts, supplemental welds and beam web; the column web panel zone, including doubler plates; and continuity plates and continuity plate welds (see Figure 3-2).

The inspection should be by visible means. Observe all exposed surfaces for cracks, buckling, yielding, and loosened or broken bolts. The area inspected should include that portion of the beam within a distance d_b (beam depth) of the face of the column, that portion of the column below the connection and within a distance d_c (column depth) of the bottom beam flange, the panel zone and all bolts and plates within these regions. Sections 2.2.1 through 2.2.6 indicate

the types of damage that may be present. All damage observed should be recorded according to the classification indicated in those sections, and documented in sketch form.
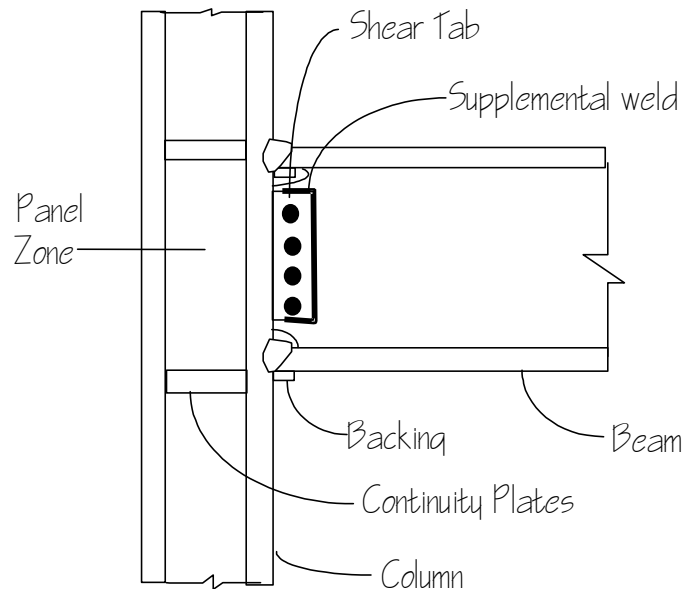
Figure 3-2   Components of Moment Connection

Note that visual inspection should not be performed casually. After a fracture forms in steel framing, it can close up again under further loading of the building. Such "closed" fractures, though obscure, can typically be detected by careful observation, sometimes aided with touch to detect roughness in the surface in the vicinity of a potential fracture. Wetting of the area of a suspected surface crack can also assist in detection. In some cases, it may be necessary to use more formal nondestructive testing methods, such as ultrasonic testing, magnetic particle testing, or liquid dye penetrant testing to confirm the presence of such cracks. Such confirmation can be performed as part of the more detailed inspections undertaken as part of a Level 1 detailed evaluation (Chapter 4) or a Level 2 detailed evaluation (Chapter 5).

Certain types of damage (C2, C3, C5, Section 2.2.2; W2, W3, Section 2.2.3) may be impossible to detect by visual observation alone, as the presence of weld backing at the underside of the beam flange will obscure the presence of the fracture. The presence of a gap between the bottom edge of the backing and the column flange is one indication of the potential presence of such damage. If such a gap is present it may be possible to explore the presence of concealed fractures by inserting a feeler gauge into the gap to determine its depth. If the feeler gauge can be inserted to a depth that exceeds the weld backing thickness, a fracture should be assumed to be present. Nondestructive testing will be required to confirm the extent of such damage, and can be performed as part of the more detailed evaluation. Alternatively, the backing can be removed to allow direct observation of any damage present. However, such removal entails either cutting or grinding operations and can not normally be performed as part of a preliminary evaluation.

3.3.4 Data Reduction and Assessment

Following the collection of data on a building, as outlined in Sections 3.3.1 and 3.3.2, it is necessary to form a preliminary opinion as to whether a building has sustained damage that creates a potential hazard, and the severity and distribution of such hazards, if present. The following sections provide recommendations in this regard. The structural engineer, on the basis of the evaluated data, or personal engineering judgment, may make a more conservative assessment.

3.3.4.1 Finding of Dangerous Condition

An assessment should be made that a building has been extensively damaged and is potentially hazardous, if any of the following conditions are observed:

- permanent interstory drift in any level of 1.0% or greater,

- unexpected severe damage to architectural elements or significant period lengthening of the building is observed in aftershocks,

- visual inspections of steel framing indicate the presence of two or more fractures of the type G7, C3, C6, C7, S3, S4, S5, S6, P6, P7 or P9, at any floor level, or

- the building experiences excessive lateral deformation or unusual amounts of additional architectural damage in moderate aftershocks.

In the event that any of the above conditions is detected, the building should be assessed on a preliminary basis as conforming to damage condition Red-1, of Table 3-2. A detailed evaluation should be recommended and notification should be made advising against continued occupancy until a more detailed determination of structural condition can be completed.

> *Commentary: The observed behavior of a building in repeated aftershocks may provide some clues as to whether it has experienced significant structural damage. In instrumented buildings it may be possible to observe a lengthening of the building period during aftershocks. In buildings without instruments, the observation of unexpected large amounts of architectural damage during aftershocks could indicate the presence of previous structural damage.*

3.3.4.2 Finding of Damaged Condition

If none of the conditions indicated in Section 3.3.4.1 are determined to exist, but one or more of the conditions indicated below are present, an assessment should be made that the building has sustained significant nonstructural damage and should be posted as damage condition Yellow-1 of Table 3-2. Appropriate precautions should be taken to limit access to hazardous areas.

- Connections of exterior fascia panels have been damaged and panels are hanging loosely on the building.

- Exterior glazing is broken above the first story.

- Connections of stair stringers to floor framing has been compromised.

- Ceiling components, including suspension systems, lights, HVAC and utilities have been damaged and are hanging into the occupied spaces or walkways.

- Gas lines are damaged or containers of unidentified or known hazardous materials have toppled and spilled.

- Egress ways are blocked or inoperable.

- Emergency lighting systems are unusable.

- Fire suppression systems required by code are inoperable.

If none of the conditions indicated in Section 3.3.4.1 are determined to exist, but one or more of the conditions indicated below are present, an assessment should be made that the building has sustained significant structural damage and should be posted as damage condition Yellow-2 of Table 3-2. Appropriate precautions should be taken to limit access to hazardous areas.

- Visual inspection of steel framing indicates shear tab damage type S3, S5 or S6 in any beam connection, in accordance with Section 2.2.4.

- Visual inspection of steel framing indicates that a beam has become dislodged from a supporting member or element.

- Visual inspection of steel framing indicates that a column has experienced type P7 damage in accordance with Section 2.2.5, or type C7 damage in accordance with Section 2.2.2.

3.3.4.3   Finding of Undamaged Condition

If none of the conditions indicated in Sections 3.3.4.1 or 3.3.4.2 are determined to exist, it is recommended that the building be assessed Green-1, Green-2, or Green-3 of Table 3-2, as appropriate, pending completion of detailed evaluations in accordance with Chapter 4 or 5.

> *Commentary: The absence of significant observable damage to steel moment-frame structures in a preliminary evaluation on sites believed to have experienced strong ground motion, per Table 3-1, should not be used as an indication that detailed evaluations are not required. Many steel moment-frame buildings that were structurally damaged by the 1994 Northridge and 1989 Loma Prieta earthquakes had little apparent damage based on casual observation.*

3.3.5   Reporting and Notification

Following performance of a preliminary evaluation, notification should be made that an evaluation has been performed and a report should be provided to the Owner. The extent of notification to be made is dependent upon the jurisdiction of the party performing the evaluation, and upon the condition of the building. If the building has been found to be dangerous, the occupants ultimately must be notified (in a timely manner).

3.3.5.1   Building Departments

When preliminary evaluations are performed by or on behalf of the Building Official, or other authority having jurisdiction, the following notifications should be made:

- A placard should be placed at the main entry to the building indicating that a preliminary (rapid) evaluation has been performed, and indicating the assessed condition designation of the building, recommended occupancy restrictions, follow-up actions, and the identity and affiliation of the person performing the evaluation. In large buildings with more than one entrance, additional placards should be placed at all other entrances (ATC, 1989). Appendix B to these *Recommended Criteria* includes sample placards (from ATC, 1995).

- If a building has been posted either as "damaged" (condition Yellow-1 or Yellow-2) or "unsafe" (condition Red-1 or Red-2), additional written notification should be served on the Owner at his/her legal address, indicating the status of the posting, the Owner's rights and any actions required on the Owner's part.

3.3.5.2   Private Consultants

If postearthquake evaluations by private consultants are permitted by the local authority having jurisdiction, the same procedures prescribed in Section 3.3.5.1 should be followed: a placard should be placed at the main entry to the building indicating that preliminary evaluation has been performed, the assessed condition of the building, recommended occupancy restrictions and follow-up actions, and the identity and affiliation of the person performing the evaluation. In large buildings with more than one entrance, additional placards should be placed at all other entrances (ATC, 1989). Appendix B to these *Recommended Criteria* includes sample placards (from ATC, 1995).

In addition, a formal report should be prepared indicating the scope of evaluation that has been performed, the findings of the evaluation, including a description of any damage encountered, the appropriate postearthquake condition designation assigned to the building and any recommendations for additional evaluation, restrictions of occupancy and/or repair action. The report should be submitted to the party requesting the evaluation and to other parties as required by law.

4. LEVEL 1 DETAILED POSTEARTHQUAKE EVALUATIONS

4.1   Introduction

Detailed evaluation is the second step of the postearthquake evaluation process. It should be performed for all buildings that are estimated to have experienced potentially damaging ground motions, using the screening procedures of Section 3.2 of these *Recommended Criteria*. As detailed evaluation can be a time consuming process, it is recommended that a preliminary evaluation, in accordance with the procedures of Chapter 3, be conducted prior to detailed evaluation, to permit rapid identification of those buildings that may have been so severely damaged that they pose an immediate threat to life safety.

Many steel moment-frame buildings damaged in past earthquakes have displayed few outward signs of structural or nonstructural damage. Consequently, except for those structures which have been damaged so severely that they are obviously near collapse, brief evaluation procedures, such as those of Chapter 3, are unlikely to provide a good indication of the extent of damage or its consequences. In order to make such determination, it is necessary to perform detailed inspections of the condition of critical structural components and connections. If structural damage is found in the course of such inspections, it is then necessary to make a determination as to the effect of discovered damage on the structure's ability to resist additional loading. Ultimately, decisions as to the significance of damage, whether occupancy should be permitted in a building and whether specific types of damage should be repaired must be made on the basis of quantitative evaluation and engineering judgment.

This chapter provides simplified procedures for a quantitative evaluation method in which occupancy and repair decisions are assisted based on the calculation of a damage index, related to the distribution and severity of different types of damage in the structure. In order to apply this method, termed a Level 1 evaluation, it is necessary to obtain an understanding of the distribution of damage in the structure. This must be obtained by performing visual inspections of critical framing and connections. It is preferred that damage indices be calculated based on a determination of the condition of all critical connections in the building; however, it is permissible to infer a distribution of damage, and calculate a damage index, based on an appropriately selected sample of connections.

Chapter 5 provides recommended criteria for an alternative method of quantitative evaluation, termed a Level 2 evaluation, based on performing structural analysis of the damaged structure's ability to resist additional strong ground shaking. In order to perform a Level 2 evaluation, it is necessary to conduct a complete inspection of all fracture-susceptible connections in the building.

> *Commentary: The Level 1 evaluation approach contained in this chapter is based upon a methodology originally presented in FEMA-267, modified to account for experience gained in the application of the FEMA-267 guidelines to real buildings and also calibrated to expert opinion on the severity of various types of damage. The Level 2 evaluation is a more comprehensive approach that is*

compatible with the overall approach developed for performance evaluation of structures.

The Level 1 detailed evaluation procedure consists of gathering available information on construction of the structure and a multi-step inspection, evaluation, decision and reporting process. Although it is preferable to conduct a complete inspection of all fracture-susceptible connections, it is permissible to inspect only a selected portion of the elements and connections and to use statistical methods to estimate the overall condition of the building. A damage index is introduced to quantify the severity of damage in the building. This damage index is calculated based on individual connection damage indices, d_i, assigned to the individual inspected connections. These connection damage indices vary between 0 and 4, with 0 representing no significant earthquake damage and 4 representing severe damage. A story-level damage index, D_{max}, is introduced which varies between 0 and 1.0, depending on the severity of damage. Based on the maximum damage index obtained for any floor level, D_{max}, or if full inspections were not made of all connections, the probability that the damage index exceeds a specified threshold, recommendations are provided to the structural engineer regarding the appropriate damage condition designation as well as decisions regarding occupancy restrictions and repair actions.

4.2   Data Collection

Prior to performing a detailed inspection and evaluation, available information on the building's construction should be collected and reviewed. This review should be conducted in a manner similar to that indicated in Section 3.3.2, but extended to include greater knowledge, for example, of the primary lateral and gravity load-resisting systems, typical detailing, and presence of irregularities. Pertinent available engineering and geotechnical reports, including any previous damage survey reports, such as the preliminary postearthquake evaluation report prepared in accordance with Chapter 3 of these *Recommended Criteria*, and current ground motion estimates, should also be reviewed. Specifications (including the original Welding Procedure Specifications), shop drawings, erection drawings, and construction inspection records should be reviewed when available.

When structural framing information is not available, a comprehensive field study should be undertaken to determine the location and configuration of all lateral-force-resisting frames, and the details of their construction, including members' sizes, material properties, and connection configurations. See Section 5.2 for additional discussion.

4.3   Evaluation Approach

Analyses of buildings with brittle connections, such as those damaged by the 1994 Northridge earthquake, show that although damage occurs slightly more often in locations predicted by analysis to have high stress and deformation demands, damaged connections tend to be widely distributed throughout building frames, often at locations that analyses would not

predict. This suggests that there is some randomness in the distribution of the damage. To detect reliably all such damage, it is necessary to subject each fracture-susceptible connection to detailed inspections. Fracture-susceptible connections include:

- Moment-resisting beam-column connections in which the beams are connected to columns using full penetration welds between the beam flanges and column, and in which yield behavior is dominated by the formation of a plastic hinge within the beam at the face of the column, or within the column panel zone.

- Splices in exterior columns of moment-resisting frames when the splices consist of partial penetration groove welds between the upper and lower sections of the column, or of bolted connections that are incapable of developing the full strength of the upper column in tension.

The inspection of all such connections within a building can be a costly and disruptive process. Although complete visual inspections of fracture-susceptible connections are recommended as part of a Level 1 evaluation, this evaluation methodology permits a representative sample of the critical connections to be selected and inspected. When only a sample of connections is inspected use is made of statistical techniques to project damage observed in the inspected sample to that likely experienced by the entire building.

In order to obtain valid projections of a building's condition, when the sampling approach is selected, samples should be broadly representative of the varying conditions (location, member sizes, structural demand) present throughout the building and samples should be sufficiently large to permit confidence in the projection of overall building damage. Two alternative methods for sample selection are provided. When substantial damage is found within the sample of connections, additional connections should be inspected to provide better, more reliable information on the building condition.

Once the extent of building damage is determined, (or estimated if a sampling approach is utilized) the structural engineer should assess the residual structural capacity and safety, and determine appropriate repair and/or modification actions. General recommendations are provided, based on calculated damage indices. As an alternative to this approach, direct application of engineering analysis (Level 2 evaluation) may also be used as provided for in Chapter 5 of these *Recommended Criteria*.

4.4   Detailed Procedure

Postearthquake evaluation should be carried out under the direct supervision of a structural engineer. Two alternative procedures are presented below depending on whether all connections in the building are inspected, or only a sample of the connections in the building are inspected. Section 4.4.1 describes the procedure when all connections are inspected. Section 4.4.2 describes the procedure when a sample of connections are inspected.

As used in these *Recommended Criteria*, the term "connection" means that assembly of elements including the beam, column, plates, bolts, and welds, that connect a single beam to a single column. Interior columns of plane frames will typically have two connections (one for

each beam framing to the column) at each floor level. Exterior columns of plane frames will
have only one connection at each floor level.

4.4.1   Method 1 - Inspection of All Connections

The following five-step procedure may be used to determine the condition of the structure
and to develop occupancy, repair and modification strategies when all critical connections in a
building are inspected and the extent of damage to all connections is known:

Step 1:   Conduct a complete visual inspection of all fracture-susceptible connections in the
            building in accordance with Section 4.3. Moment-resisting connections should be
            inspected in accordance with Section 4.4.1.1, with supplemental nondestructive
            examination, as suggested in that section.

Step 2:   Assign a connection damage index, d_i, to each inspected connection in accordance
            with Section 4.4.1.2.

Step 3:   Calculate the floor damage index at each floor, D_j, pertinent to lateral force resistance
            of the building in each of two orthogonal directions, in accordance with Section
            4.4.1.3. Determine the maximum of the floor damage indices, D_{max}.

Step 4:   Based on the calculated floor damage indices, determine appropriate occupancy, and
            structural repair strategies, in accordance with Section 4.4.1.5. If deemed appropriate,
            the structural engineer may conduct detailed structural analyses of the building in the
            as-damaged state, to obtain improved understanding of its residual condition and to
            confirm that the recommended strategies are appropriate or to suggest alternative
            strategies. Recommendations for such detailed evaluations are contained in
            Chapter 5.

Step 5:   Report the results of the inspection and evaluation process to the building official and
            building owner.

4.4.1.1   Detailed Connection Inspections

In order to perform a detailed inspection of beam-column joints, it is necessary to remove any
fireproofing or other obscuring finishes to allow direct visual observation of the connection area.
Detailed inspections may be conducted in stages. An initial stage inspection may be performed
by removing only the limited amount of fireproofing indicated in Figure 4-1 and following the
inspection checklist of Section 4.4.1.1.1. If such initial inspection indicates the presence or
potential presence of damage, than a complete inspection, in accordance with the checklist of
Section 4.4.1.1.2 should be performed at each connection where such damage is detected. To
accommodate a complete inspection, removal of fireproofing as indicated in Figure 4-2 is
necessary. At the discretion of the engineer, a complete inspection in accordance with Section
4.4.1.1.2 may be performed without first performing the initial inspection of Section 4.4.1.1.1.
Refer to Chapter 3 for cautions with regard to removal of fireproofing materials.
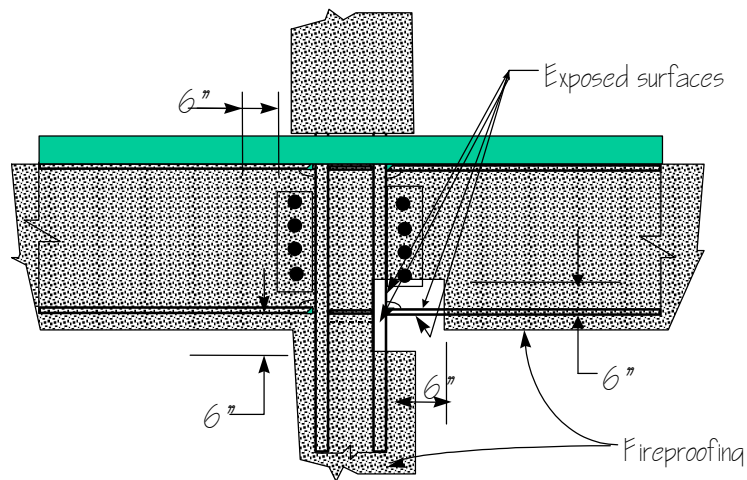
Figure 4-1   Fireproofing Removal for Initial Connection Inspection
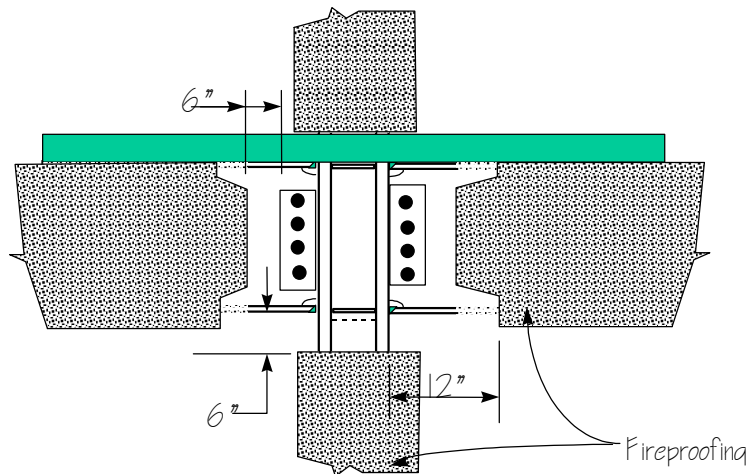
Figure 4-2   Fireproofing Removal for Complete Connection Inspection

The findings of detailed inspections of moment-resisting connections should be recorded on appropriate forms, documenting the location of the connection, the person performing the inspection, the date of the inspection, the extent of the inspection, the means of inspection (visual or nondestructive testing), the location and type of any observed damage, and, if no damage was observed, an indication of this. Appendix C includes forms suggested for this purpose. Detected damage should be classified in accordance with the system of Chapter 2.

> *Commentary: The largest concentration of reported damage following the 1994*
> *Northridge earthquake occurred at the welded joint between the bottom girder*
> *flange and column, or in the immediate vicinity of this joint. To a much lesser*
> *extent, damage was also observed in some connections at the joint between the*
> *top girder flange and column. If damage at either of these locations is*
> *substantial, then damage is also possible in the panel zone or shear tab areas.*
> *For this reason, and to minimize inspection costs, these Recommended Criteria*

*suggest that it is appropriate to initially inspect only the welded joint of the
bottom beam flange to the column, and only if damage is found at this location to
extend the inspection to the remaining connection components.*

4.4.1.1.1 Initial Inspections

The checklist below may be used as a guide for initial inspections. Prior to performing the
inspection, remove fireproofing (see Section 3.3.3), as indicated in Figure 4-1. If there are
indications of damage, then perform a complete inspection in accordance with the procedures of
Section 4.4.1.1.2.

☑ Observe the beam framing into the connection for trueness to line, and potential indications
of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).

☑ Observe condition of fireproofing along the beam within one beam depth of the column for
cracking or spalling of the fireproofing material along the beam surface, indicating potential
yielding or buckling of the beam flanges (damage types .G1, G2, Section 2.2.1).

☑ Observe the top and bottom surface of the exposed beam bottom flange for fractures (damage
types G3, G4, Section 2.2.1).

☑ Observe the exposed surfaces of the complete joint penetration weld between the beam
bottom flange and column for fractures (damage types W2, W3, W4 Section 2.2.3).

☑ Observe the exposed surfaces of the column flange for fractures (damage types C1, C2, C3,
Section 2.2.2).

☑ Observe the condition at the bottom of weld backing on the bottom flange. If gaps are
present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section
2.2.2).

☑ Observe the bottom surface of the top flange fireproofing at the locations where the beam
flanges join the column flanges (or continuity plates for minor axis connections) for cracks or
losses of fireproofing material that could indicate cracking at the complete joint penetration
weld (damage types G3, Section 5.3.1; C1, C3 and C4 Section 2.2.2; W2, W3, W4, Section
2.2.3).

☑ Observe the condition of the fireproofing at the beam web, in the vicinity of the connection
from the beam web to the column for loosened, cracked or spalled material indicative of
potential damage to shear tabs (damage types S1 through S5, Section 2.2.4).

☑ Observe the condition of the fireproofing at the column panel zone for cracks, loosened or
spalled material, indicative of damage to the panel zone or continuity plates (damage types
P1 through P8, Section 2.2.5).

☑ Observe the flanges of the column at and beneath the joint with the beam flange for loosened, spalled or cracked material, indicative of buckled or yielded sections (damage type C6, Section 2.2.2).

4.4.1.1.2   Detailed Inspections

When an initial inspection conducted in accordance with Section 4.4.1.1.1 indicates the presence or likely presence of damage in a connection, the more detailed inspections and observations indicated in the checklist below should be performed for that connection. Prior to performing the inspection, remove fireproofing (see Section 3.3.3), as indicated in Figure 4-2. Note that inspection of the top surface of the top flange of the beam and the adjacent column flange will typically be obscured by the diaphragm. If inspections from the exposed bottom surface of the top beam flange indicate a potential for damage to be present, then the diaphragm should be locally removed to allow a more thorough inspection.

☑ Observe the beam framing into the connection for trueness to line, and potential indications of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).

☑ Observe condition of fireproofing along the beam within one beam depth of the column for cracking or spalling of the fireproofing material along the beam surface, indicating potential yielding or buckling of the beam flanges (damage types G1, G2, Section 2.2.1).

☑ Observe the top and bottom surface of the exposed beam bottom flange and the bottom surface of the top flange for fractures (damage types G3, G4, Section 2.2.1).

☑ Observe the exposed surfaces of the complete joint penetration welds between the beam top and bottom flanges and column for fractures (damage types W2, W3, W4 Section 2.2.3).

☑ Observe the exposed surfaces of the column flanges for fractures (damage types C1, C2, C3, Section 2.2.2).

☑ Observe the condition at the bottom of weld backing on the top and bottom flanges. If gaps are present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section 2.2.2). See Chapter 2 for additional information.

☑ Observe the condition of the shear tab for deformation of the tab, fractures or tearing of the welds and loosening or breaking of the bolts (damage types S1 through S5, Section 2.2.4).

☑ Observe the column panel zone for cracks, or distortion (damage types P1 through P8, Section 2.2.5).

☑ Observe the exposed flanges of the column for distortion (damage type C6, Section 2.2.2).

4.4.1.2   Damage Characterization

Characterize the observed damage at each of the inspected connections by assigning a connection damage index, d_j, obtained either from Table 4-1a or Table 4-1b. Table 4-1a presents damage indices for individual classes of damage. Table 4-1b provides indices for the more

common combinations of damage and also provides a method for developing indices for other combinations.

Commentary: The connection damage indices provided in Table 4-1 (ranging from 0 to 4) represent judgmental estimates of the relative severity of the various types of damage. Damage severity is judged in two basic respects, the impact of the damage on the connection's ability to participate in the frame's global stability and lateral resistance, and the impact of the damage on the local gravity load carrying capacity of the individual connection. An index of 0 indicates no impact on either global or local stability while an index of 4 indicates very severe impact.

When initially developed, in support of the publication of FEMA-267, these connection damage indices ranged from 0 to 10 and were conceptualized as estimates of the connection's lost capacity to reliably participate in the building's lateral-force-resisting system in future earthquakes (with 0 indicating no loss of capacity and 10 indicating a complete loss of capacity). However, due to the limited data available, no direct correlation between these damage indices and the actual residual strength and stiffness of a damaged connection was possible. In these Recommended Criteria, the damage indices have been simplified, to remove the apparent accuracy implied by a scale ranging from 0 to 10. It should be noted that although the damage indices do not correlate directly with the loss of strength or stiffness experienced by a connection, they do provide a convenient qualitative measure of the extent of damage that various connections in a building have experienced.

Analyses conducted to explore the effect of connection fractures on the global behavior of frames have revealed that the loss of a single flange connection (top or bottom) at each joint, consistently throughout a moment-resisting frame results in only a modest increase in the vulnerability of the structure to developing P-delta instability and collapse. However, if a number of connections develop fractures at both flanges of the beam-column connection, significant increase in vulnerability occurs. As a result of this, damage that results in the loss of effectiveness of a single flange joint to transfer flexural tension stress is assigned a relatively modest damage index of 2, if not combined with other types of damage at the connection. Damage types that result in an inability of both flanges to transfer flexural demands are assigned a high damage index, of 4, as are types of damage that could potentially result in impairment of a column or beam's ability to continue to carry gravity loads. Other types of damage are assigned proportionately lower damage indices, depending on the apparent effect of this damage on structural stability and load carrying capacity.

Table 4-1a   Connection Damage Indices

Type	Location	Description[1]	Index d_j
G1	Girder	Buckled Flange	2
G2	Girder	Yielded Flange	0
G3	Girder	Top or Bottom Flange fracture in Heat Affected Zone (HAZ)	2
G4	Girder	Top or Bottom Flange fracture outside HAZ	2
G5	Girder	Not used	-
G6	Girder	Yielding or Buckling of Web	2
G7	Girder	Fracture of Web	4
G8	Girder	Lateral-torsional Buckling	2
C1	Column	Minor column flange surface crack	1
C2	Column	Flange tear-out or divot[4]	2
C3	Column	Full or partial flange crack outside HAZ	3
C4	Column	Full or partial flange crack in HAZ	3
C5	Column	Lamellar flange tearing	2
C6	Column	Buckled Flange	3
C7	Column	Fractured column splice	4
W2	CJP weld	Crack through weld metal exceeding t/4	2
W3	CJP weld	Fracture at girder interface	2
W4	CJP weld	Fracture at column interface	2
S1	Shear tab	Partial crack at weld to column	2
S2	Shear tab	Crack in Supplemental Weld (beam flanges sound)	1
S3	Shear tab	Fracture through tab at bolt holes	4
S4	Shear tab	Yielding or buckling of tab	3
S5	Shear tab	Damaged, or missing bolts[3]	2
S6	Shear tab	Full length fracture of weld to column	4
P1	Panel Zone	Fracture, buckle, or yield of continuity plate[2]	1
P2	Panel Zone	Fracture of continuity plate welds[2]	1
P3	Panel Zone	Yielding or ductile deformation of web[2]	0
P4	Panel Zone	Fracture of doubler plate welds[2]	1
P5	Panel Zone	Partial depth fracture in doubler plate[2]	1
P6	Panel Zone	Partial depth fracture in web[2]	3
P7	Panel Zone	Full (or near full) depth fracture in web or doubler plate[2]	4
P8	Panel Zone	Web buckling[2]	2
P9	Panel Zone	Fully severed column	4

Notes To Table 4-1a:

1. See Figures 2-2 through 2-6 for illustrations of these types of damage.
2. Panel zone damage should be reflected in the damage index for all moment connections that are attached to the damaged panel zone within the assembly.
3. Missing or loose bolts may be a result of construction error rather than damage. The condition of the metal around the bolt holes, and the presence of fireproofing or other material in the holes can provide clues to this. Where it is determined that construction error is the cause, the condition should be corrected and a damage index of "0" assigned.
4. Damage type C2 is very similar to type W3, the primary differentiation being the depth of the concave fracture surface into the column flange. If the fracture surface is relatively shallow within the column flange and does not result in the removal of substantial column flange material, type C2 fractures may be classified as type W3 and the corresponding damage index utilized.

Table 4-1b Connection Damage Indices for Common Damage Combinations

Girder, Column or Weld Damage	Shear Tab Damage	Damage Index	Girder, Column or Weld Damage	Shear Tab Damage	Damage Index
G3 or G4 Fracture of Girder Top or Bottom Flange	S1	4	C5 Column Flange Tearing parallel to rolling direction	S1	4
	S2	3		S2	3
	S3	4		S3	4
	S4	3		S4	3
	S5	4		S5	4
	S6	4		S6	4
C2 Column Flange Tear-out or Divot	S1	4	W2, W3, or W4 CJP Weld Fracture	S1	4
	S2	3		S2	3
	S3	4		S3	4
	S4	3		S4	3
	S5	4		S5	4
	S6	4		S6	4
C3 or C4 Column Flange Crack	S1	4			
	S2	4			
	S3	4			
	S4	4			
	S5	4			
	S6	4			

Note: For other combinations of damage, indices are obtained as follows:

  a. Two types of damage with individual $d_i \leq 1$, Combination $d_i = 2$
  b. Two types of damage with both individual $d_i > 1$, Combination $d_i = 4$.
  c. Two types of damage with only one individual $d_i \geq 2$, Combination d_i = largest individual $d_i + 1 \leq 4$.
  d. Three types of damage with all $d_i \leq 1$, Combination $d_i = 3$.
  e. Three types of damage with any $d_i \geq 2$, Combination $d_i = 4$.
  f. More than three types of damage, Combination $d_i = 4$.

4.4.1.3   Determine Damage Index at Each Floor for Each Direction of Response

Divide the connections in the building into two individual groups. Each group of connections should consist of those connections, which are part of frames that provide primary lateral-force resistance for the structure in one of two orthogonal building directions. For example, one group of connections will typically consist of all those connections located in frames that provide north-south lateral resistance, while the second group will be all those connections located in frames that provide east-west lateral resistance.

For each group of connections, determine the value of the damage index for the group at each floor, from the equation:

$$D_i = \frac{1}{n} \sum_{j=1}^{n} \frac{d_j}{4} \qquad (4\text{-}1)$$

where D_i is the floor damage index at floor "i" for the group,

    n is the number of connections in the group at floor level "i," and.

    d_j is the damage index, from Tables 4-1a and 4-1b, for the jth connection in the group at that floor.

4.4.1.4   Determine Maximum Floor Damage Index

Determine the maximum floor damage index for the building, D_{max}, consisting of the largest of the D_i values calculated in accordance with the Section 4.4.1.3.

4.4.1.5   Determine Recommended Recovery Strategies for the Building

Recommended postearthquake recovery strategies are as indicated in Table 4-2, based on the maximum damage index, D_{max}, determined in accordance with Section 4.4.1.4.

Table 4-2   Recommended Repair and Modification Strategies

Values of D_{max} [2]	Recommended Strategy	Note
$0 < D_{max} \leq 0.5$	Repair all connections discovered to have $d_j \geq 1$	
$D_{max} > 0.5$	A potentially unsafe condition should be deemed to exist unless a Level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j \geq 1$.	1

Notes to Table 4-2:

    1.   The determination that an unsafe condition may exist should be maintained until either:

        a.   Level 2 analyses indicate that a dangerous condition does not exist, or

        b.   recommended repairs are completed for all connections having $d_j \geq 2$.

    2.   See Section 4.4.1.4

Commentary: Recommendations to close a damaged building to occupancy should not be made lightly, as such decisions will have substantial economic impact, both on the building owner and tenants. A building should be closed to occupancy whenever, in the judgment of the structural engineer, damage is such that the building no longer has adequate lateral-force-resisting capacity to withstand additional strong ground shaking, or if gravity-load-carrying elements of the structure appear to be unstable.

When a building has been damaged, it is recommended that in addition to repair, consideration also be given to upgrade. This is particularly the case when damage is severe (computed D_{max} exceeding 0.5) and the estimated ground shaking that caused the damage is substantially less than that which would be used to design the building under currently applicable building codes. In such conditions, it can reasonably be expected that the building would not be able to reliably resist the levels of ground shaking that could credibly occur at the building site. In addition to these basic safety considerations, there are also economic reasons to consider upgrading a building concurrently with damage

repair. A significant portion of structural upgrade costs are a result of the need to move occupants out of construction areas as well as the need to selectively demolish and replace building finishes and utilities in areas affected by the work. Often the magnitude of such costs required to implement repairs are comparable to those that would be incurred in performing an upgrade, permitting improved future performance to be attained with relatively little increment in construction cost. Structural repair, by itself, will not typically result in substantial reduction in the vulnerability of the structure to damage from future earthquakes, while selected connection upgrade has the potential to greatly reduce future damage and losses.

A companion document to this publication, FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings, provides guidelines for assessing the probable performance of steel moment-frame buildings and for designing upgrades to improve this performance.

4.4.2 Method 2 – Inspection of a Sample of Connections

The following eight-step procedure may be used to determine the condition of the structure and to develop occupancy, repair and modification strategies when only a sample of the building's critical connections are inspected:

Step 1:   Categorize the moment-resisting connections in the building into two or more groups comprising connections expected to have similar probabilities of being damaged.

Complete steps 2 through 7 below, for each group of connections.

Step 2:   Determine the minimum number of connections in each group that should be inspected and select the specific sample of connections to be inspected.

Step 3:   Inspect the selected sample of connections using the procedures of Section 4.4.1 and determine connection damage indices, d_j, for each inspected connection.

Step 4:   If inspected connections are found to be seriously damaged, perform additional inspections of connections adjacent to the damaged connections.

Step 5:   Determine the average damage index d_{avg} for connections in each group, and then the average damage index at a typical floor for each group.

Step 6:   Given the average damage index for connections in each group, determine the probability P that, had all connections been inspected, the connection damage index for any group, at a floor level, would exceed 0.50, and determine the probable maximum floor damage index, D_{max}.

Step 7:   Based on the calculated damage indices and statistics, determine appropriate occupancy and structural repair strategies. If deemed appropriate, the structural engineer may conduct detailed structural analyses of the building in the as-damaged state, to obtain improved understanding of its residual condition and to confirm that

the recommended strategies are appropriate or to suggest alternative strategies. However, for such analyses to be meaningful, full inspections of all connections are required. Procedures for such detailed evaluations are contained in Chapter 5.

Step 8:   Report the results of the inspection and evaluation process to the building official and building owner.

Sections 4.4.2.1 through 4.4.2.7 indicate, in detail, how these steps should be performed.

Commentary: Following an earthquake, structural engineers and technicians qualified to perform these evaluations may be in high demand. Prudent owners may want to consider having an investigation plan already developed (Steps 1 and 2) before an earthquake occurs, and to have an agreement with appropriate structural engineering and inspection professionals and organizations to give priority to inspecting their buildings rapidly following the occurrence of an earthquake.

4.4.2.1   Evaluation Step 1 — Categorize Connections by Groups

The welded moment-resisting connections participating in the lateral-force-resisting system for the building are to be categorized into a series of connection groups. Each group consists of connections expected to behave in a similar manner (as an example, a group may consist of all those connections that are highly stressed by lateral forces applied in a given direction). As a minimum, two groups of connections should be defined - each group consisting of connections that primarily resist lateral movement in one of two orthogonal directions. It may be appropriate to define additional groups to account for unique conditions, including building configuration, construction quality, member size, grade of steel, or other factors that are likely to result in connection behavior substantially different from other connections in the building. Each connection in the building, including connections at the roof level, should be uniquely assigned to one of the groups, and the total number of connections in each group determined.

In buildings that have significant torsional irregularity, it may be advisable to define at least four groups—one group in each orthogonal direction on each side of an assumed center of resistance.

4.4.2.2   Step 2 — Select Samples of Connections for Inspection

Assign a unique identifier to each connection within each group. Consecutive integer identifiers are convenient to some of the methods employed in this Section.

For each group of connections, select a representative sample for inspection in accordance with either of Methods A or B, below. If the evaluation is being performed to satisfy a requirement imposed by the building official, a letter indicating the composition of the groups, and the specific connections to be inspected should be submitted to the building official prior to the initiation of inspection. The owner or structural engineer may at any time in the investigation process elect to investigate more connections than required by the selected method. However,

the additional connections inspected may not be included in the calculation of damage statistics under Step 4 (Section 4.4.2.4) unless they are selected in adherence to the rules laid out for the original sample selection, given below.

> *Commentary: The purpose of inspection plan submittal prior to the performance of inspections is to prevent a structural engineer, or owner, from performing (1) a greater number of inspections and (2) reporting data only on those which provide a favorable economic result with regard to building disposition. The building official need not perform any action with regard to this submittal other than to file it for later reference at the time the structural engineer's evaluation report is filed. During the inspection process, it may be decided to inspect additional connections to those originally selected as part of the sample. While additional inspections can be made at any time, the results of these additional inspections should not be included in the calculation of the damage statistics, in Step 5, as their distribution may upset the random nature of the original sample selection. If the additional connections are selected in a manner that preserves the distribution character of the original sample, they may be included in the calculation of the damage statistics in Step 5.*

4.4.2.2.1   Method A — Random Selection

In this method, connections should be selected for inspection such that a statistically adequate, random sample is obtained. The minimum number of connections to be inspected for each group should be determined in accordance with Table 4-3. For groups containing a population of 100 connections or more, the sample size need not exceed 18, unless damage with $d_j \geq 1.0$ in accordance with Table 4-1a is found in the inspection of these 18 connections. In the event that such damage is found in this initial sample, the sample size shall be expanded to the full amount shown in Table 4-3, while retaining the random character of the selection.

The following limitations apply to the selection of specific connections:

1. Up to a maximum of 20% of the total connections in any sample may be pre-selected as those expected by rational assessment to be the most prone to damage. Acceptable criteria to select these connections could include:

    - Connections shown by a rational analysis to have the highest demand/capacity ratios or at locations experiencing the largest drift ratios.

    - Connections that adjoin significant structural irregularities and which therefore might be subjected to high localized demands. These include the following irregularities:

        - re-entrant corners

        - set-backs

        - soft or weak stories

        - torsional irregularities (connections at perimeter columns)

- diaphragm discontinuities

- Connections incorporating the largest size framing elements.

2. The balance of the sample should be selected randomly from the remaining connections in the group, except that up to 10% of the connections in the sample may be replaced by other connections in the group to which access may more conveniently be made.

For buildings designed and constructed following the 1994 Northridge earthquake, and conforming to the recommendations contained in Chapter 7 of *FEMA-267*, or conforming to the design recommendations for Special Moment Frames contained in the 1997 or later edition of *AISC Seismic Provisions*, the scope of inspection may be reduced to 1/2 the number of connections indicated in Table 4-3. If in the course of this reduced scope of inspection, significant structural damage is found (damage to any connection with a damage index $d_j \geq 1.0$ from Table 4-1 (a or b)), then full inspections should be performed, as for buildings with other types of connections.

Table 4-3   Minimum Sample Size for Connection Groups

Number of connections in Group[1]	Minimum number of connections to be inspected	Number of Connections in Group[1]	Minimum number of connections to be inspected
6	3	200	30
10	4	300	40
15	5	400	50
20	6	500	60
30	8	750	75
40	10	1000	100
50	12	1250	110
75	16	1500	125
100	20	2000	150

Note: 1. For other connection numbers use linear interpolation between values given, rounding up to the next highest integer.

Commentary: The number of connections needed to provide a statistically adequate sample depends on the total number of connections in the group, the amount of damage present in the building, and the amount of damage it is acceptable not to find. Assuming that damage is randomly distributed within a connection group, if no damage is found in a randomly selected inspection sample of 18 connections, this indicates at least a 95% level of confidence that less than 15% of the connections in the group have been damaged for a group of any size. For smaller groups of connections, smaller samples will provide similar levels of confidence. However, if damage is present within the sample of connections selected for a group, then a larger sample size will be required to assure with confidence that the percentage of connections within the group that have been damaged is within a tolerable level. When implemented in the inspection procedures contained in these recommended criteria, the inspection sample sizes

specified in Table 4-3 will produce greater than a 95% level of confidence of finding damage in groups of connections with 20% or more of the connections damaged. Military standard, MIL-STD-105D can be used to determine appropriate sample sizes to obtain other levels of confidence or to obtain similar levels of confidence for reduced levels of damage, if desired.

If relatively few connections within a group are inspected, the standard deviation for the computed damage index will be large. This may result in prediction of excessive damage when such damage does not actually exist.   The structural engineer may elect to investigate more connections than the minimum indicated in order to reduce the standard deviation of the sample and more accurately estimate the total damage to the structure. These additional inspections may be performed at any time in the investigative process. However, care should be taken to preserve the random characteristics of the sample, so that results are not biased either by selection of connections in unusually heavy (or lightly) damaged areas of the structure.

It is recognized that in many cases the structural engineer may wish to pre-select those connections believed to be particularly vulnerable. However, unless these pre-selected connections are fairly well geometrically distributed, a number that is more than about 10% of the total sample size will begin to erode the validity of the assumption of random selection of the sample. If the structural engineer has a compelling reason for believing that certain connections are most likely to be damaged, and that more than 10% should be pre-selected on this basis, either the alternative approach of Method B should be used, or the connections that are believed to have particular vulnerability should be classified as an independent group, and treated accordingly.

It is also recognized that there is often a practical incentive to select connections that are in specific unoccupied or more accessible areas. It is suggested that no more than 10% of the total sample be composed of connections pre-selected for this reason. These connections, rather than having a higher disposition for damage, might well have a lower than average tendency to be damaged. An excessive number of this type of pre-selected connection would quickly invalidate the basic assumption of random selection. It is also recognized that during the inspection process conditions will be discovered that make it impractical to inspect a particular connection, e.g., the architectural finishes are more expensive to remove and replace than in other areas, or a particular tenant is unwilling to have their space disturbed. However, as discussed above, not more than 10% of the total connections inspected should be selected based on convenience.

There are a number of methods available for determining the randomly selected portion of the sample. To do this, each connection in the group (excluding pre-selected connections) should be assigned a consecutive integer

identifier. The sample may then be selected with the use of computer spread sheet programs (many of which have a routine for generation of random integers between specified limits), published lists of random numbers, or by drawing of lots.

4.4.2.2.2 Method B - Analytical Selection

In this method, connections should be selected for inspection in accordance with the following criteria:

1. The minimum number of connections within the group to be inspected shall be indicated in Table 4-3. As with Method A, if a randomly selected sample of 18 connections from a group is inspected, and found to contain no damage, no further inspections of connections from that group are required.

2. Up to 50% of the connections may be selected based on the results of rational analysis indicating those connections most likely to be damaged.

3. The remaining connections in the group to be inspected are selected such that the sample contains connections distributed throughout the building, including upper, middle and lower stories. The rules of Section 4.4.2.2.1 should be followed in a general way.

Prior to initiation of the inspections, the rational analysis and list of connections to be inspected should be subjected to a qualified independent third party review. The peer review should consider the basis for the analysis, consistency of the assumptions employed, and assure that overall, the resulting list of connections to be inspected provides an appropriate sampling of the building's connections.

During the inspection process, up to 10% of the connections in the sample may be replaced by other connections to which access may more conveniently be made. Substitution for more than 10% of the connection sample may be made provided that the independent third party reviewer concurs with the adequacy of the resulting revised sample.

> *Commentary: In analyses conducted of damaged buildings, there has been a generally poor correlation of the locations of damage and the locations of highest demand predicted by analysis. This is primarily attributed to the fact that the propensity for a fracture to initiate in a connection is closely related to the workmanship present in the welded joints, which tends to be a randomly distributed quantity. Moreover, typical analysis methods do not capture the complex nonlinear stress state that occurs in actual buildings. However, there has been some correlation. Analysis is a powerful tool to assist the structural engineer in understanding the expected behavior of a structure, damaged or undamaged. The specific analysis procedure used should be tailored to the individual characteristics of the building. It should include consideration of all building elements that are expected to participate in the building's seismic response, including, if appropriate, elements not generally considered to be part of the lateral-force-resisting system. The ground motion characteristics used for*

the analysis should not be less than that required by the building code for new construction, and to the extent practical, should contain the spectral characteristics of the actual ground motion experienced at the site. Qualified independent review is recommended to assure that there is careful consideration of the basis for the selection of the connections to be inspected and that a representative sample is obtained.

4.4.2.3   Step 3 — Inspect the Selected Samples of Connections

4.4.2.3.1 Inspection

All moment-resisting connections within each sample are to be visually inspected as indicated in Section 4.4.1.1.2. Where visual inspection indicates the potential for damage that is not clearly visible, further investigation using nondestructive testing should be performed. Characterize all damage discovered by visual inspection and nondestructive testing for each inspected connection as described in Section 4.4.1.1 An individual data sheet (Appendix C) should be filled out for each connection inspection, recording its location and conditions observed. In addition, plan and elevation sketches for the building's structural system should be developed and conditions of observed damage recorded on these sketches.

Commentary: The largest concentration of reported damage following the 1994 Northridge earthquake occurred at the welded joint between the bottom girder flange and column, or in the immediate vicinity of this joint. To a much lesser extent, damage was also observed in some buildings at the joint between the top girder flange and column. If damage at either of these locations is substantial, then damage is also commonly found in the panel zone or shear tab areas.

For a Level 1 evaluation, these Recommended Criteria permit inspection, by visual means, of all of the potential damage areas for a representative sample of the connections in the building. Most of the damage reported in buildings following the 1994 Northridge earthquake consisted of fractures that initiated at the roots of complete joint penetration welds joining beam flanges to column flanges, and which then propagated through the weld or base metal, leaving a trace that was generally detectable by careful visual examination. Careful visual examination requires removal of all obscuring finishes and fireproofing, and examination from a range of a few inches. Most fractures are visually evident. However, some fractures are rather obscure since deformation of the building following the onset of fracture can tend to close up the cracks. In some cases, it may be appropriate to use magnifying glasses or other means to verify the presence of fractures. If doubt exists as to whether a surface indication is really a fracture, magnetic particle testing and other forms of nondestructive examination can be used to confirm the presence of a fracture. The surface must be carefully cleaned prior to testing.

Some types of fractures extend from the root of the beam flange weld into the column flange and may not be detectable by visual examination. Such fractures,

*typified by types C3 and C5 (see Section 2.2.2) can only be detected by removal of
the backing, or by nondestructive testing. Often, when such fractures are present,
a readily visible gap can be detected between the base of the backing and the
column flange. Where such indications are present, a feeler gauge should be
inserted into the gap to determine its depth. If the feeler gauge can be inserted to
a depth that exceeds the backing thickness, a fracture should be assumed to be
present. Removal of the backing, or nondestructive testing, or both, will be
required to confirm the extent of the crack.*

*The practice of inspecting a small sample of the total connections present in a
building, in order to infer the probable overall condition of the structure is
consistent with that followed by most engineers in the Los Angeles area, following
the 1994 Northridge earthquake. However, the typical practice following that
event included the extensive use of ultrasonic testing (UT) in addition to visual
inspection. This UT revealed a number of apparent conditions of damage at the
roots of the full penetration welds between beam and column flanges. These
conditions, which were widespread, were typically reported by testing agencies
and engineers as damage. This practice was encouraged by the FEMA-267
guidelines, which classified weld root indications as type W1 "damage".*

*As a result of limitations in the accuracy of ultrasonic testing techniques it
was often found upon removal of weld backing material to allow repair of these
root conditions, that the actual condition of the weld root was significantly
different from that indicated by UT. Sometimes, no flaws at all were found at the
roots of welds reported to have W1 conditions while in other cases, the size and
location of actual flaws were found to be significantly different from that
indicated by the UT.*

*In the time since, substantial evidence has been gathered that suggests that
many of the W1 conditions reported following the 1994 Northridge earthquake
were not damage, but rather latent construction defects, including slag inclusions
and lack of fusion that had never been detected during the original construction
quality control and quality assurance processes. For these reasons, these Recom-
mended Criteria have de-emphasized, relative to the recommendations of FEMA-
267, the importance of employing NDT in the postearthquake inspection process.*

4.4.2.3.2   Damage Characterization

The observed damage at each of the inspected connections is characterized by assigning a
connection damage index, d_j obtained either from Table 4-1a or Table 4-1b, of Section 4.4.1.2.
Table 4-1a presents damage indices for individual classes of damage. Table 4-1b provides
combined indices for the more common combinations of damage and a rule for combining
indices where a connection has more than one type of damage. Refer to Chapter 2 for
descriptions of the various damage types and to Section 4.4.1.2 for commentary relative to these
damage indices.

4.4.2.4   Step 4 — Inspect Connections Adjacent to Damaged Connections

Regardless of the method used to select the connection sample, perform additional inspections of moment-resisting connections near connections with significant damage as follows:

- When a connection is determined to have a damage index within the range $1 \leq d_j \leq 2$, inspect all of the moment-resisting connections in that line of framing on both sides of the affected column and of the column(s) adjacent to the affected column at that floor level and on the affected column at the floor level immediately above and below the damaged connection (See Figure 4-3). Also inspect any connections for beams framing into the column in the transverse direction at that floor level, at the damaged connection.

- When a connection is determined to have a damage index $d_j \geq 3$, inspect all of the moment-resisting connections in that line of framing on both sides of the affected column and of the column(s) adjacent to the affected column at that floor level and on the affected column at the two floor levels immediately above and below the damaged connection (See Figure 4-4). Also inspect any connections for beams framing into the column in the transverse direction at that floor level at the damaged connection.

- ● Damaged moment-resisting connection with $1.0 \leq d_j \leq 2.0$
- ○ Adjacent moment-resisting connection - to be inspected
- Δ Transverse connection to be inspected
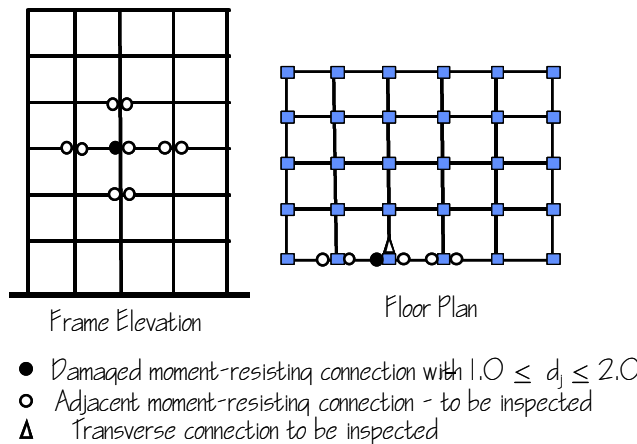
Figure 4-3   Inspection of Connections Adjacent to Damaged Connection ($1 \leq d_j \leq 2$)

- ■ Damaged moment-resisting connection with dj ³³
- ○ Adjacent moment-resisting connection to be inspected
- Δ Transverse connection to be inspected
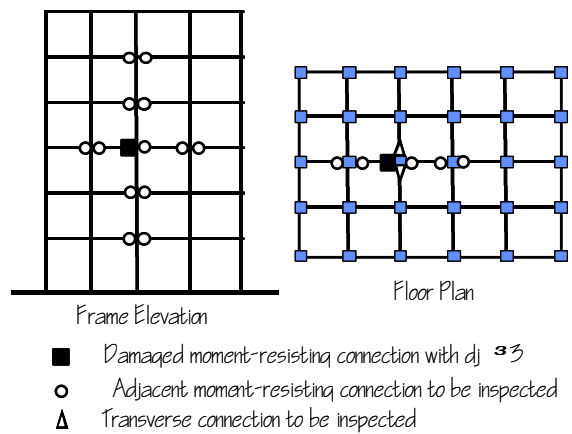
Figure 4-4   Inspection of Connections Adjacent to Damaged Connection ($d_j \geq 3$)

Assign damage indices d_j per Tables 4-1a and 4-1b to each additional connection inspected. If significant damage is found in these additional connections ($d_j \geq 1$), then inspect the connections near these additional connections, as indicated above. Continue this process, until one of the following conditions occurs:

- The additional connection inspections do not themselves trigger more inspections, or

- All connections in the group have been inspected. In this case, proceed with the evaluation of damage indices for this group in accordance with the guidelines of Section 4.4.1.3.

The results of these added connection inspections, performed in this step, are not included in the calculation of average damage index d_{avg} in Section 4.4.2.6, but are included in the calculation of the maximum likely floor damage index D_{max} and the probability of excessive damage P in Section 4.4.2.7.

4.4.2.5   Step 5 — Determine Damage Statistics for Each Group

For each group of connections, determine the estimated average value of the damage index for the group d_{avg} and its standard deviation σ from the equations:

$$d_{avg} = \frac{1}{n}\sum_{j=1}^{n} d_j \tag{4-2}$$

$$s = \sqrt{\frac{1}{n-1}\sum_{j=1}^{n}\left(d_j - d_{avg}\right)^2} \tag{4-3}$$

where n is the number of connections in the original sample selected for inspection under Step 2 (Section 4.4.2.2), and

  d_j is the damage index, from Tables 4-1a and 4-1b, for the jth inspected connection in the original sample

The additional connections selected using the procedure of Section 4.4.2.4 (Step 4) are not included in the above calculation.

4.4.2.6   Step 6 — Determine the Probability that the Connections in a Group at a Floor Level Sustained Excessive Damage

In this procedure, the probable maximum floor damage index at a floor D_{max} is estimated from the damage indices determined for all of the connections actually inspected, including those additional connections inspected in accordance with the requirements of Section 4.4.2.4. In addition, the probability P that had all connections in the building been inspected, D_{max} would exceed a value of 0.50, is determined.

First determine the average floor damage index D and its standard deviation S from the equations:

$$D = \frac{d_{avg}}{4} \tag{4-4}$$

$$S = \frac{\sigma}{4\sqrt{k}} \tag{4-5}$$

where d_{avg} is the average connection damage index, computed from Equation 4-2,

σ is the standard deviation of the connection damage index, computed from Equation 4-3,

k is the total number of connections (both inspected and not inspected) in the group at a typical floor.

Second, determine the probability P that the set of connections within the group at any floor has a floor damage index that is greater than or equal to 0.50. This may be done by using the parameters D and S to calculate a factor b, which represents the number of multiples of the standard deviation of a normal distribution above the mean that would be required to exceed 1/2. The factor b is calculated from the equation:

$$b = \left(\frac{1}{2} - D\right)\Big/ S \tag{4-6}$$

Using the value of b calculated from equation 4-6, determine P_f, from Table 4-4. P_f is the probability that if all connections had been inspected, the cumulative damage index at any floor would have been found to exceed 0.50. If the probability P_f is high, this strongly suggests the possibility that there has been a significant reduction in seismic resisting capacity.

Next, determine the probability P that if all connections within the group had been inspected, the connections within the group on at least one floor (out of q total floors in the group) would have been found to have a cumulative damage index of 0.50 or more from the equation:

$$P = 1 - (1 - P_f)^q \tag{4-7}$$

Finally, for each floor i in the group for which an inspection has been performed, determine the floor damage index D_i from the equation:

$$D_i = \frac{(k_i - m_i)d_{avg}}{4k_i} + \left(\frac{1}{k_i}\right)\sum_{j=1}^{m_i}\frac{d_j}{4} \tag{4-8}$$

where: k_i is the total number of connections in the group at floor i

m_i is the number of inspected connections in the group at floor i including the additional connections inspected under Step 4

Take D_{max} as the largest of the D_i values calculated for each floor of the group.

Table 4-4 P_f as a Function of Parameter b

b	P_f	b	P_f
-1.2816	0.90	1.2265	0.11
-0.8416	0.80	1.2816	0.10
-0.5244	0.70	1.3408	0.09
-0.2533	0.60	1.4051	0.08
0.0000	0.50	1.4395	0.075
0.2533	0.40	1.4758	0.07
0.5244	0.30	1.5548	0.06
0.8416	0.20	1.6449	0.05
0.8779	0.19	1.7507	0.04
0.9154	0.18	1.8808	0.03
0.9542	0.17	1.9600	0.025
0.9945	0.16	2.0537	0.02
1.0364	0.15	2.1701	0.015
1.0803	0.14	2.3263	0.01
1.1264	0.13	3.0962	0.001
1.1750	0.12	3.7190	0.0001

Note: Intermediate values of P_f may be determined by linear interpolation

Commentary: The criterion for damage evaluation used in these Recommended Criteria is to assume that a cumulative damage index of 0.50 marks the threshold at which a structure may become dangerous. Such a damage index could correspond to cases where 1/2 of the connections at a floor level have been severely damaged, or cases where all of the connections at a floor level have experienced moderate damage, or some combination of these, and therefore represents a reasonable point at which to begin serious consideration of a building's residual ability to withstand additional loads.

Although the actual form of the distribution of the probability of damage for an individual connection is not known, as the number of connections increases, the distribution of damage for a structure tends to a normal distribution, regardless of the form of the distribution for individual connections, by the Central Limit Theorem. Therefore, the probability that a damage index of 0.50 has been exceeded at a floor, in a group with k connections, may be approximated by determining how many multiples b times the standard deviation S, when added to the mean damage index D, equals 1/2. Or, in equation form :

$$D + bS = 0.50 \qquad\qquad (4\text{-}9)$$

Solution of this equation for the multiplier b results in the required relationship of equation 4-6.

In spite of the somewhat arbitrary nature of the 0.50 damage index criterion and the judgmental nature of the suggested way of testing whether that criteria has been exceeded, it is believed that the results of these procedures will lead to reasonable conclusions in most cases. However, it is always the prerogative of

*the responsible structural engineer to apply other rational techniques, such as
direct analyses of the remaining structural strength, stiffness, and deformation
capacity as a verification of the conclusions provided by these procedures.
Particularly in anomalous or marginal cases, such additional checks based on
engineering judgment are strongly encouraged.*

4.4.2.7 Step 7--Determine Recommended Recovery Strategies for the Building

Recommended postearthquake recovery strategies are as indicated in Table 4-5, based on the
calculated damage indices and statistics determined in the previous steps.

Table 4-5   Recommended Condition Designation and Repair Strategies

Values of D_{max} and P	Condition Designation	Recommended Strategy (Cumulative)	Note
$P \leq 10\%$ and $D_{max} \leq 0.2$	Green - 3	Repair all connections discovered to have $d_j \geq 1$	1,2
$10\% < P \leq 25\%$ or $0.2 < D_{max} \leq 0.5$	Green - 3	Inspect all connections in the group. Repair all connections with $d_j \geq 1$	1,2
$P > 25\%$ or $D_{max} > 0.5$	Red – 2	A potentially unsafe condition should be deemed to exist unless a level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j \geq 1$. Consider structural upgrade.	3

Notes to Table 4-5:
1. Includes damage discovered either as part of Step 2 or Step 3.
2. If all of the discovered damage is relatively minor ($d_j < 1$), at the discretion of the engineer, this
   need not be repaired. However, if some of the discovered damage is significant ($d_j \geq 1$), all of the
   damage should be repaired.
3. The determination that an unsafe condition may exist should continue until either:
   a. full inspection reveals that the gravity system is not compromised, and that the damage index
      at any floor does not exceed 0.50, or
   b. level 2 analyses indicate that a dangerous condition does not exist, or
   c. recommended repairs are completed for all connections having $d_j \geq 1$.

*Commentary: Recommendations to close a damaged building to occupancy
should not be made lightly, as such decisions will have substantial economic
impact, both on the building owner and tenants. A building should be closed to
occupancy whenever, in the judgment of the structural engineer, damage is such
that the building no longer has adequate lateral-force-resisting capacity to
withstand additional strong ground shaking, or if gravity-load-carrying elements
of the structure appear to be unstable.*

When a building has been damaged, it is recommended that, in addition to repair, consideration also be given to upgrade. Refer to the additional commentary in Section 4.4.1.5.

4.4.3 Additional Considerations

Regardless of the value calculated for the damage indices, in accordance with the previous sections, and the recommended actions of Section 4.4.2.7, the engineer should be alert for any damage condition that results in a substantial lessening of the ability of the structure as a whole, or of any part of the structure to resist gravity loads. Should such a condition be encountered, the engineer should inform those with legal standing to take appropriate steps either to limit entry to the affected portion(s) of the structure, or to ensure that adequate shoring is provide to prevent the onset of partial or total building collapse.

4.5 Evaluation Report

Upon completion of a detailed evaluation, the responsible structural engineer should prepare a written evaluation report and submit it to the person requesting the evaluation, as well as any other parties required by law to receive such a report. In particular, the building official should be notified whenever a hazardous condition is determined to exist. The report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following items:

- Building address

- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, the type and location of lateral-force-resisting elements. Include a description of the grade of steel specified for beams and columns and, if known, the type of welding (for example, shielded metal are welding or flux-cored arc welding) present. Indicate if moment connections are provided with continuity plates. The narrative description should be supplemented with sketches (plans and evaluations) as necessary to provide a clear understanding of pertinent details of the building's construction. The description should include an indication of any structural irregularities as defined in the Building Code.

- A description of nonstructural damage observed in the building, especially as relates to evidence of the drift or shaking severity experienced by the structure.

- If a letter was submitted to the building official before the inspection process was initiated that indicated how the connections were to be divided into groups and indicating the specific connections to be inspected, a copy of this letter should be included.

- A description of the inspection and evaluation procedures used, including the signed inspection forms for each individual inspected connection.

- A description, including engineering sketches, of the observed damage to the structure as a whole (e.g., permanent drift) as well as at each connection, keyed to the damage types in Table 4-1a, photographs should be included for all connections with damage index $d_i \geq 1$.

- Calculations of d_{avg}, D_i, and D_{max} for each group, and if all connections in a group were not inspected, P_f and P.

- A summary of the recommended corrective actions (repair and modification measures) and any recommendations on occupancy restrictions.

The report should include identification of any potentially hazardous conditions which were observed, including corrosion, deterioration, earthquake damage, pre-existing rejectable conditions, and evidence of poor workmanship or deviations or alterations from the approved drawings. In addition, the report should include an assessment of the potential impacts of observed conditions on future structural performance and recommendations for remediation of any adverse conditions. The report should include the Field Inspection Reports of damaged connections, as an attachment, and should bear the seal of the structural engineer in charge of the evaluation.

5. LEVEL 2 DETAILED POSTEARTHQUAKE EVALUATIONS

5.1   Introduction

Detailed evaluation is the second step of the postearthquake evaluation process. It should be performed for all buildings that are estimated to have experienced potentially damaging ground motions, using the screening procedures of Section 3.2 of these *Recommended Criteria*. As detailed evaluation can be a time-consuming process, it is recommended that a preliminary evaluation, in accordance with the procedures of Chapter 3, be conducted prior to detailed evaluation, to permit rapid identification of those buildings that may have been so severely damaged that they pose an immediate threat to life safety.

Many steel moment-frame buildings damaged in past earthquakes have displayed few outward signs of structural or nonstructural damage. Consequently, except for those structures which have been damaged so severely that they are obviously near collapse, brief evaluation procedures, such as those of Chapter 3, are unlikely to provide a good indication of the extent of damage or its consequences. In order to make such determination, it is necessary to perform detailed inspections of the condition of critical structural components and connections. If structural damage is found in the course of such inspections, it is then necessary to make a determination as to the effect of discovered damage on the structure's ability to resist additional loading. Ultimately, decisions as to the significance of damage, whether occupancy should be permitted in a building and whether specific types of damage should be repaired must be made on the basis of quantitative evaluation and engineering judgement.

Chapter 4 provides a series of recommended criteria for a detailed evaluation method in which occupancy and repair decisions are made based on the calculation of damage indices based on the observed distribution of damage in the structure. The distribution of damage is determined on the basis of detailed inspections of fracture-susceptible connections. Although it is preferred that all fracture-susceptible connections be inspected, the procedures of Chapter 4 permit inspections to be limited to a representative sample. This chapter provides procedures for a detailed evaluation processes based on structural analysis of the damaged structure's ability to resist additional strong ground shaking. In order to perform such an analysis it is necessary to inspect all fracture-susceptible connections in the building in order to understand their condition.

> *Commentary: The Level 1 evaluation approach of Chapter 4 is based on the
> methodology developed immediately after the 1994 Northridge earthquake and
> first presented in FEMA-267. The Level 2 evaluation approach described in this
> chapter is a more comprehensive analytical approach that is compatible with the
> analytical methodology that forms the basis for design and performance
> evaluation criteria contained in the suite of FEMA/SAC publications on steel
> moment frames.*

5.2   Data Collection

Prior to performing a detailed evaluation, the original construction drawings should be reviewed (if available) to identify the primary lateral and gravity load-resisting systems, typical detailing, presence of irregularities, and other features pertinent to structural performance. Pertinent available engineering and geotechnical reports, including any previous damage survey reports and current estimates of ground motion intensity for the damage causing event, should also be reviewed. Specifications (including the original Welding Procedure Specifications) shop drawings, erection drawings, and construction records should be reviewed when available.

When structural framing information is not available, a comprehensive field study must be undertaken to determine the location and configuration of all vertical frames, and the details of their construction including member sizes, material properties, and connection configurations. A companion publication, *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings,* provides procedures for obtaining as-built information and determining material properties for steel moment-frame buildings.

> *Commentary: It is important to collect data on all framing, whether or not it was originally intended as part of the design to participate in the lateral force resistance of the structure. Studies have shown that vertical frames provided only for gravity load resistance can provide substantial supplemental stiffness and strength in steel moment-frame structures and the analytical procedures of this chapter include direct consideration of such framing. Data collection should obtain sufficient information on this framing, as well as that intended to provide the structure's lateral-force resistance to permit an accurate analytical model of the structure to be developed.*

In addition to reviewing available documentation, a complete inspection of all critical framing and connections in the building should be undertaken, to determine their condition. Connections to be inspected include all fracture-susceptible moment-resisting framing connections and column splices. The following connections are considered to be fracture-susceptible:

- Moment-resisting beam-column connections in which the beams are connected to columns using full penetration welds between the beam flanges and column, and in which yield behavior is dominated by the formation of a plastic hinge within the beam at the face of the column, or within the column panel zone.

- Splices in the exterior columns of steel moment frames when the splices consist of (1) partial penetration groove welds between the upper and lower sections of the column, or (2) bolted connections that are incapable of developing the full strength of the upper column in tension.

Section 4.4.1.1 provides procedures for conducting connection inspections, and for classifying and recording any damage found.

*Commentary: Most welded, moment-resisting beam-column connections
constructed prior to 1994 will be of the fracture susceptible type described here.
Following the 1994 Northridge earthquake, guidelines for improved connection
designs and details were developed and were rapidly adopted throughout the
western United States, particularly in zones of high seismicity, including
California, Washington, Utah and Alaska. However, fracture-susceptible
connections may exist in some post-1994 buildings, particularly those constructed
in zones of lower seismicity.*

5.3   Evaluation Approach

In a Level 2 evaluation, inspections are conducted of all critical structural elements and
connections. An analytical model is then developed for the building representing its strength and
stiffness in the damaged state and an analysis is performed to provide information on the residual
capacity of the building to resist additional earthquake loading. The results of the analysis are
used together with engineering judgement and evaluation of other important factors including the
nature of the building's occupancy, the economic and other impacts of loss of building use and/or
building failure, in order to form an opinion as to appropriate postearthquake disposition for the
building. Alternative actions that may be appropriate in different situations include:

- Accept the damage as being stable and not detrimental to future building performance, in
  which case no repair action will be required.

- Determine that repairs of some or all of the damage must be undertaken to provide an
  acceptable level of risk for long-term occupancy, but that the building remains an acceptable
  risk for occupancy until such time as the repairs are completed.

- Determine that the building is an unacceptable risk for occupancy until such time as
  temporary stabilization or permanent repair can be undertaken.

- Determine that the building is an unacceptable risk for occupancy until such time as repair
  and structural upgrade can be undertaken.

- Determine that the building is an unacceptable risk for occupancy and impractical to repair
  and upgrade, in which case the building should be demolished.

A number of alternative analytical approaches may be used in support of the formation of
recommendations for postearthquake building disposition. Individual engineers and building
officials may choose to use any or perhaps several of these approaches, in support of the
postearthquake decision making process:

- **Determine the capacity of the damaged building relative to current code requirements**.
  In this approach the ability of the damaged building to meet the strength and drift criteria
  specified by the building code for new construction is evaluated. Decisions relative to repair
  and occupancy are triggered based on the extent of compliance of the damaged building with
  new building requirements. For example, if a damaged building provides 90% or more of the
  strength and stiffness required of new buildings, and the damage is stable, i.e., not subject to
  further degradation, then it may be appropriate to accept the damage and conduct no repairs.

If the degraded strength or stiffness of the building fall below 50% of that required for a new building it may be appropriate to restrict occupancy. It should be noted that the engineer, and building official, may select any appropriate "trigger" rules when using this approach.

- **Determine the capacity of the damaged building relative to pre-earthquake conditions.** In this approach, the amount that the strength and stiffness of the building has degraded as a result of the damage incurred relative to the pre-earthquake condition is used as an index to guide decisions. For example, if a building retains 90% of the strength and stiffness that existed prior to the earthquake, and the damage is stable, than it may be appropriate to accept the damage and conduct no repairs. If the degraded strength or stiffness of the building fall below 50% of the pre-earthquake values, then it may be appropriate to restrict occupancy. As noted above, the engineer and building official may select any appropriate "trigger" rules when using this approach.

- **Determine the probability of earthquake-induced collapse of the damaged building**. In this approach, a direct evaluation of the building's ability to resist collapse for a defined level of ground shaking (or at defined hazard probability) is determined and used as a basis for making decisions. For example, if analyses permit a high level of confidence to be developed that a damaged building can provide Collapse Prevention performance for ground shaking demands with a 10% chance of exceedance in 50 years, and the damage is stable, it may be appropriate to accept the damage, without repair. Similarly, if a high degree of confidence can not be developed that the damaged building could survive ground shaking demands with 50% chance of exceedance in 50 years, it may be decided to restrict building occupancy. Again, the specific "trigger" rules may be selected based upon the judgement of the engineer and building official.

The recommended criteria of this chapter adopt the last approach indicated above. Specifically, a methodology is provided whereby the engineer can determine a level of confidence with regard to the ability of the damaged building to resist a repeat of the same ground shaking that caused the initial damage, without collapse. If a high degree of confidence is obtained that the building could survive such ground shaking without significant risk to life safety then the building can remain occupied. If there is low confidence that the building can protect life safety in a repeat of the same event, then occupancy restrictions are recommended.

The basic tool used to implement any of the evaluation approaches described above is a structural analysis of the damaged building. In addition to presenting detailed criteria for the probabilistic evaluation process, this chapter also provides guidance on modeling of damaged structures that can be useful with any analytical approach selected by the engineer in assessing appropriate postearthquake actions.

> *Commentary: As noted, a number of different criteria have historically been used to determine whether a building has sustained so much damage that it should not continue to remain occupied. In all of these, the decision to post a building against occupancy is based on a finding that the building is likely to endanger life safety if subjected to additional strong ground shaking. Approaches that have most commonly been used in the past include:*

- *comparison of the building's residual lateral-force-resisting capacity with that specified by the building code for design of new structures,*

- *comparison of the building's residual lateral-force-resisting capacity with that which existed prior to the onset of damage, and*

- *application of the engineer's judgment as to the extent which the building poses an imminent or extreme hazard.*

Each of these approaches has drawbacks. If a comparison of the building's residual lateral-force-resisting capacity with that specified by the building code is used, it will often be found that a building that has not been damaged or has only minimal damage falls below the trigger level that indicates a "dangerous" condition, just due to the fact that the building was designed to earlier editions of the code that had less stringent design criteria. This results in a paradox, in that engineers typically do not post buildings as "unsafe", even if they have low calculated lateral-force-resisting capacity, unless they have been severely damaged.

The second approach, in which the computed degradation of a building's lateral-force-resisting capacity is used as the measure of whether or not a building should be occupied is somewhat more attractive in that it provides a direct measure of the effect of the damage sustained on the safety of the building and thereby differentiates low-strength conditions that are a result of original design characteristics, as opposed to those resulting from damage. However, this approach is also somewhat flawed in that some buildings have significant over-strength and reserve capacity and can sustain substantial reduction in initial capacity without becoming hazardous.

Approaches limited to application of the engineers judgment are attractive to many engineers, but inherently arbitrary. Further, different engineers will form different judgments as to the hazard that damage has caused in a building and will recommend different posting actions.

Review of statistics of past earthquakes indicates that within the relatively brief period of a year or so following a major earthquake in a region, the most likely events that the region will experience are of a similar or reduced magnitude to the original shock. Therefore, these procedures recommend evaluation of damaged structures for their ability to resist collapse (ability to provide Collapse Prevention performance) for such an event. For the purposes of accounting for variability in the likely locations and magnitudes of major aftershocks, and also to permit development of confidence levels for ability of the building to provide Collapse Prevention performance, a one-year return period is assumed for an arbitrary aftershock, comparable in intensity at the building site to the initial shock. Variability in ground motion is somewhat arbitrarily accounted for by

assuming a distribution of likely ground shaking at the building site due to such an aftershock that has a mean value equal to that which caused the original damage and having a coefficient of variation of 0.5.

The safety evaluation approach presented in this section is intended only for use in assessing whether a building should remain occupied while it is repaired, based on the probability of collapse during the period immediately following the earthquake. It is not intended as a tool for evaluating the adequacy of building performance over the longer term of the building's remaining life. For guidelines on such performance evaluations refer to the companion publication, FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.

5.4   Field Inspection

Prior to performing an analytical evaluation of building safety, a thorough inspection of the building should be conducted to determine its condition. This inspection should include visual inspection of all critical connections including moment-resisting beam-column connections and column splices, supplemented by nondestructive testing where visual inspection reveals the fracture-susceptible potential damage that cannot be quantified by visual means alone. Beam-column connections should be inspected, and the damage recorded, as indicated in Section 4.4.1.

Geologic site hazards such as fault rupture, landslide, rock fall, and liquefaction may influence the damage in a building and also its future performance. A detailed discussion of these hazards is provided in *FEMA 273* and should be considered as part of a postearthquake evaluation. The structure should be inspected to detect whether differential settlement has occurred as differential movement between columns in a frame has the potential to place severe demands on the moment connections.

Commentary: Foundation inspection is typically difficult to accomplish since most foundations are buried. In most cases, inspection of foundation condition can be performed by observing floors for indications of settlement. Where significant settlements are indicated, local excavation to expose the foundation condition for inspection should be considered.

5.5   Material Properties and Condition Assessment

In order to perform a meaningful evaluation, it is necessary to understand the structure's basic configuration, its condition, and certain basic material properties. Original construction documents, including the drawings and specifications, supplemented by damage survey reports, prepared in accordance with Chapter 4 of these *Recommended Criteria*, will provide sufficient data for the evaluation of most damaged steel moment-frame buildings, so long as the building was actually constructed in accordance with these documents. 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.

5.5.1 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 characteristic 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.

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 5-1 may be presumed. Alternatively, a limited program of material sample removal and testing may be conducted to confirm the likely grades of these materials.

Table 5-1 Default Material Specifications for Steel Moment-Frame 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 or E7024[1]
	1964-1994	E70T4 or E70T7[2]
	1994-1999	See note 3

Note 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.

Note 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.

Note 3   Following the 1994 Northridge earthquake, a wide range of weld filler metals were incorporated in steel moment-frame construction. Most of these filler metals had minimum ultimate tensile strengths of 70ksi and minimum rated toughness of 20 ft-lbs at -20°F. However, due to the variability of practice, particularly in the period 1994-1996, a limited sampling of weld metal in buildings constructed in this period is recommended to confirm these properties.

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 steel moment 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. 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 5-2, 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 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 toughness of the weld metal.

Table 5-2   Lower Bound and Expected Material Properties for Structural Steel Shapes of Various Grades[2]

Material Specification	Year of Construction	Yield Strength (ksi)		Tensile Strength (ksi)	
		Lower Bound	Expected	Lower Bound	Expected
ASTM A7, A373	Pre - 1960	30	35	60	70
ASTM A36	1961-1990				
Group 1		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	1970 – 1997				
Group 1		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 for material are mean minus two standard deviations from statistical data. Expected values for material are mean values from statistical data.
2.   For wide-flange shapes produced prior to 1997, indicated values are representative of material extracted from the web of the section.
3.   For material conforming to ASTM A992, the values for ASTM A572, Grade 50 may be used. No adjustment in values, per note 2, should be taken.
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.

Commentary: In general, great accuracy in the determination of the material properties of structural steel elements in steel moment-frame 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 toughness of the weld metal.

Weld Filler Metal

Welds in most steel moment-frame buildings constructed in the period 1964-1994 were made with the Flux Cored Arc Welding (FCAW) process, employing either E70T4 or E70T7 weld filler metal. This material generally has low notch toughness. 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 detailing prevalent until 1994. However, if weld metal with significant notch toughness (40 ft-lbs at service temperature) has been used in a building, even connections of the type typically constructed prior to the 1994 Northridge earthquake can provide some 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 low toughness 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 1964, sampling and testing of weld filler metal may indicate the presence of weld metal 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 notch 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 welds 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, beam-flange-to-column-flange joint). Samples should consist of both local base and weld metal, such that the composite strength of the connection can be assessed. If ductility is required at or near the weld, the design professional may conservatively assume, in lieu of testing, that no ductility is available.

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 5-1 and 5-2. 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 tensile strength 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 for 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 5-1 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.

5.6   Structural Performance Confidence Evaluation

The basic process of postearthquake evaluation, as contained in these procedures, is to develop a mathematical model of the damaged structure, and by performing structural analysis, to determine the likelihood that the building will resist ground shaking demands that can be anticipated to occur during the immediate postearthquake period, without collapse. The structural analysis is used to predict the value of various structural response parameters. These include:

- interstory drift, and

- axial forces on columns and column splices.

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

Once an analysis is performed, predicted demands are adjusted by two factors, an analysis uncertainty factor g_a that corrects the analytically predicted demands for bias and uncertainty inherent in the analysis technique, and a demand variability factor g that accounts for other sources of variability in structural response. These predicted demands are compared against acceptance criteria, which have also been factored, by resistance factors, f, to account for uncertainties and variation inherent in structural capacity prediction. If the factored demands are less than the factored acceptance criteria (capacities), then the structure is indicated to be capable of meeting the desired performance, with at least a mean level of confidence. If the factored demands exceed the factored acceptance criteria, then there is less than a mean level of confidence that the desired performance will be attained. Procedures are given to calculate the level of confidence, based on the ratio of factored demand to factored capacity. If the predicted level of confidence is inadequate, then the occupancy of the structure should be suspended until such time as the structure can be temporarily shored, and/or repaired, and a suitable level of confidence attained. In some cases it may be possible to improve the level of confidence with regard to the ability of a building to resist collapse by performing a more detailed analysis. More detailed and accurate analyses allow better understanding of the structure's probable behavior to be attained, resulting in modifications to the demand and capacity factors.

Table 5-3 summarizes the recommended posting condition for a building, as a function of the level of confidence determined with regard to the structure's ability to resist collapse for the level of ground shaking likely to be experienced in the immediate postearthquake period. Refer to Table 3-2 for information on the recommended actions related to each posting.

Table 5-3   Recommended Occupancy Actions, Based on Detailed Evaluation

Confidence Level of Attaining Collapse Prevention Performance	Recommended Occupancy Posting
50% or greater confidence of non-collapse	Green-1, Green-2, or Green-3, as appropriate
25% or greater confidence of non-collapse but less than 50%	Red-1
Less than 25% confidence of non-collapse	Red-2

Note: Refer to Table 3-2 for explanation of postings.

Four alternative analytical procedures are considered by these recommendations, for the prediction of building response parameters. These are the same basic procedures contained in *FEMA-273* and include the Linear Static Procedure (LSP), the Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP). Section 5.8 outlines these procedures in some detail. The reader is referred to *FEMA-273* for additional information and discussion.

> *Commentary: These Recommended Criteria adopt a Demand and Resistance Factor Design (DRFD) model for performance evaluation. This approach is similar to the Load and Resistance Factor Design (LRFD) approach adopted by the AISC design specifications except that the LRFD provisions are conducted on an element basis, rather than structural system basis, and the demands in these procedures can be drifts as well as forces and stresses. The purpose of this DRFD approach is to quantify the level of confidence associated with estimation of a damaged building's ability to provide Collapse Prevention performance given the probable ground shaking that may be experienced in the period immediately following a damaging earthquake, taken as one year.*
>
> *First, it is necessary to presume a hazard relationship for the site, during the immediate postearthquake period. Most strong earthquakes are followed by a large number of aftershocks, that decrease in frequency over time. Aftershocks typically occur on the same fault on which the main shock occurred, though, occasionally, an earthquake on a nearby fault has been triggered by the redistribution in crustal strains produced by the main shock. Aftershocks typically have less magnitude than the main shock, though there are some instances when an aftershock has actually exceeded the first shock. This forces a change in the naming of the two shocks, to foreshock and main shock. Generally, aftershock activity decays to insignificant levels within a period of approximately a year following the main event.*

The actual motion experienced at a site during aftershock activity is dependent on the size of the individual events, their location relative to the site and the faulting mechanism of the individual events. It is possible for aftershocks to produce stronger motion at a specific site than is experienced in the main earthquake. For the purposes of this guideline, it is assumed that the probable maximum intensity value for aftershock-induced ground shaking at the building site is the same as that experienced in the original damaging earthquake, that the variability in this intensity is normally distributed and that it has a coefficient of variation of 50%. While these assumptions may not be accurate for any specific earthquake, and will be conservative for most earthquakes, they present a reasonable planning scenario for postearthquake building safety assessments.

With the above assumptions in place, together with an estimate of the intensity of motion that actually occurred at the site during the damaging earthquake, it is possible to construct a hazard curve indicating the annual probability of exceeding ground motion of defined intensity at the site. For the purposes of evaluations conducted in accordance with these Recommended Criteria, the hazard curve is plotted as a function of the spectral response acceleration, S_a, at the fundamental period of the damaged building, and the annual probability of exceedance for these accelerations. Figure 5-1 presents such a hazard curve, with spectral response acceleration normalized to the value actually thought to have been experienced in the first damaging earthquake. The primary parameters of importance from this hazard curve are the slope of the curve evaluated at S_a and the value of S_a itself.
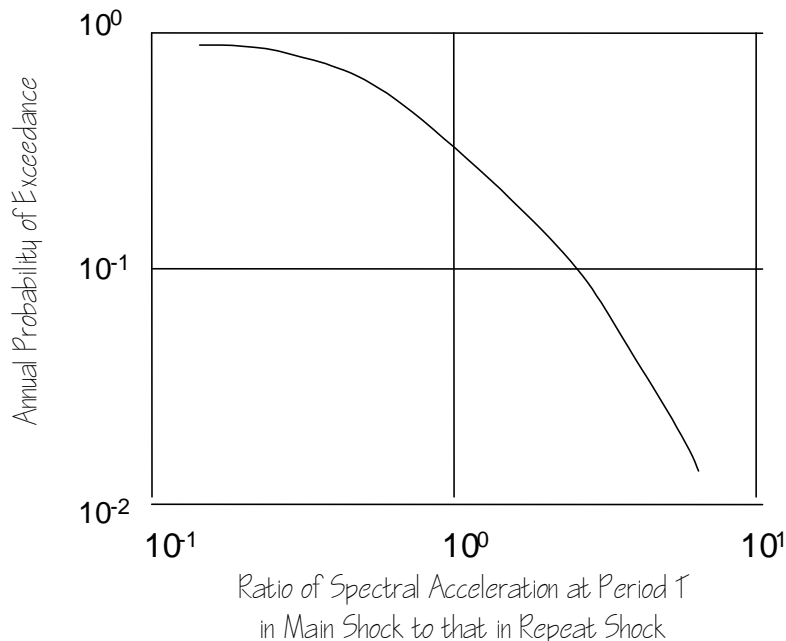
Figure 5-1   Presumed Postearthquake Hazard Curve

Using the S_a value estimated to have been experienced during the first damaging earthquake, a structural analysis is performed to determine the maximum interstory drift demand for the damaged structure under a repeat of that event, as well as the maximum axial forces on critical columns. These demands are factored by a demand variability factor $\boldsymbol{g}$ to account for the variation associated with estimation of the character of the ground motion and its effect on structural response, and an analysis uncertainty factor $\boldsymbol{g_a}$ to account for the uncertainty and bias inherent in the selected analytical approach.

The factored demand, $\boldsymbol{g_a g}D$ calculated from the analysis represents a mean estimate of the probable maximum demand during the immediate postearthquake period, given the assumed distribution of ground shaking during this period, as represented by the assumed hazard curve.

These Recommended Criteria also specify median estimates of capacity for individual elements and the global structure. These capacities are dependent on frame and connection configuration. In addition to capacities, capacity reduction, or resistance, factors $\boldsymbol{f}$ that adjust the estimated capacity of the structure to a mean value are also provided.

Once the factored demands and capacities are determined, a factored-demand-to capacity parameter λ is calculated from the equation:

$$\boldsymbol{l} = \frac{\boldsymbol{g_a g}D}{\boldsymbol{f}C} \qquad\qquad (5\text{-}1)$$

where D and C are respectively, the demand and capacity. The value of $\boldsymbol{l}$ is then used directly to determine an associated confidence level for the desired performance, based on tabulated values related to the uncertainty inherent in the estimation of the building's demands and capacities. Values of $\boldsymbol{l}$ less than 1.0 indicate greater than mean confidence of achieving the desired performance. Values greater than 1.0 indicate less than mean confidence.

5.7   Ground Motion Representation

The damaged structure should be analyzed for ground shaking demands representative of those that caused the initial damage. Ground shaking demands should be represented in the form of a 5% damped elastic response spectrum or with ground acceleration time-histories, compatible with this spectrum as required by the selected analytical procedure. Ground shaking demands may be determined by one of the following approaches.

5.7.1   Instrumental Recordings

When an actual recording of the ground shaking that caused the damage, obtained from the building site, or a nearby site with similar conditions is available, this may be used directly to

perform analyses of the damaged structure. The ground acceleration time-history should be converted into a smoothed, 5% damped response spectrum, similar in form to the generalized response spectrum described in *FEMA-273*, and completely enveloping the actual response spectrum obtained for the acceleration record over the period range $0.5T$ to $2.0T$, where T is the computed fundamental period of the damaged structure. If the selected analytical procedure is response history analysis, a suite of accelerograms constructed in accordance with the recommendations of *FEMA-273* and matched to the spectrum, should be used, one of which should be the actual site recording.

> *Commentary: The best possible estimate of ground shaking experienced at a site consists of actual ground motion recordings obtained from a free-field instrument located at the building site. Free field instruments are preferable to instruments located within the building or another structure as they will not be influenced by structural response effects.*
>
> *Even in zones of high seismicity, few buildings have strong motion instrumentation, so it is highly unlikely that such records will be available for most buildings. Recordings of ground shaking obtained from other nearby sites may be used providing that the site of the instrument is at a comparable distance and azimuth to the fault rupture as the damaged building, and providing that site soil conditions are reasonably similar. Site soil conditions may be considered to be reasonably similar if they are of the same site class, as defined in FEMA-302, the NEHRP Recommended Provisions.*
>
> *The intent of postearthquake analyses is not to evaluate the damaged building's response for the actual ground shaking that caused the original damage, but rather to evaluate this response for ground shaking likely to be experienced in the immediate postearthquake period. As previously discussed, this is assumed to be similar, though not identical to that which caused the original damage. For this reason, response spectra obtained from actual ground motion recordings are smoothed, to approximate a standard Newmark and Hall spectrum, as described in FEMA-273.*

5.7.2   Estimated Ground Motion

When instrumental recordings of the damaging ground shaking, as described in Section 5.7.1 are not available, an estimated response spectrum for this ground shaking should be constructed. These spectra should be constructed as recommended by *FEMA 273* except that rather than using mapped values for the parameters S_S and S_1, these parameters should be calculated using standard attenuation relationships and appropriate estimates of the magnitude of the damage causing event, its distance from the building site, the site soil characteristics, faulting mechanism and other parameters required by the attenuation equation. Alternatively, these parameters may be estimated based on available recordings of ground shaking from the damage causing event.

Acceleration time histories, if required, should be constructed in accordance with the recommendations of *FEMA-273*.

5.8   Analytical Procedures

In order to evaluate the performance of a damaged steel moment-frame structure it is necessary to construct a mathematical model of the damaged 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 procedures for selecting an appropriate analysis procedure and for modeling. General requirements for the mathematical model are presented in Section 5.9.

Four alternative analytical procedures are available. The basic 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 basic procedures are:

- Linear static procedure – an equivalent lateral force technique, similar, but not identical to that contained in the 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 monotonically increasing lateral loading is evaluated using a series of incremental elastic analyses of structural models that are sequentially degraded to represent the effects of structural nonlinearity.

- Nonlinear dynamic procedure – a nonlinear dynamic analysis procedure in which the response of a structure to a suite of ground motion time 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 postearthquake assessment process is to predict the values of key response parameters, that are indicative of the structure's performance, when it is subjected to ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy (appropriate level of confidence of achieving desired performance) using the basic approach outlined in Section 5.6.*
>
> *Analyses conducted in these procedures take a markedly different approach than those used in the standard design process under the building code requirements. 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 under the design ground motion.*

The ability of these procedures to estimate reliably the probable performance of the structure is dependent on the ability of the analysis 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 less accurately accounts for the dynamic characteristics of the structure. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behavior, particularly if they are irregular. However, it does not accurately account for the effects of higher mode response. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the real structure in response to ground motion; however, there are considerable uncertainties associated even with the values of the response parameters predicted by this technique.

5.8.1 Procedure Selection

Table 5-4 indicates the recommended analysis procedures for various conditions of structural regularity and dynamic properties. Note that structural regularity in these procedures is as determined in *FEMA-273*, rather than as alternatively defined in the building codes. Both regularity and dynamic properties shall be as determined for the building in the damaged state.

5.8.2 Linear Static Procedure (LSP)

5.8.2.1   Basis of the Procedure

Linear static procedure analysis of damaged steel moment-frame structures shall be conducted in accordance with the *FEMA-273 Guidelines,* except as specifically noted herein. In this procedure, lateral forces are applied to the masses of the structure, and deflections and component forces under this applied loading is 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 5.10.

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

Table 5-4   Selection Criteria for Analysis Procedure to Achieve Collapse Prevention

Structural Characteristics			Analytical Procedure			
Fundamental Period, T	Regularity	Ratio of Column to Beam Strength	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
$T \leq 3.5T_s$[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.   2.A structure qualifies as having a strong column condition if, at every floor level, the quantity $\sum M_{pr_c} / \sum M_{pr_b}$ 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, respectively, that participate in the moment-resisting framing in a given direction of structural response.

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. Earthquake demands for the LSP are represented by the static lateral forces whose sum is equal to the pseudo lateral load. 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 displacement amplitudes approximating maximum displacements that are expected during the ground shaking under evaluation. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during this ground shaking. If the building responds inelastically to the earthquake ground shaking, as will commonly be the case, when ground shaking is severe, the internal forces that would develop in the

yielding building will be less than the internal forces calculated on an elastic basis.

In addition to global structural drift, the collapse of steel moment-frame structures is closely related to inelastic deformation demands on the various elements that comprise the structure, such as plastic rotation demands on beam-column assemblies and tensile demands on column splices. Linear analysis methods do not permit direct evaluation of such demands. However, through a series of analytical evaluations of typical buildings for a number of earthquake records, it has been possible to develop statistical correlation between the interstory drift demands predicted by a linear analysis and the actual inelastic deformation demands determined by more accurate nonlinear methods. These correlation relationships are reasonably valid for regular structures, using the definitions of regularity contained in FEMA-273. Thus, the performance evaluation process using Linear Static Procedures (LSP) consists of performing the LSP analysis to determine an estimate of interstory drift demands, adjustment of these demands with the demand factors, g and g_a, and comparison with tabulated interstory drift capacities.

Although performance of steel moment-frame structures is closely related to interstory drift demand, there are some failure mechanisms, notably, 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 LSP analysis is performed for structures that respond in an inelastic manner, column axial demands should be estimated using a supplementary plastic analysis approach.

Two basic assumptions apply in this evaluation approach. First, that the distribution of deformations predicted by an elastic analysis is similar to that which will occur in actual nonlinear response; second, that the ratio of computed strength demands from an elastic analysis to yield capacities is a relative indication of the inelastic ductility demand on the element. These assumptions are never particularly accurate but become quite inaccurate for structures that are highly irregular and experience large inelastic demands.

Most damaged structures will behave in a more non-linear manner than will undamaged structures, even when subjected to relatively low levels of ground shaking. Beam-column connections with fractures at the bottom flange of the beam, for example, will behave much like undamaged, fully restrained joints when loaded such that the fractured flange is in compression, and will behave much like pinned joints when loading produces tension at the bottom flange. Such behavior can not be accurately reflected in elastic analysis. In order to minimize the potential for analysis inaccuracies to result in overly optimistic estimates of the actual response of a damaged structure, these Recommended Criteria suggest

*what are believed to be conservative modeling assumptions for damaged framing
elements. However, the uncertainties inherent in the use of linear methods to
model highly damaged structures are so large that it is recommended they not be
used for this purpose.*

5.8.2.2   Modeling and Analysis Considerations

When damage results in a structure having different stiffness and strength for loading applied
positively along one of the principal axes than it does for loading applied negatively, a separate
model shall be developed and analysis performed for each direction of loading.

5.8.2.2.1 Period Determination

A fundamental period shall be calculated for each of two orthogonal directions of building
response, using standard methods of modal analysis. The model used for this purpose should
account for the damage sustained. Where damage results in a significantly different stiffness in
the positive direction of response relative to the negative direction, separate analyses shall be
performed for each such response direction.

> *Commentary: Modal analysis of a model of the building that includes
> representation of the structural damage is required to determine the building's
> period. This is because approximate formulae, used, for example, in FEMA-302
> for this purpose, may be inaccurate for damaged structures.*

5.8.2.3   Determination of Actions and Deformations

5.8.2.3.1 Pseudo Lateral Load

A pseudo lateral load, given by Equation 5-2, shall be independently calculated for each of
two orthogonal directions of building response, and applied to a mathematical model of the
building structure. Where damage results in a significantly different stiffness or strength in the
positive direction of loading than in the negative direction, separate analyses shall be performed
for each such response direction.

$$V = C_1 C_2 C_3 S_a W \tag{5-2}$$

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 where the fundamental period of response of the structure, T, is greater than T_s
and shall be taken as having a value of 2.0 where 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 = characteristic period of the response spectrum, defined as the period associated with the transition from the constant spectral response acceleration segment of the spectrum to the constant spectral response velocity segment of the spectrum as defined in *FEMA-302*.

C_2 = modification factor to represent the effect of hysteretic pinching on 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 5-5 or alternatively, shall be calculated from the equation:

$$C_3 = 1 + \frac{5(q_i - 0.1)}{T} \geq 1.0 \qquad (5\text{-}3)$$

where:

q_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.

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

- in storage and warehouse occupancies, a minimum of 25% of the floor live load,

- the actual partition weight or minimum weight of 10 psf of floor area, whichever is greater,

- the applicable snow load – see *FEMA-302,* and

- the total weight of permanent equipment and furnishings.

Table 5-5   Modification Factors C_3 for Linear Static Procedure

	C_3
Ductile fully-restrained connections	1.2
Brittle fully-restrained connections	1.4

Notes:

- Ductile connections are those connections capable of sustaining at least 0.03 radians, median, plastic rotation capacity without fracturing or sustaining significant loss of strength.
- Brittle connections are those connections not qualifying as ductile. Typical unreinforced moment-resisting connections in which beam flanges are CJP welded to the column, using low notch toughness weld filler metal shall be considered brittle unless laboratory data are available to substantiate their capability of behaving as indicated for ductile connections.

Commentary: The pseudo lateral force, when distributed over the height of the linearly-elastic analysis 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 5-2 may be significantly larger than the actual strength of the structure to resist this force. The acceptance evaluation procedures in Section 5.10 are developed to take this into account.

The values of the C_3 coefficient contained in Table 5-5 are conservative for most structures, and will generally result in calculation of an unduly low level of confidence. Use of Equation 5-3 to calculate C_3 is one way to improve calculated confidence without extensive additional effort, and is recommended.

5.8.2.3.2   Vertical Distribution of Seismic Forces

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

5.8.2.3.3   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.

5.8.2.3.4   Determination of Interstory Drift

Interstory drifts shall be calculated using lateral loads in accordance with this section. Factored interstory drift demands, $\boldsymbol{g_a g d_i}$, at each story "*i*", shall be determined by applying the appropriate demand variability factor $\boldsymbol{g}$ and analytical procedure uncertainty factor $\boldsymbol{g_a}$ obtained from Section 5.10.

5.8.2.3.5 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 $\boldsymbol{g}_a$ and demand variability factor $\boldsymbol{g}$ obtained in Section 5.10.3. 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:

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

where:

$\left(\sum_{i=x}^{n}\dfrac{M_{pe}}{L}\right)_L$ = 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

$\left(\sum_{i=x}^{n}\dfrac{M_{pe}}{L}\right)_R$ = 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 an orthogonal direction of response.

> *Commentary: When determining axial demands on columns using Equation 5-4, the value of the quantity $2\dfrac{M_{pe}}{L}$ may be reduced for beams with fractured connections, when the direction of response of the structure is such that loading tends to open the fracture in tension. For such loading, the M_{pe} value at the fracture may be reduced to 30% of the nominal value calculated for the beam. Thus, if a beam has a fracture at one end, rather than using the value $2\dfrac{M_{pe}}{L}$ for the axial load contribution from this beam, the quantity $1.3\dfrac{M_{pe}}{L}$ could be used, when loading tends to place this fracture in tension. If a beam has fractures at*

both ends that open in tension simultaneously, the contribution for this beam

could be reduced to $0.6\dfrac{M_{pe}}{L}$

5.8.3   Linear Dynamic Procedure (LDP)

5.8.3.1   Basis of the Procedure

Linear dynamic procedure analysis of damaged steel moment-frame structures should generally be conducted in accordance with the *FEMA-273 Guidelines*, except as specifically noted herein. Coefficients C_1, C_2, and C_3 should be taken as indicated in Section 5.8.2.3.1 of these *Recommended Criteria*.

Estimates of interstory drift and column axial demands shall be evaluated using the applicable procedures of Section 5.10. Calculated displacements and column axial demands are factored by the applicable analytical uncertainty factor $\boldsymbol{g}_a$ and demand variability factor $\boldsymbol{g}$, obtained from Section 5.10, and compared with factored capacity values. 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 the previous section. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, it is somewhat more accurate. Coefficients C_1, C_2, and C_3, which account in an approximate manner for the differences between elastic predictions of drift response and inelastic behavior are the same as for the linear static method. Under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, dynamic analysis. Note that although the LDP is more accurate than the LSP for analysis purposes, it can still be quite inaccurate when applied to heavily damaged structures and should be used with caution.*
>
> *The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the Linear Static Procedure (LSP). The main exception is that the response calculations are carried out using modal spectral analysis. Modal spectral analysis is carried out using linearly-elastic response spectra that are not modified to account for anticipated nonlinear response. As with the LSP, it is expected that the LDP will produce displacements that are approximately correct, but will produce internal forces that exceed those that would be obtained in a yielding building.*

5.8.3.2   Modeling and Analysis Considerations

5.8.3.2.1   General

The Linear Dynamic Procedure (LDP) should conform to the criteria of this section. The analysis should be based on appropriate characterization of the ground motion, as described in Section 5.7. The LDP should conform to the criteria in Section 3.3.2.2 of *FEMA 273*. 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 should 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 5.8.3.3, should be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root of 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 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. Where damage to the structure results in unsymmetrical response in either the A or B directions, then independent analyses should be performed with elements modeled to represent the behavior of the structure when pushed in the positive and negative senses along either the A or B directions.

5.8.3.3   Determination of Actions and Deformations

5.8.3.3.1   Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by mulitplying the results of the response spectrum analysis by the product of the modification factors, C_1, C_2, and C_3 defined in Section 5.8.2.3 and by the analytical procedure uncertainty factor g_a and demand variability factor g obtained from Section 5.10.

5.8.3.3.2   Determination of Column Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces, as given in Section 5.8.2.3.5, by the applicable analysis uncertainty factor γ_a and demand variability factor γ obtained from Section 5.10.3.

5.8.4   Nonlinear Static Procedure (NSP)

5.8.4.1   Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the inelastic material and geometric response of the damaged structure 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 damaged 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, in response to the ground shaking anticipated during the immediate postearthquake period. The target displacement shall be calculated by the procedure presented in Section 5.8.4.3.1. Because the mathematical model accounts directly for effects of material and geometric inelastic response, the calculated internal forces will be reasonable approximations of those expected during the anticipated ground shaking, presuming that an appropriate pattern of loading has been applied.

Results of the Nonlinear Static Procedure (NSP) are to be evaluated using the applicable acceptance criteria of Section 5.10. Calculated interstory drifts and column and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

5.8.4.2   Modeling and Analysis Considerations

5.8.4.2.1 General

In the context of these procedures, the Nonlinear Static Procedure (NSP) involves the monotonic application of lateral forces, or displacements, to a nonlinear mathematical model of a building, until the displacement of the 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, such as often occurs in damaged buildings, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging to the target displacement d_t, given by Equation 3-11 of *FEMA 273*. Postearthquake assessment shall be based on those column forces and interstory drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement d_t.

Gravity loads should be applied to appropriate components and elements of the mathematical model during the NSP. The loads and load combinations should be as follows:

1. 100% of computed dead loads and permanent live loads should be applied to the model.

2. 25% of transient floor live loads should be applied to the model, except in warehouse and storage occupancies, where the percentage of live load used in the analysis should 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.

The modeling and analysis considerations set forth in Section 5.9 should apply to the NDP unless the alternative considerations presented below are applied.

> *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.*

5.8.4.2.2 Control Node

The NSP requires definition of the control node in a building. These procedures 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. The displacement of the control node is compared with the target displacement—a displacement that characterizes the effects of earthquake shaking.

5.8.4.2.3 Lateral Load Patterns

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

5.8.4.2.4 Period Determination

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

5.8.4.2.5 Analysis of Three-Dimensional Models

Static lateral forces should 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 5.8.4.3 of these *Recommended Criteria.*.

5.8.4.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2, or the orthogonal A and B directions) 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.

5.8.4.3 Determination of Actions and Deformations

5.8.4.3.1 Target Displacement

The target displacement d_t for buildings with rigid diaphragms at each floor level shall be estimated using the procedures of Section 3.3.3.3 of FEMA-273. Actions and deformations corresponding to the control node displacement equal to the target displacement shall be used for evaluation in accordance with Section 5.10.

5.8.4.3.2 Diaphragms

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

5.8.4.3.3 Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum interstory drift calculated at the target displacement by the analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.2.

5.8.4.3.4 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 analytical uncertainty factor g_a and demand variability factor g from Section 5.10.3.

5.8.5 Nonlinear Dynamic Procedure (NDP)

5.8.5.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), design 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 of 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 ground motion time-histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during ground shaking.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 5.10. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values.

5.8.5.2   Modeling and Analysis Assumptions

5.8.5.2.1   General

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

5.8.5.2.2   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 ground motion described in Section 5.7 of these *Recommended Criteria*. A minimum of three pairs of ground motion records should be used. Each pair should consist of two orthogonal components of ground motion records.

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*.

5.8.5.3   Determination of Actions and Deformations

5.8.5.3.1   Response Quantities

Response quantities should 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) should 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.

5.8.5.3.2   Factored Interstory Drift Demand

Factored interstory drift demand shall be obtained by multiplying the maximum of the interstory drifts calculated in accordance with Section 5.8.5.3.1 by the analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.2.

5.8.5.3.3   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 5.8.5.3.1 by the applicable analytical uncertainty factor g_a and demand variability factor g obtained from Section 5.10.3.

5.9   Mathematical Modeling

5.9.1   Modeling Approach

In general, a damaged steel frame building should be modeled, analyzed and designed as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate design information for regular, symmetric structures and structures with flexible diaphragms, three-dimensional mathematical models should be used for analysis and design of buildings with plan irregularity as defined by *FEMA-302*.

Two-dimensional modeling, analysis, and evaluation of buildings with stiff or rigid diaphragms is acceptable if torsional effects are either sufficiently small to be ignored or indirectly captured.

Vertical lines of moment frames with flexible diaphragms may be individually modeled, analyzed, and evaluated as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

If linear or static analysis methods are used, it may be necessary to build separate models to simulate the behavior of the structure to ground shaking demands in the positive and negative response directions, to account for the differing effects of damage in each direction of response.

> *Commentary: An inherent assumption of linear seismic analysis is that the structure will exhibit the same stiffness and distribution of stresses regardless of whether loads are positively or negatively loaded. However, damage tends to create non-symmetrical conditions in structures. For example, fracture damage at the bottom flange of a beam will result in a substantial reduction in the connection's stiffness under one direction of loading, but will have negligible effect for the reverse direction of loading. In order to capture this behavior using linear analysis approaches, it is necessary to build two separate models, one in which the damage is effective and one in which the damage is not, to simulate the separate response in each direction of loading. A similar approach is required for nonlinear static analysis, in that the nonlinear behavior will be different, depending on the direction of loading. Only nonlinear dynamic analysis is capable of accurately simulating the effects of such damage with a single analytical model.*

5.9.2   Model Configuration

The analytical model should include all frames capable of providing non-negligible stiffness for the structure, whether or not intended by the original design to participate in the structure's lateral force resistance. The model should accurately account for any damage sustained by the structure. Refer to Section 5.9.11 for procedures on modeling damaged connections.

> *Commentary: Gravity framing, in which beams are connected to columns with either clip angels or single clip plates can provide significant secondary stiffness*

to a structure and should in general be modeled when performing postearthquake assessment analyses. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams which act as struts, and the diaphragms. As a secondary effect, the relatively small rigidity provided by the gravity connections provides some additional overall frame stiffness.

5.9.3   Horizontal Torsion

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

- 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

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

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

5.9.4   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.*

5.9.5 Diaphragms

Floor and roof diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic-force-resisting system. Development of the mathematical model should reflect the stiffness of the diaphragms. 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.

5.9.6 *P-Δ* Effects

P-Δ effects, caused by gravity loads acting on the displaced configuration of the structure, may be critical in the seismic performance of steel moment-frame structures, particularly for damaged structures that may have significant permanent lateral offset as part of the damage.

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 $\mathbf{y}_i$ should be calculated for each direction of response, as follows:

$$\mathbf{y}_i = \frac{P_i \mathbf{d}_i}{V_{yi} h_i}$$

(5–5)

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

d_i =   lateral drift in story i, including any permanent drift, 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 y_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity y_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by *P-**D*** effects. When y_i in a story exceeds 0.3, the structure shall be considered unstable, unless a detailed global stability capacity evaluation for the structure, considering *P-**D*** effects, is conducted in accordance with the procedures 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, provided in Section 5.10, and the corresponding resistance factors, were computed considering P-**D** effects (FEMA-355F). For a given structure, it is believed that if the value of **y** is less than 0.3 the effects of P-**D** have been adequately considered by these general procedures. For values of **y** greater than this limit the statistics on frame interstory drift capacities contained in Section 5.10 are inappropriate. For such frames explicit determination of interstory drift capacities, considering P-**D** 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:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pG\,j}}{h_i} \qquad (5\text{-}6)$$

where:

M_{pGj} = the plastic moment capacity of each girder "j" participating in the moment resisting framing at the floor level on top of the story. For girders with damaged connections, the quantity $2M_{pGi}$ should be taken as the sum of the plastic moment capacities at each end of the girder, accounting for the effect of damage on connection capacity as recommended in Section 5.9.11.

$n =$ *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 meet the strong-column -weak-beam criterion, the plastic story shear quantity V_{yi} may be calculated from the equation:

$$V_{yi} = \frac{2\sum_{k=1}^{n} M_{pC_k}}{h_i} \tag{5-7}$$

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.

5.9.7 Elastic Framing Properties

The complete axial area of rolled shapes should be used. For built-up sections, the effective area should be reduced if adequate load transfer mechanisms are not available. For elements fully encased in concrete, the axial stiffness may be calculated assuming full composite action if most of the concrete may be expected to remain after additional ground shaking. Composite action may not be assumed for strength unless adequate load transfer and ductility of the concrete can be assured.

The shear area of the elements should be based on standard engineering procedures. The comments above regarding built-up section, concrete encased elements, and composite floor beam and slab, apply.

The calculation of rotational stiffness of steel beams and columns in bare steel frames should follow standard engineering procedures. For components encased in concrete, the stiffness shall include composite action, but the width of the composite section should be taken as equal to the width of the flanges of the steel member and should not include parts of the adjoining floor slab, unless there is an adequate and identifiable shear transfer mechanism between the concrete and the steel.

5.9.8 Nonlinear Framing Properties

The elastic component properties, should be computed as outlined in Section 5.9.7. Appropriate nonlinear moment-curvature and interaction relationships should be used for beams and beam-columns to represent the effects of plastification.

5.9.9 Verification of Analysis Assumptions

Each component should be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length. Further, each component should be evaluated by rational analysis for adequate postearthquake residual gravity load capacity, considering reduction of stiffness caused by earthquake damage to the structure.

Where moments in horizontally-spanning primary components, due to the gravity loads, exceed 50% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than components ends should be specifically investigated by comparing flexural actions with expected component strengths. Modeling should account for formation of flexural plastic hinges away from component ends when this is likely to occur.

5.9.10 Undamaged Connection Modeling

Undamaged connections should be modeled in accordance with the following procedures.

5.9.10.1 Fully Restrained Connections

Framing connected with typical welded fully restrained moment-resisting connections, such as shown in Figure 5-2, should be modeled as indicated herein.
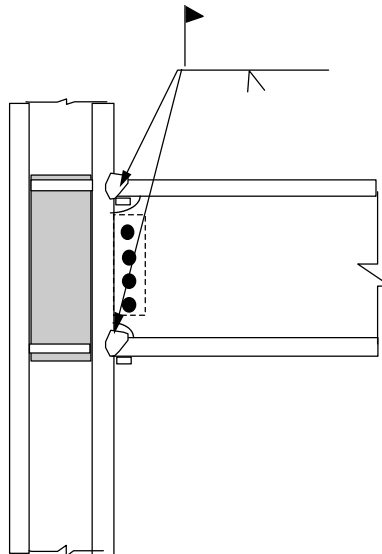
Figure 5-2   Welded Unreinforced Fully Restrained Connection (pre-1994)

5.9.10.1.1 Linear Modeling

Undamaged fully-restrained connections should be modeled using the gross cross section properties and assuming rigid attachment between the beams and columns. Modeling may use either center-line-to-center-line dimensions for beams and columns, or alternatively, rigid or flexible column panel zones may be modeled to offset the ends of the beams and columns from the intersection of the center lines of these members. Rigid offsets, used to represent the panel

zone, should not exceed 80% of the dimension of the actual panel zone. Panel zone flexibility may be directly considered by adding a panel zone element to the model.

5.9.10.1.2 Nonlinear Modeling

Prior to developing a mathematical model for nonlinear analysis of beam-column assemblies with welded unreinforced fully restrained moment-resisting connections, an analysis should be conducted to determine the controlling yield mechanism for the assembly. This may consist of flexural yielding of the beam at the face of the column, flexural yielding of the column at the top and/or bottom of the panel zone; shear yielding of the panel zone itself, or a combination of these mechanisms. Elements capable of simulating the nonlinear behaviors indicated in these analyses should be implemented in the model. Regardless of whether or not panel zones are anticipated to yield, panel zones should be explicitly modeled. If calculations indicate that panel zones are unlikely to yield in shear, panel zones may be modeled as rigid links. If significant yielding is indicated to occur, a suitable element that models this behavior should be used. Expected yield strengths F_{ye} should be used for all nonlinear elements to indicate the expected onset of nonlinear behavior. Flexural strain hardening of beams and columns should be taken as 5% of the elastic stiffness, unless specific data indicates a more appropriate value. Panel zones may be assumed to strain harden at 20% of their elastic stiffness.

5.9.10.2   Simple Shear Tab Connections

This section presents modeling guidelines for 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. Figure 5-3 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. A concrete floor slab, or slab on metal deck may be present at the top flange of the beam.

Major Axis of Column   Minor Axis of Column
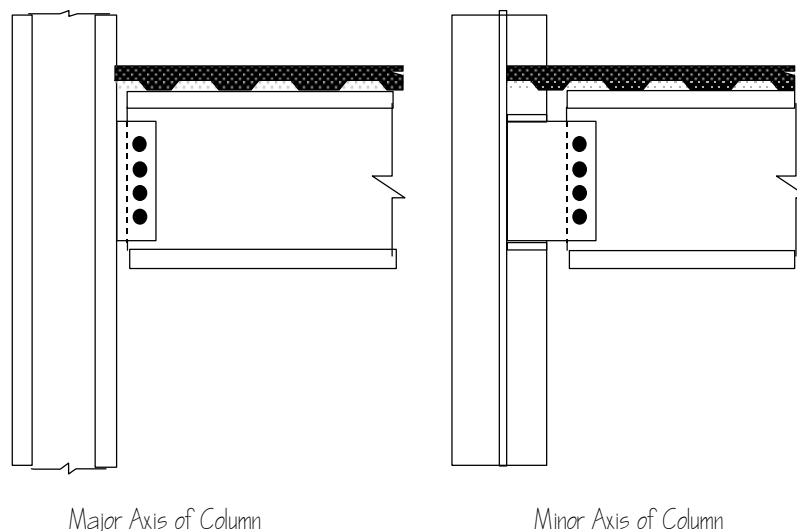
Figure 5-3   Typical Simple Shear Tab Connection with Slab

5.9.10.2.1 Modeling Guidelines - Linear Analysis

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

$$K_q = 28000(d_{bg} - 5.6)$$ (5-8)

where d_{bg} is the depth of the bolt group, measured center-line-to-center-line of the outermost bolts, in inches and K_q is in units of k-inches per radian. In lieu of explicit modeling of the connection, beams that frame into columns with simple shear tab connections may be modeled with an equivalent rigidity, EI_{eq} taken as:

$$EI_{eq} = \frac{1}{\dfrac{6h}{l_b^2 K_q} + \dfrac{1}{EI_b}}$$ (5-9)

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, $(inches)^4$
$l_b =$ the beam span center to center of columns, inches

Commentary: The presence of gravity framing, utilizing shear tab connectors, can provide substantial stiffening to the steel moment-frame system provided as the basic lateral force resisting system. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams, which act as struts, and the diaphragms. The flexural restraint on the columns represented by the spring stiffness given by Equations 5-8 and 5-9 is a secondary effect but can provide stability for frames at large displacements.

5.9.10.2.2 Modeling Guidelines - 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 5-8. 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.

5.9.11 Damage Modeling

This section presents procedures for modeling various conditions of damage. In general, damage results in anisotropic frame behavior with affected framing exhibiting different hysteretic

properties for loading in a positive direction, than it does for loading in the reverse direction.
Except for nonlinear dynamic analyses, it is generally necessary to utilize multiple models to
represent these different behaviors, with loading applied in an appropriate direction for each
model.

5.9.11.1 Fully Restrained (FR) Connection Damage

Damaged type FR connections should be modeled in accordance with the guidelines of this
section. Refer to Chapter 2 for detailed descriptions of the various damage conditions.

- Connections with any one of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6 damage
  at the bottom flange only or the top flange only may be modeled as undamaged for loading
  conditions in which lateral loading will tend to place the fractured surfaces into compression.
  For loading conditions in which the fracture is placed into tension, the connection should be
  modeled as an undamaged simple shear tab connection, per Section 5.9.10.2.

- Connections with any combination of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6
  damage at the top and bottom flanges should be modeled as an undamaged simple shear tab
  connection, per Section 5.9.10.2 for loading in either direction.

- If any of the above conditions is present in combination with shear tab damage, types S1, S2,
  S3, S4, S5, or S6, then the connection should be modeled as a simple pin connection for both
  directions of loading.

- Connections with type P7 damage should be modeled as follows. The beam and column
  above the diagonal plane formed by the fracture should be assumed to be rigidly restrained to
  each other. The beam and column below the diagonal plane formed by the fracture should
  similarly be assumed to be rigidly restrained to each other. The two assemblies consisting of
  the rigidly restrained beam-column joint above and below the diagonal fracture should be
  assumed to be unconnected for loading that places the fracture into tension and should be
  assumed to be connected to each other with a "pin" for conditions of loading that place the
  fracture into compression.

- Connections with type P9 damage and oriented as indicated in Figure 5-4 should be modeled
  with the beams and columns below the fracture surface assumed to be rigidly connected. The
  column above the fracture surface should be assumed to be unconnected for loading that
  places the column into tension and should be assumed to be "pin" connected for loading that
  places the column into compression. If the orientation of type P9 damage is opposite that
  shown in Figure 5-4, then the instructions above for "top" and "bottom" columns should be
  reversed.

5.9.11.2 Column Damage

- If a column has type C1 or C3 damage in any flange, the column should be modeled as if
  having a pinned connection (unrestrained for rotation) at that location for loading conditions
  that induce tension across the fracture. The column may be modeled as undamaged for
  loading conditions that produce compression across the fracture surfaces.
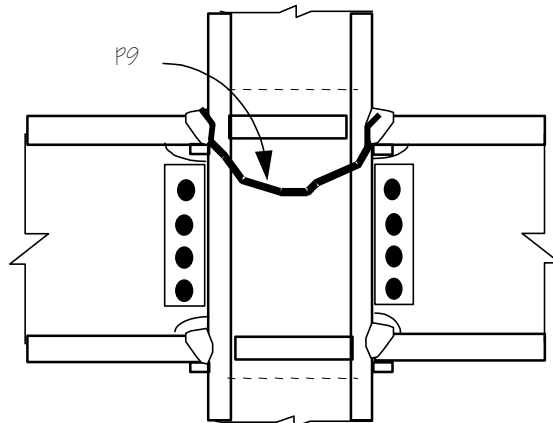
Figure 5-4   Type P9 Panel Zone Damage

- If a column has type C7, column splice fracture damage, it should be assumed to be unconnected across the splice for load conditions that place the column in tension and should be assumed to have a "pin" connection for load conditions that place the column in compression.

- If a column has type C6, buckling damage of a flange, the buckled length of the column should be modeled with a separate element with flexural properties calculated using only 30% of the section of the buckled element.

5.9.11.3   Beam Damage

- Beams that have lateral torsional buckling, type G8, should be modeled with a flexural pin at the center of the buckled region.

- Beams that have type G1, buckling damage of a flange should be modeled with the buckled length of the beam represented by a separate element with flexural properties calculated using only 30% of the section of the buckled flange.

5.9.11.4   Other Damage

Damage other than indicated in Sections 5.9.11.1, 5.9.11.2, or 5.9.11.3 need not be modeled unless in the judgment of the engineer, it results in significant alteration of the stiffness or load distribution at the connection. In such cases, the engineer should use judgment in developing the model such that it accurately reflects the behavior of the damaged elements.

5.10   Acceptance Criteria and Confidence Evaluation

A level of confidence with regard to the building's ability to provide Collapse Prevention performance for a repeat of the original damaging ground motion should be determined. Each of the parameters in Table 5-6 must be independently evaluated, using the procedures of Section

5.10.1 and the parameters and acceptance criteria of Sections 5.10.2, 5.10.3, and 5.10.4. The controlling parameter is that which results in the calculation of the lowest confidence for building performance.

Table 5-6   Performance Parameters Requiring Evaluation of Confidence

Parameter	Discussion
Interstory drift	The maximum interstory drift computed for any story of the structure shall be evaluated for global and local behaviors. Refer to Section 5.10.2
Column axial load	The adequacy of each column to withstand the calculated maximum compressive demand for that column shall be evaluated. Refer to Section 5.10.3
Column splice tension	The adequacy of column splices to withstand calculated maximum tensile demands for the column shall be evaluated. Refer to Section 5.10.4

5.10.1 Factored-Demand-to-Capacity Ratio

Confidence level is determined by first evaluating the factored-demand-to-capacity ratio λ given by the equation:

$$\lambda = \frac{\gamma_a \gamma D}{\phi C} \tag{5-10}$$

where:

$C =$ capacity of the structure, as indicated in Sections 5.10.2, 5.10.3, and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

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

$\gamma =$ 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 5.10.2, 5.10.3, and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

$\gamma_a =$ 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 5.10.2, 5.10.3 and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively,

$\phi =$ 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 5.10.2, 5.10.3 and 5.10.4, for interstory drift demand, column compressive demand and column splice tensile demand, respectively, and

λ =   a confidence index parameter from which a level of confidence can be obtained. See Table 5-7.

Factored-demand-to-capacity ratio λ shall be calculated using Equation 5-10 for each of the performance parameters indicated in Table 5-6, which also references the appropriate section of this document where the various parameters, g_a, g, and f required to perform this evaluation may be found. These referenced Sections 5.10.2, 5.10.3, and 5.10.4 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 structural framing system (OMF or SMF), the type of analytical procedure employed, and the performance level being evaluated. Table 5-7 indicates the level of confidence associated with various values of the factored-demand-to-capacity ratio λ calculated using Equation 5-10, for various values of the uncertainty parameter β_{UT}. Linear interpolation between the values given in Table 5-7 may be used for intermediate values of factored-demand-to-capacity ratio λ and uncertainty β_{UT}.

Table 5-7   Factored-Demand-to-Capacity Ratios λ and Uncertainty β_{UT}, for Specific Confidence Levels

Uncertainty Parameter β_{UT}	Factored-Demand-to-Capacity Ratios λ										
0.2	1.43	1.31	1.23	1.16	1.11	1.05	0.99	0.93	0.86	0.79	0.70
0.3	1.84	1.62	1.47	1.35	1.25	1.16	1.07	0.97	0.85	0.76	0.63
0.4	2.49	2.10	1.84	1.65	1.49	1.35	1.21	1.06	0.89	0.77	0.59
0.5	3.54	2.86	2.44	2.12	1.87	1.65	1.43	1.22	0.99	0.82	0.59
0.6	5.30	4.10	3.38	2.86	2.46	2.12	1.79	1.48	1.14	0.91	0.62
Confidence Level	10	20	30	40	50	60	70	80	90	95	99

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 on the capacity of the elements to

resist these demands. If the actual values of all 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 to which 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, $\mathbf{b}_U$. The coefficient $\mathbf{b}_{UT}$ is the total coefficient of variation, considering all sources of uncertainty. It is used, together with other factors to calculate the demand and resistance factors. It is assumed 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 the factored-demand-to-capacity ratio, $\mathbf{l}$, lies above or below a mean value. Table 5-7 provides a solution for this calculation, using a value of 5.0 for the hazard parameter, k, that is representative of the assumed regional seismicity during the year following a major earthquake. Further information on this method may be found in Appendix A.

5.10.2 Performance Limited by Interstory Drift Angle

5.10.2.1   Factored Interstory Drift Angle Demand

Factored interstory drift demand should be computed as the quantity, $\mathbf{g}\mathbf{g}_a D$, where the demand D is the largest interstory drift in any story, computed from structural analysis, $\mathbf{g}_a$ is the coefficient obtained from Table 5-8, and γ is the coefficient obtained from Table 5-9.

Commentary: 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-$\mathbf{D}$ instability and collapse. It is also closely related to plastic rotation, or drift angle, demand on individual beam-column connection assemblies, and therefore a good predictor of the performance of beams, columns and connections. For tall slender structures, a significant portion of interstory drift is a result of axial elongation (and shortening) of the columns. Although modeling of the structure should account for this frame flexibility, that portion of interstory drift resulting from axial column deformation in stories below the story under consideration should be neglected in determining local connection performance. Unfortunately, this portion of the interstory drift must be

*determined manually as most computer programs do not separately calculate this
quantity.*

Table 5-8   Interstory Drift Angle Analysis Demand Uncertainty Factors, γ

Analysis Procedure System Characteristic	LSP	LDP	NSP	NDP
Type 1 Connections				
Low Rise (<4 stories)	0.73	0.86	0.91	1.06
Mid Rise (4-12 stories)	1.05	1.32	1.02	1.19
High Rise (> 12 stories)	1.37	1.24	1.02	1.17
Type 2 Connections				
Low Rise (<4 stories)	1.03	1.40	1.35	1.06
Mid Rise (4-12 stories)	1.25	1.70	1.46	1.11
High Rise (> 12 stories)	0.96	1.51	1.71	1.17

**Table 5-9   Interstory Drift Angle Demand Variability Factors, γ
Type 1 and Type 2 Connections**

Building Height	γ
Type 1 Connections[1]	
Low Rise (< 4 stories)	1.6
Mid Rise (4 stories – 12 stories)	1.4
High Rise (>12 stories)	2.0
Type 2 Connections[2]	
Low Rise (< 4 stories)	1.7
Mid Rise (4 stories – 12 stories)	2.0
High Rise (>12 stories)	2.6

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.

5.10.2.2 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 5.10.2.2.1 provides values for global interstory drift capacity for regular, well-configured structures. Global interstory drift capacities for irregular structures must be determined using the detailed procedures of Appendix A. Section 5.10.2.2.2 provides procedures for evaluating local interstory drift angle capacity, as limited by connection behavior.

5.10.2.2.1 Global Interstory Drift Angle

Factored interstory drift capacity, fC, as limited by global response of the building, shall be based on the product of the resistance factor f and capacity C, which are obtained from Table 5-10, for connections with 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 5-11 provides values of the uncertainty coefficient b_{UT} to be used with global interstory drift evaluation.

Table 5-10   Global Interstory Drift Angle Capacity and Resistance Factors

Structure Type	Interstory Drift Capacity	Resistance factor f
Type 1 Connections		
Low Rise (< 4 stories)	0.10	0.85
Mid Rise (4 stories – 12 stories)	0.10	0.75
High Rise (>12 stories)	0.085	0.60
Type 2 Connections		
Low Rise (< 4 stories)	0.10	0.75
Mid Rise (4 stories – 12 stories)	0.079	0.60
High Rise (>12 stories)	0.057	0.60

Table 5-11   Uncertainty Coefficient b_{UT} for Global Interstory Drift Evaluation

Building Height	Connection Type	
	Type 1	**Type 2**
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.40	0.45
High Rise (> 12 stories)	0.50	0.55

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

5.10.2.2.2 Local Interstory Drift Angle

Factored interstory drift angle capacity, fC, limited by local connection response, shall be based on the capacity of the connection, C, and resistance factor, f, obtained from Table 5-12, for the connection types present in the building. Table 5-13 provides values of the uncertainty coefficient b_{UT} to be used with local interstory drift evaluation

Table 5-12   Local Interstory Drift Angle Capacity and Resistance Factors

Connection Type	Interstory Drift Capacity	Resistance factor f
Pre-Northridge connection with low notch toughness weld metal	$0.053-0.0006d_b$	0.7
Pre-Northridge connection with notch tough weld metal (Note 1)	$0.060-0.0006d_b$	0.85
Shear tab connections	$0.16-0.0036d_b$	0.7
Post-Northridge connection intended for steel moment-frame Service (Note 2)	0.04	0.85

Notes:
1. Weld metal with a notch toughness 40 ft –lbs at anticipated service temperature
2. Many types of connections approved for steel moment-frame service in the post-Northridge period are capable of better performance than this. Refer to *FEMA-350*, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* for more detailed data.

Table 5-13 Uncertainty Coefficient b_{UT} for Local Interstory Drift Evaluation

Building Height	Connection Type	
	Type 1	Type 2
Low Rise (3 stories or less)	0.30	0.35
Mid Rise (4 – 12 stories)	0.35	0.40
High Rise (> 12 stories)	0.40	0.40

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

5.10.3 Performance Limited by Column Compressive Capacity

5.10.3.1   Column Compressive Demand

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

D = the compressive axial load on the column determined as the sum of Dead Load, 25% of unreduced Live Load, and Seismic Demand. Seismic Demand shall be determined by either 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 5.8.2 or 5.8.3 of these *Recommended Criteria*.

Plastic: The axial demands may be taken based on plastic analysis, as indicated by Equation 5-4 of Section 5.8.2.3.5 of these *Recommended Criteria*.

Nonlinear Static: The axial demands may be taken based on the computed forces from a nonlinear static analysis, at the target displacement, in accordance with Section 5.8.4 of these *Recommended Criteria*.

Nonlinear Dynamic: The axial demands may be taken based on the computed design forces from a nonlinear dynamic analysis, in accordance with Section 5.8.5 of these *Recommended Criteria*.

g_a = Analytical demand uncertainty factor, taken from Table 5-14.

g = Demand variability factor, taken as having a value of 1.1.

The uncertainty coefficient b_{UT} shall be taken as indicated in Table 5-14 based on the procedure used to calculate column compressive demand D.

Table 5-14   Analysis Uncertainty Factor g and Total Uncertainty Coefficient b_{UT} for Evaluation of Column Compressive Demands

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

Note: β may be taken as the coefficient of variation of the axial load values determined from the suite of nonlinear analyses.

Commentary: The value of g has been computed assuming a coefficient of variation for axial load values resulting from material strength variation and uncertainty in dead and live loads of 15%. The values of g_a have been calculated assuming coefficients of variation of 30%, 0% and 15% related to uncertainty in the analysis procedures for linear, plastic and nonlinear static analyses, respectively. In reality, for structures that are stressed into the inelastic range, elastic analysis will typically overestimate axial column demands, in which case, a value of 1.0 could be used. 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 a hazard parameter k, having a value of 5.0, representative of the assumed seismicity for the immediate postearthquake period.

5.10.3.2   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, f, and the nominal axial strength of the column, C, 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 f shall be assigned a value of 0.90.

5.10.4 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 has a minimum notch toughness of 40 ft-lbs at the lowest anticipated service temperature, need not be evaluated.

5.10.4.1 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 5.10.3.1. 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 5-14. The total uncertainty coefficient β_{UT} shall also be taken as indicated in Table 5-14.

5.10.4.2 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 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 0.85.

5.11 Evaluation Report

Regardless of the level of evaluation performed, the responsible structural engineer should prepare a written evaluation report and submit it to the owner upon completion of the evaluation. When the building official has required evaluation of a steel moment-frame building, this report should also be submitted to the building official. This report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following material:

- Building address

- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, and the type and location of lateral-force-resisting elements. Include a description of the grade of steel specified for beams and columns and, if known, the type of welding (e.g., Shielded Metal Arc Welding, or Flux-Cored Arc Welding) present. Indicate if moment connections are provided with continuity plates. The narrative description should be supplemented with sketches (plans and evaluations) as necessary to provide a clear understanding of pertinent details of the building's construction. The description should include an indication of any structural irregularities as defined in the Building Code.

- A description of nonstructural damage observed in the building.

- An estimate of the ground shaking intensity experienced by the building, determined in accordance with Section 5.7.

- A description of the inspection and evaluation procedures used, including the signed inspection forms for each individual inspected connection.

- A description, including engineering sketches, of the observed damage to the structure as a whole (e.g., permanent drift) as well as at each connection, keyed to the damage types in Chapter 5; photographs should be included for all connections with significant visible damage.

- Calculations demonstrating the determination of a confidence level with regard to the building's ability to resist collapse in the immediate postearthquake period.

- A summary of the recommended actions (repair and modification measures and occupancy restrictions).

The report should include identification of any potentially hazardous conditions that were observed, including corrosion, deterioration, earthquake damage, pre-existing rejectable conditions, and evidence of poor workmanship or deviations from the approved drawings. In addition, the report should include an assessment of the potential impacts of observed conditions on future structural performance. The report should include the Field Inspection Reports of damaged connections (visual inspection and nondestructive testing records, data sheets, and reports), as an attachment, and should bear the seal of the structural engineer in charge of the evaluation.

> *The nature and scope of the evaluations performed should be clearly stated in the structural engineer's written evaluation report. If the scope of evaluation does not permit an informed judgment to be made as to the extent with which the building complies with the applicable building codes, or as to a statistical level of confidence that the damage has not exceeded an acceptable damage threshold, this should be stated .*

6. POSTEARTHQUAKE REPAIR

6.1   Scope

This section provides criteria for structural repair of earthquake damage. *Repair* constitutes any measures taken to restore earthquake damaged elements of the building, including individual members or their connections, or the building as a whole, to their original configuration, strength, stiffness and deformation capacity. It does not include routine correction of non-conforming conditions resulting from the original construction or upgrades intended to result in improvement in future seismic performance of the building. Repair must typically be performed under a building permit, requiring submittal, to the building department of construction documents, inspection and testing reports.

Sections 6.2 through 6.3 provide recommended methods of repair for various types of damage. These recommendations are not intended to be used for the routine repair of construction non-conformance commonly encountered in fabrication and erection work. Industry standard practices are acceptable for such repairs. Recommendations for assessment of the seismic performance capability of existing buildings and upgrade of buildings to improve performance capability may be found in a companion publication, *FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*.

Commentary: Based on the observed behavior of actual buildings in the Northridge earthquake, as well as recent test data, welded steel moment frame buildings constructed with the typical detailing and construction practice prevalent prior to 1994 do not have the deformation capacity they were presumed to possess at the time of their design and therefore present significantly higher risks than was originally thought. When these buildings are damaged or have latent construction defects, this risk is higher still.

Based on limited testing, it is believed that the repair recommendations contained in these Recommended Criteria can be effective in restoring a building's pre-earthquake condition, and to the extent that the detailing, workmanship and materials of repair work are superior to the original construction, provide some marginal improvement in seismic performance capability. This does not imply, however, that the repaired building will be an acceptable seismic risk. As a minimum, it should be assumed that buildings that are repaired, but not upgraded, can sustain similar and possibly more severe damage in future earthquakes than they did in the present event. If this is unacceptable, either to the owner or the building official, then the building should be upgraded to provide improved future performance. Seismic upgrade can consist of local reinforcement of individual moment connections, column splices and other critical connections, as well as alteration of the basic lateral-force-resisting characteristics of the structure through addition, for example, of braced frames, shear walls, base isolation, and energy dissipation devices. Performance

evaluation and structural upgrade are beyond the scope of these Recommended Criteria. Criteria for performance evaluation and structural upgrade may be found in a companion document, FEMA-351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings.

6.2 Shoring and Temporary Bracing

6.2.1 Investigation

Prior to engaging in repair activity, the structural engineer should investigate the entire building and perform an evaluation to determine if any imminent collapse or life-safety hazard conditions exist and to determine if the structure as a whole provides adequate stability to safeguard life during the repair process. The Level 2 evaluation process of Chapter 5 is one method of confirming both the building's global structural stability, and the ability of individual connections to withstand ground shaking. Where hazardous conditions or lack of stability are detected, shoring and or temporary bracing should be provided prior to commencement of any repairs.

> *Commentary: In projects relating to construction of new buildings, it is common practice to delegate all responsibility for temporary shoring and bracing of the structure to the contractor. Such practice may not be appropriate for severely damaged buildings. The structural engineer should work closely with the contractor to define shoring and bracing requirements. Some structural engineers may wish to perform the design of temporary bracing systems. If the contractor performs such design, the structural engineer should review the designs for adequacy and potential effects on the structure prior to implementation.*

6.2.2 Special Requirements

Conditions that may become collapse or life-safety hazards during the repair operations should be considered in the development of repair details and specifications, whether they involve the damage area directly or indirectly. These conditions should be brought to the attention of the contractor by the structural engineer, and adequate means of shoring these conditions should be developed. Consideration should be given to sequencing of repair procedures for proper design of any required shoring. For column repair details that require removal of 20% or more of the damaged cross section, consideration should be given to the need for shoring to prevent overstress of elements due to redistribution of loads.

> *Commentary: In general, contractors will not have adequate resources to define when such shoring is necessary. Therefore, the Contract Documents should clearly indicate when and where shoring is required. Design of this shoring may be provided by the structural engineer, or the contract documents may require that the contractor submit a shoring design, prepared by another registered structural engineer, to the structural engineer for review.*

6.3   Repair Details

The scope of repair work should be shown on drawings and specifications prepared by a structural engineer. The drawings should clearly indicate the areas requiring repair, as well as all repair procedures, details, and specifications necessary to properly implement the proposed repair. Sample repair details for various types of damage are included in these recommended criteria, for reference, only.

Commentary: Examples of repair details are provided for some classes of damage, based on approaches successfully performed in the field following the 1994 Northridge earthquake. Limited testing indicates these repair methods can be effective. Details are not complete in all respects and should not be used verbatim, as construction documents. Many repairs will require the application of more than one operation, as represented by a given detail. The sample details indicated may not be directly applicable to specific repair conditions. The structural engineer is cautioned to thoroughly review the conditions at each damaged element, connection or joint, and to determine the applicability and suitability of these details based on sound structural engineering judgment, prior to employing them on projects.

In typical practice for construction of new buildings, the selection of means and methods used to construct design details are typically left to the contractor. In structural repair work, the members are typically under greater load and also restraint during the fabrication and erection process than is common in new construction. Therefore, the typical construction practices may not be appropriate and many contractors may not have the knowledge or experience to select appropriate methods for repair work. As a result, much greater specification of means and methods is recommended than is common in new construction. Although it is recommended that the engineer provide such specification as part of the construction documents, the engineer should also be open to suggestions for alternative procedures if the contractor desires to submit such procedures. If there is doubt as to the ability of alternative procedures to provide acceptable construction, a full-scale mock-up test of the proposed procedure should be considered.

6.3.1   Approach

Based on the nature and extent of damage several alternative approaches to repair should be considered. Repair approaches may include, but should not be limited to:

- replacement of damaged portions of base metal (i.e. column and beam section),
- replacement of damaged connection elements,
- replacement of connection welds, or
- repairs to portions of any of the aforementioned components.

Any or all of these techniques may be appropriate. The approaches used should consider adjacent structural components that may be affected by the repair or the effects of the repair.

Where base material is to be removed and replaced with plates or shapes, clear direction should be given to orient the new material with the direction of rolling parallel to the direction of application of major axial loads to be resisted by the section.

6.3.2 Weld Fractures - Type W Damage

Prior to repair of fractures or rejectable defects in welds, sufficient material should be removed to completely eliminate any existing discontinuity or defect in the weld metal and if applicable, adjacent base material. Nondestructive Testing should be used to determine the extent of fracture or defect present and sufficient material should be removed to encompass the damaged area. It is suggested that material removal extend 2 inches beyond the apparent end of the fracture or defect. Simple fillet welds may be repaired by back gouging to eliminate unsound weld material and replacement of the damaged weld with sound material. Complete joint penetration (CJP) welds fractured through the full thickness should be replaced with sound material deposited in strict accordance with an appropriate Welding Procedure Specification (WPS) and the project specifications. Weld backing, existing end dams, and weld tabs should be removed from all welds that are being repaired. End dams should not be permitted in new work. After backing and tab elements are removed, the weld root should be back gouged to sound material, re-welded and a reinforcing fillet added.

The structural engineer is cautioned to observe the provisions of AISC regarding intermixing of weld metals deposited by different weld processes (see *AISC LRFD Manual of Steel Construction*, second edition, page 6-77, and the 1989 *AISC ASD Steel Construction Manual*, ninth edition, page 5-69). As an example, E7018 shielded metal arc welding (SMAW) electrodes should not be used to weld over self-shielded flux cored arc welding (FCAW-S) deposits, unless appropriate precautions are taken (*FEMA 355b*). Typically, three to four passes of E7018 or similar notch tough filler metal should be deposited to ensure that the underlying FCAW-S filler metal has not degraded the overlying notch tough filler metal. Removed weld material from fractures not penetrating the full weld thickness should be replaced in the same manner as full thickness fractures. For other types of W damage, existing backing, end dams, and weld tabs should also be removed in a like manner to CJP weld replacement. Table 6-1 provides an index to suggested repair details for type W damage.

Table 6-1   Reference Details for Type W Damage

Damage or Defect Class	Figure
Rejectable defects at weld root	Figure 6-1, Figure 6-2
W2	Figure 6-3
W3	Figure 6-3
W4	Figure 6-3
W5	Figure 6-3

Notes:

1. Remove existing backing.
2. Taper the depth of grinding or air arc gouging at each end to the face of flange with a minimum 2:1 (horizontal/vertical) taper. Provide a minimum root radius of ¼".
3. Grind all surfaces on which weld metal will be deposited. Surfaces should be smooth, uniform and free from fins, tears, fractures and other discontinuities that would adversely affect weld strength.
4. A fillet weld should be applied to reinforce the joint. The size of the reinforcing fillet should be equal to 1/4 of the beam flange thickness, but not less than 1/4". It need nor be more than 3/8".
5. On joints to be repaired, remove all remaining weld tabs and excess weld metal beyond the length of the joint and grind smooth. Imperfection less than 1/16" should be removed by grinding. Repair as necessary.

Figure 6-1   Gouge and Re-weld of Root Defect or Damage

Notes:

1. Remove the entire fracture plus 1/8" of sound metal beyond each end.
2. For additional notes, refer to Figure 6-1
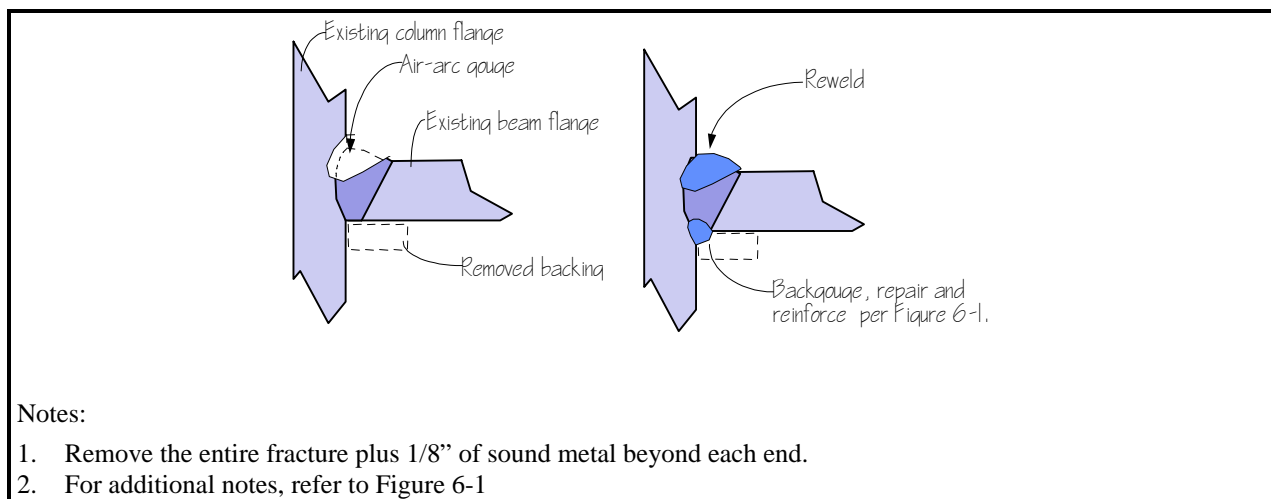
Figure 6-2   Gouge and Re-weld of Fractured Weld
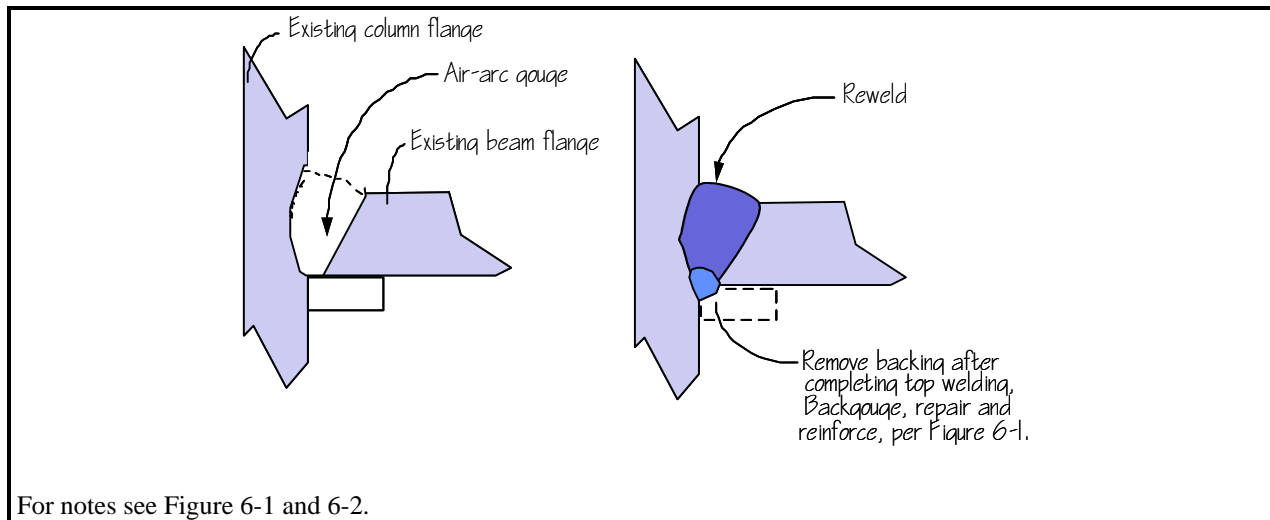
For notes see Figure 6-1 and 6-2.

Figure 6-3   Backgouge and Reweld Repair

*Commentary: Flux-cored arc-welding (FCAW-S) utilizes approximately 1-2%
aluminum in the electrode to protect the weld from mixing with atmospheric
nitrogen and oxygen. By itself, aluminum can reduce the toughness and ductility
of weld metal. The design of FCAW-S electrodes requires the balance of other
alloys in the deposit to compensate for the effects of aluminum. Other welding
processes rely on fluxes and/or gasses to protect the weld metal from the
atmosphere, relieving them of any requirement to contain aluminum or other
elements that offset the effects of aluminum. If the original weld that is being
repaired consists of FCAW-S and subsequent repair welds are made with
shielded-metal arc-welding, SMAW (stick), using E7018, for example, the SMAW
arc will penetrate into the FCAW-S deposit, resulting in the addition of some
aluminum into the SMAW deposit. The notch toughness and/or ductility of the
resultant weld metal may be substantially reduced as compared to pure E7018
weld metal, based on the depth of penetration into the FCAW-S material.*

*Various types of FCAW-S electrodes may be mixed one with the other without
potentially harmful effect. Further, notch tough FCAW-S may be used to weld
over other types of weld deposits without potentially harmful interaction. The
structural engineer could specify all repairs on FCAW-S deposits be made with
FCAW-S. Alternatively, intermixing of FCAW-S and other processes could be
permitted provided the subsequent composition is demonstrated to meet material
specification requirement, or adequate layers of reinforcing notch-tough filler
metal are installed to avoid segregation (FEMA 355B).*

*The recommendations contained in Chapters 3, 4, and 5 for inspection and
evaluation of damaged buildings do not require extensive nondestructive
examination of welds to detect defects or fractures that are not detectable by
visual inspection but are rejectable under the AWS D1.1 provisions.*

Nevertheless, it is likely that in the course of performing inspection and repair work, some such rejectable conditions will be found. It is recommended that any such detected conditions be repaired as part of the overall building repair program, as their presence in welds make the welds significantly more vulnerable to future fracturing under loading, particularly if the welds are composed of material with limited notch toughness.

In the past, there has been considerable disagreement as to whether or not small cracks and defects at the root of a weld are earthquake damage or not. Proper observation by knowledgeable persons can reveal whether a root defect is a slag inclusion or lack of fusion, both conditions relating to the original construction, or an actual crack. It should be noted that cracks may not necessarily be caused by the building's earthquake response. Some cracking invariably occurs in structures during the erection process as a result of residual stress conditions and thermal stresses. It is almost impossible to distinguish such cracks from those caused by an earthquake. Through detailed examination of the fracture surface for evidence of oxidation or other signs of age it may be possible to obtain clues as to when a crack initiated. Many researchers believe that the low toughness weld metal commonly used in construction prior to 1994 was incapable of arresting an earthquake induced fracture, once it initiated in a joint and that small cracks that do not penetrate through the metal are unlikely to be earthquake related. However, there have been reports from laboratory testing that indicate that small cracks do form in the weld metal and arrest prior to development of unstable fracture conditions, even in low-toughness weld metals. Therefore, without detailed examination of an individual fracture by knowledgeable individuals, no conclusive statement can be made as to whether weld cracking is earthquake induced.

6.3.3 Column Fractures - Types C1 to C5 and P1 to P6

Any column fracture observable with the naked eye or found by NDT and classified as rejectable in accordance with the AWS D1.1 criteria for Static Structures should be repaired. Repairs should include removing the fracture such that no sign of rejectable discontinuity or defect within a six (6) inch radius around the fracture remains. Removal should include eliminating any zones of fracture propagation, with a minimum of heat used in the removal process. Following removal of material, magnetic particle testing (MT) and/or Liquid Dye Penetrant testing (PT) should be used to confirm that all fractured material has been removed. Repairs of removed material may consist of replacement of portions of column section, build-up with weld material where small portions of column were removed, or local replacement of removed base metal with weld material. Procedures of weld fracture repair should be applied to limit the heat-affected area and to provide adequate ductility to the repaired joint. Table 6-2 indicates representative details for these repairs. In many cases, it may be necessary to remove a portion of the girder framing to a column, in order to attain necessary access to perform repair work, per Figure 6-4. Refer to Section 6.3.5 and Figures 6-9 and 6-10 for repair of girders, or if access is restricted, as an alternative beam repair method.
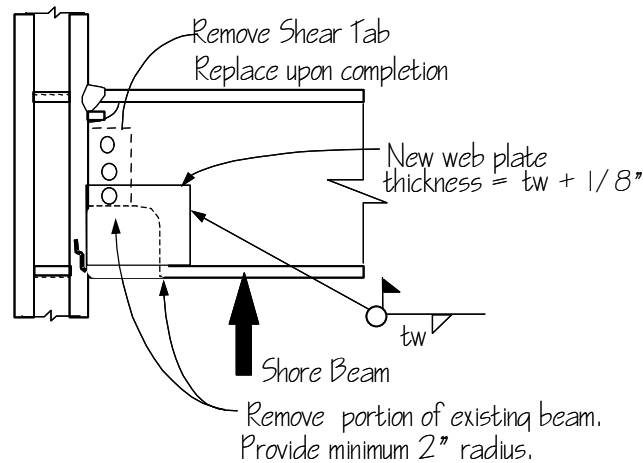
Figure 6-4   Temporary Removal of Beam Section for Access

When the size of divot (type C2) or transverse column fractures (types C1, C3, C4) dictate a total cut-out of a portion of a column flange or web (types P6, P7), the replacement material should be ultrasonically tested in accordance with ASTM A578-92, Straight-Beam Ultrasonic Examination of Plane and Clad Steel Plates for Special Applications, in conjunction with AWS K6.3 Shearwave Calibration. Acceptance criteria should be that of Level III. The replacement material should be aligned with the rolling direction matching that of the column.

Table 6-2   Reference Details for Type C and P Damage

Damage Class	Figure
Beam Access	Figure 6-4
C1	Figures 2-3, 6-4, 6-5
C2	Figures 2-3, 6-4, 6-6
C3	Figures 2-3, 6-4, 6-5
C4	Figures 2-3, 6-4, 6-5
C5	Figures 2-3, 6-4, 6-6
P1	Figure 2-6; remove, prepare, replace
P2	Figure 2-6; arc-gouge and reweld
P4	Figure 2-6; arc-gouge and reweld
P5	Figures 2-6, 6-7
P6	Figures 2-6, 6-7
P7	Figures 2-6, 6-7
P8	Figures 2-6, 6-8

Commentary: Special attention should be given to conditions where more than 20% of the column cross section will be removed at one time, as special temporary shoring may be warranted. In addition, care should be taken when applying heat to a flange or web containing a fracture, as fractures have been observed to propagate with the application of heat. This can be prevented by drilling a small diameter hole at the end of the fracture, to prevent it from running.

Notes:
1. Investigate extent of fracture by UT to confirm that fractures are contained with the 45 degree angle zone of a standard pre-qualified CJP groove weld as defined in Figure 2.4, AWS D1.1, Joint Designation B-U4a-G
2. Provide 10° bevel on lower flange plate, to channel slag out of joint.
3. Grind all surfaces upon which weld metal will be deposited to smooth, uniform surface.

Figure 6-5   Backgouge and Reweld of Column Flange

Note: Provide new flange plate material of the same strength, and width as the existing column flange. Align rolling direction of plate with that of column flange. New plate should be of the same thickness as the existing flange with a tolerance of $-0"/+\frac{1}{4}"$. The welding should be sequenced to connect the column flange to new flange plate welds prior to welding the column web to new flange plate. Bevel the lower edge of the column flange, and upper edge of the splice plate down 10°, to channel slag out of joint.

Figure 6-6   Replacement of Column Flange Repair

Web with Doubler Plate   Web without Doubler Plate

Notes:
1. Prepare fractured section of doubler by air-arc gouging, grinding and rewelding, using web as backing.
2. Prepare fractured section of web by air-arc gouging, grinding and rewelding, using doubler as backing or backgouge and reweld from reverse side, if no doubler present.
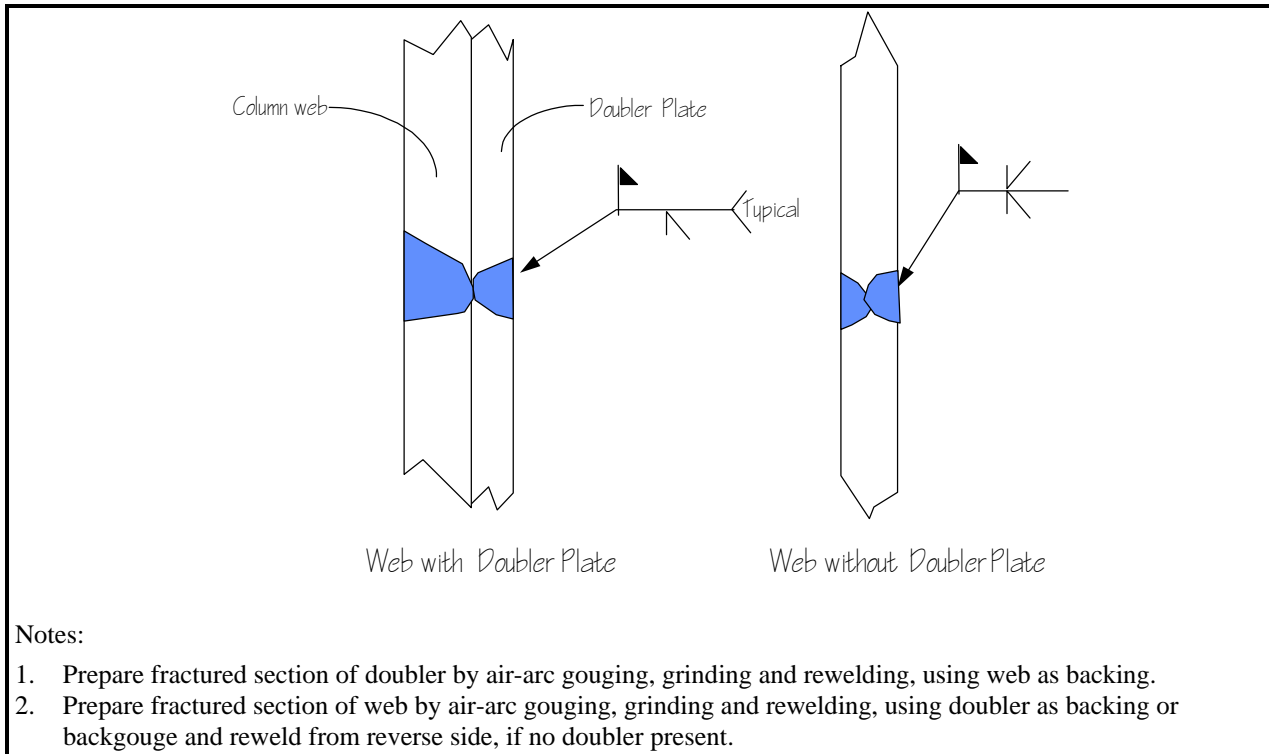
Figure 6-7   Reweld Repair of Web plate and Doubler plate

Flange removal and replacement
per Figure 6-6, if required

Weld access holes as required
for weld terminations

Notes:

1. Sequence removal of portions of column and provide shoring as required to safely support existing column loads.
2. Thickness of new web plate to match existing column web (tolerance –0"/+ ").
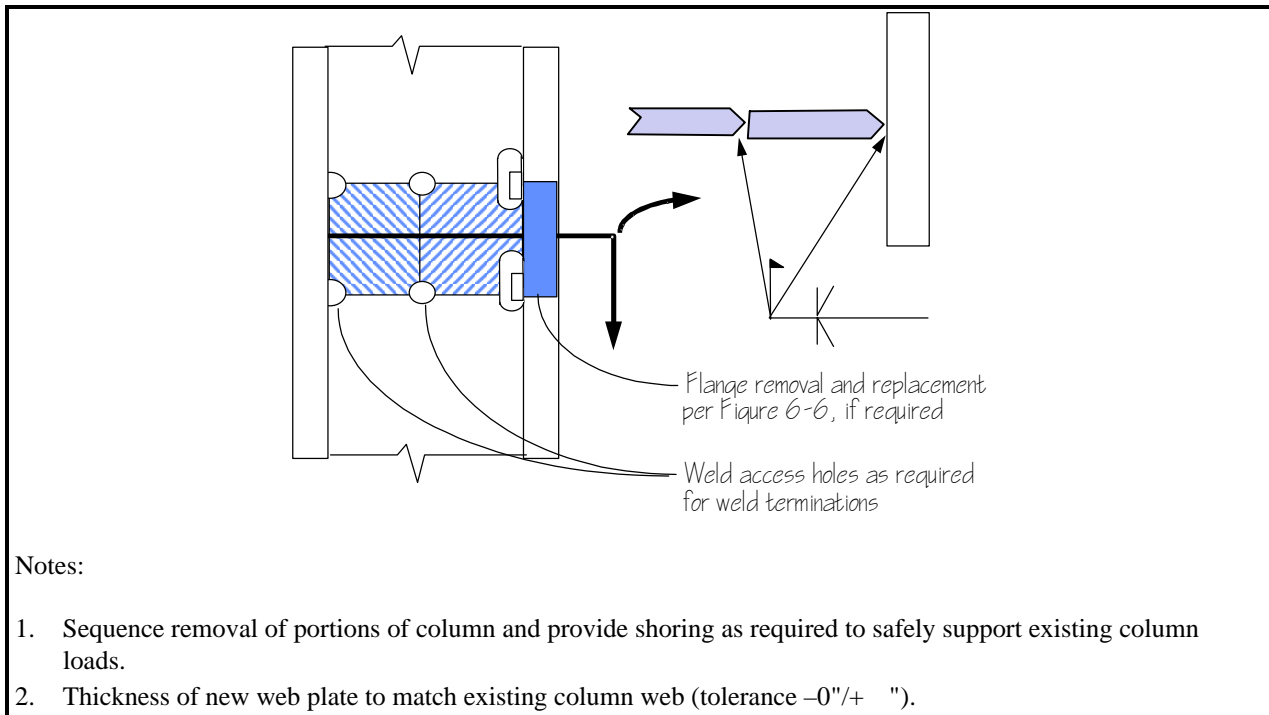
Figure 6-8   Alternative Column Web Repair - Columns without Doubler Plates

6.3.4 Column Splice Fractures - Type C7

Any fractures detected in column splices should be repaired by removing the fractured material and replacing it with sound weld material. For partial joint penetration groove welds, remove up to one half of the material thickness from one side and replace with sound material. Where complete joint penetration groove welds are required, it may be preferable to provide a double bevel weld, repairing one half of the material thickness completely prior to preparing and repairing the other half. Alternatively, if calculations indicate that column loads may safely be resisted with the entire section of column flange removed, or if suitable shoring is provided, it may be preferable to use a single bevel weld.

Commentary: Special attention should be given to these conditions, as the removal of material may require special temporary shoring. Also, since partial penetration groove welds can serve as fracture initiators in tension applications, consideration should be given to replacing such damaged splice areas with complete joint penetration welds (see Figures 6-5 and 6-6). Also, the addition of flange plates to the outside face of each flange may be considered.

6.3.5 Girder Flange Fractures - Type G3 to G5

Repair of fractures in girder flanges may be performed by several methods. One method is to remove the fracture by air arc gouging such that no sign of discontinuity or defect within a six (6) inch radius around the fracture remains, preparing the surface by grinding and welding new material back. Alternatively, damaged portions of the girder flange may be removed and replaced with new plate as shown in Figure 6-9 or Figure 6-10.

Commentary: Due to accessibility difficulties or excessive weld build-up requirements, it may become necessary to remove a portion of the girder flange to properly complete the joint repair. A minimum of six inches of girder flange may be removed to facilitate the joint repair, with the optimum length being equal to the flange width. After removal of the portion of flange, the face of column and cut edge of girder flange may then be prepared to receive a splice plate matching the flange in grade and width. Thickness should be adjusted as required to make-up the depth of the girder web and fillet removed as part of the preparation process.

In the case of restricted access on one side of the beam (facade interference) it may be advantageous to make the plate narrower than the beam flange and perform all welding overhead. A CJP weld and fillet weld should be used to connect the plate to the column flange and beam flange, respectively.

It is recommended that a double bevel joint be utilized in replacing the removed plate to eliminate the need for backing, consequently also eliminating the need for removal of the backing upon joint completion. A suggested joint detail is a B-U3/TC-U5, per AWS D1.1, with 1/3 t_{flange} to 2/3 t_{flange} bevels on the plate. The web of the girder should be prepared at the column and butt weld

areas to allow welding access. Weld tabs may be used at the column and beam flange butt welds, but should be removed following joint completion. The weld between the splice plate and the column flange should be completed first. If a double bevel weld is selected, the welder may choose to weld the first few passes from one face, then backgouge and weld from the second side. This may help to keep the interpass temperature below the maximum without down time often encountered in waiting for the weld to cool.
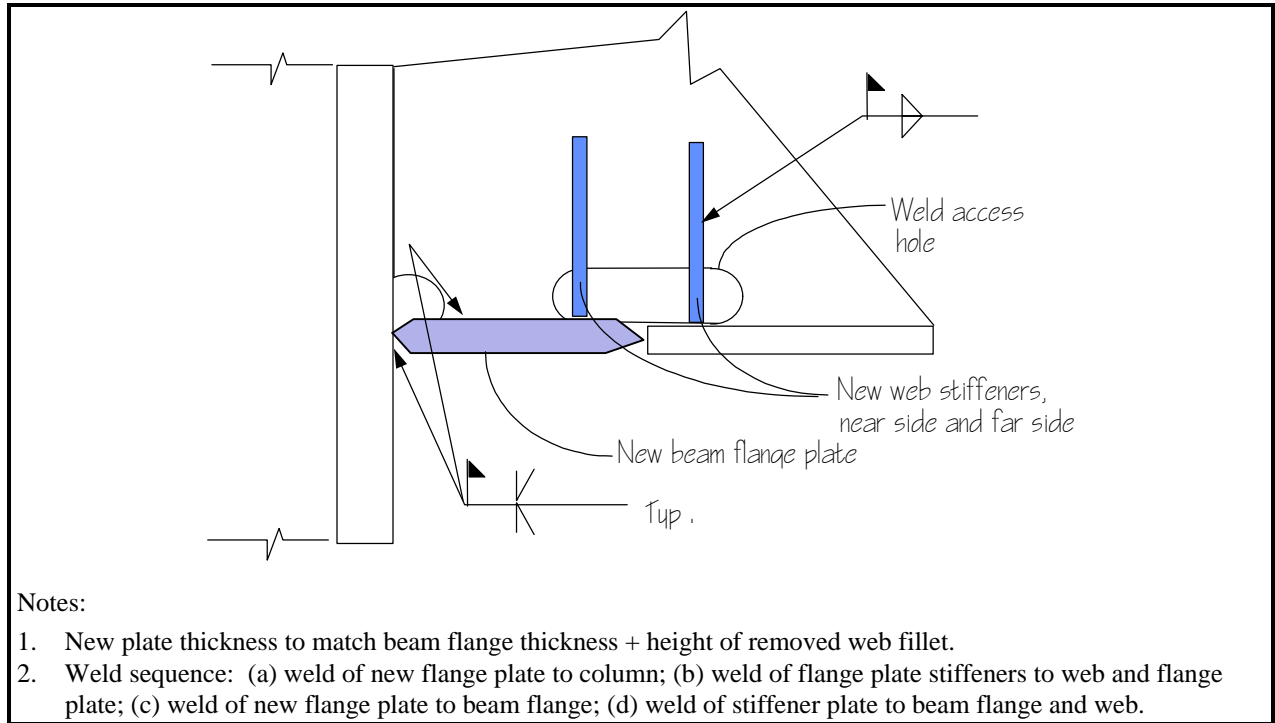
Notes:
1. New plate thickness to match beam flange thickness + height of removed web fillet.
2. Weld sequence: (a) weld of new flange plate to column; (b) weld of flange plate stiffeners to web and flange plate; (c) weld of new flange plate to beam flange; (d) weld of stiffener plate to beam flange and web.

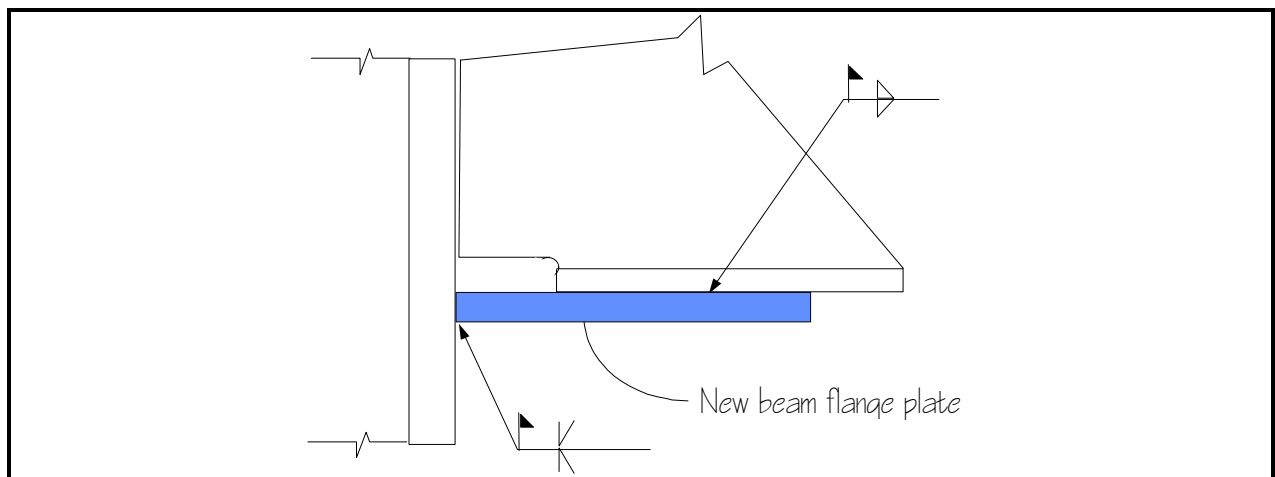**Figure 6-9   Beam Flange Plate Replacement**

Figure 6-10   Alternative Beam Flange Plate Replacement

6.3.6 Buckled Girder Flanges - Type G1

Where the top or bottom flange of a girder has buckled, and the rotation between the flange and web is less than or equal to the mill rolling tolerance given in the 1989 or 1994 *AISC Manual of Steel Construction* the flange need not be repaired. Where the angle is greater than mill rolling tolerance, repair should be performed and may consist of adding full height stiffener plates on the web over each portion of buckled flange, contacting the flange at the center of the buckle, (Figure 6-11) or using heat straightening procedures. Another available approach is to remove the buckled portion of flange and replace it with plate, similar to Figures 6-9 and 6-10.
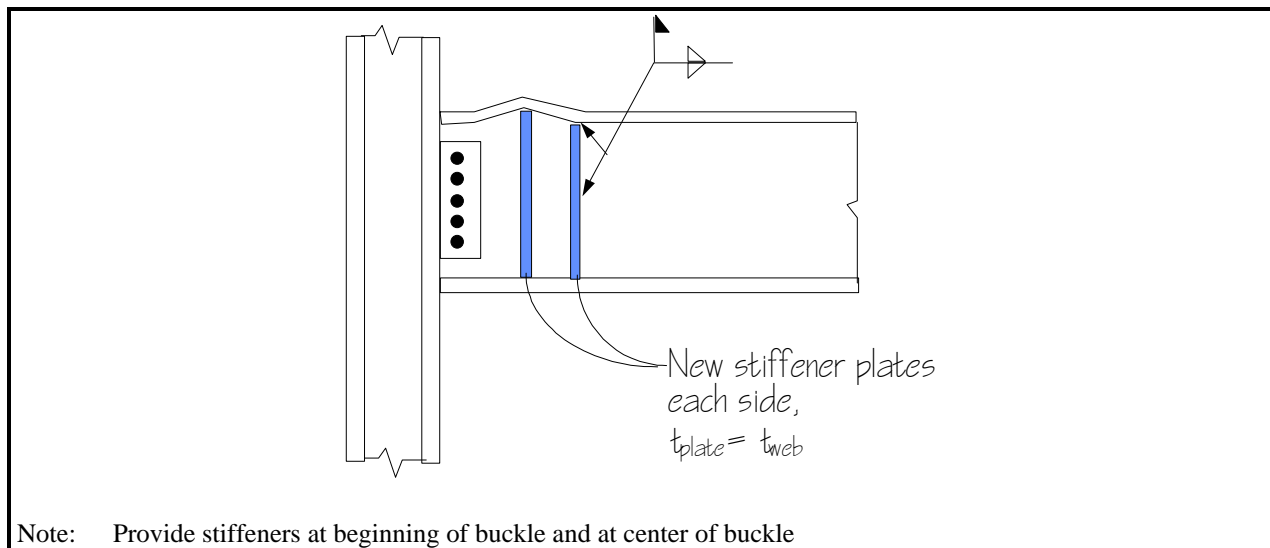

Note: Provide stiffeners at beginning of buckle and at center of buckle

Figure 6-11   Addition of Stiffeners at Buckled Girder Flange

Commentary: Should flange buckling occur on only one side of the web, and the buckle repair consists of adding stiffener plates, only the side that has buckled need be stiffened. In case of partial flange replacement, special shoring requirements should be considered by the design engineer.

6.3.7 Buckled Column Flanges - Type C6

Any column flange or portion of a flange that has buckled to the point where it exceeds the rolling tolerances given in the *AISC Manual of Steel Construction* should be repaired. Flange repair may consist either of flame straightening or of removing the entire buckled portion of the flange and replacing it with material with yield properties similar to the actual yield properties of the damaged material similar to Figure 6-6. If workers with the appropriate skill to perform flame straightening are available, this is the preferred method.

Commentary: For flange replacement, shoring is normally required. This shoring should be designed by the structural engineer, or may be designed by the contractor provided the design is reviewed by the structural engineer.

Flame straightening can be an extremely effective method of repairing buckled members. It is performed by applying heat to the member in a triangular pattern, in order to induce thermal strains that straighten the member out. Very large bends can be straightened by this technique. However, the practice of this technique is not routine and there are no standard specifications available for controlling the work. Consequently, the success of the technique is dependent on the availability of workers who have the appropriate training and experience to perform the work. During the heat application process, the damaged member is locally heated to very high temperatures. Consequently, shoring may be required for members being straightened in this manner.

A number of references are available that provide more information on this process and its applications, published by AISC and others (Avent, 1992; Shonafelt and Horn, 1984)

6.3.8   Gravity Connections

Connections not part of the lateral force-resisting system may also be found to require repair due to excessive rotation or demand caused by distress of the lateral-force-resisting system. These connections should be repaired to a capacity at least equivalent to the pre-damaged connection capacity. Shear connections that are part of the lateral-force-resisting system should be repaired in a similar manner, with special consideration given to the nature and significance of the overall structural damage.

Commentary: Testing of typical gravity shear connections conducted as part of the research performed in support of the development of these Recommended Criteria indicates that shear tab connections are capable of sustaining very large rotation demands without compromise of their gravity load carrying capacity (FEMA-355D). These connections tend to degrade in strength only when the imposed rotation becomes large enough to induce contact between the beam flange and the adjacent member. Once this contact occurs, rotational resistance of the connection stiffens substantially with large forces generated both through bearing of the beam flange against the adjacent member and as an axial force transmitted through the shear tab. The resulting forces can compromise, the bolts, the weld of the shear tab to the supporting member or the shear tab itself. Such behavior has not occurred in laboratory testing until connection rotations in excess of 0.1 radians were achieved.

6.3.9   Reuse of Bolts

Bolts in a connection displaying bolt damage or plate slippage should not be re-used, except as indicated herein. As indicated in the *AISC Specification for Structural Joints using ASTM A325 or A490 Bolts* (AISC, 1985), A490 bolts and galvanized A325 bolts should not be re-tightened and re-used under any circumstances. Other A325 bolts may be reused if determined to be in good condition. Touching up or re-tightening previously tightened bolts which may have

been loosened by the tightening of adjacent bolts need not be considered as reuse provided the snugging up continues from the initial position and does not require greater rotation, including the tolerance, than that required by Table 5 of the *AISC Specification*.

> *Commentary: Proper performance of high strength bolts used in slip critical applications requires proper tensioning of the bolt. Although a number of methods are available to ensure that bolts are correctly tensioned, the most common methods relate to torquing of the nut on the bolt. When a bolt has been damaged, the torquing characteristics will be altered. As a result, damaged bolts may either be over-tightened or under-tightened, if reinstalled. The threads of ASTM A490 bolts and galvanized ASTM A325 bolts become slightly damaged when tightened, and consequently, should not be reused. To determine if an ungalvanized ASTM A325 bolt is suitable for re-use, a nut should be run up the threads of the bolt. If this can be done smoothly, without binding, then the bolt may be re-used.*

6.3.10 Welding Specifications

Welded repairs involving thick plates and conditions of high restraint should be specified with caution. These conditions can lead to large residual stresses and in some cases, initiation of cracking before the structure is loaded. The potential for problems can be reduced by specifying appropriate joint configurations, welding processes, control of preheat, heat input during welding and cooldown, as well as selecting electrodes appropriate to the application. Engineers who do not have adequate knowledge to confidently specify these parameters should seek consultation from a person with the required expertise.

6.4   Preparation

6.4.1   Welding Procedure Specifications

A separate Welding Procedure Specification (WPS) should be established for every different weld configuration, welding position, and material specification. The WPS is a set of focused instructions to the welders and inspectors stating how the welding is to be accomplished. Each type of weld should have its own WPS solely for the purpose of that weld. The WPS should include instructions for joint preparation based on material property and thickness, as well as welding parameters. Weld process, electrode type, diameter, stick-out, voltage, current, and interpass temperature should be clearly defined. In addition, joint preheat and postheat requirements should be specified as appropriate, including insulation guidelines if applicable. The WPS should also list any requirements that are mandated by the project specification. Two categories of qualified welding procedures are given in *AWS D1.1*. These are pre-qualified welding procedures and qualified-by-test welding procedures. Regardless of the type of qualification of a proposed welding procedure, a WPS should be prepared by the contractor and reviewed by the structural engineer responsible for the repairs.

> *Commentary: Preparation of the WPS is normally the responsibility of the fabricator/erector. Sample formats for WPS preparation and submission are*

*included in AWS D1.1. Some contractors fill out the WPS by inserting references
to the various AWS D1.1 tables rather than the actual data. This does not meet
the intent of the WPS, which is to provide specific instructions to the welder and
inspector on how the weld is to be performed. The actual values of the
parameters to be used should be included in the WPS submittal.*

6.4.2 Welder Training

Training of welders should take place at the outset of the repair operations. Welders and
inspectors should be familiar with the WPS, and should be capable of demonstrating familiarity
with each of its aspects. A copy of the WPS should be located on site, preferably at the
connection under repair, accessible to all parties involved in the repair.

6.4.3 Welder Qualifications

Welders must be qualified and capable of successfully making the repair welds required. All
welders should be qualified to the *AWS D1.1* requirements for the particular welding process and
position in which the welding is to be performed. Successful qualification to these requirements,
however, does not automatically demonstrate a welder's ability to make repair welds for all the
configurations that may be encountered. Specific additional training and/or experience may be
required for repair situations. Inexperienced welders should demonstrate their ability to make
proper repair welds. This may be done by welding on a mock-up assembly (see Section 6.4.4)
that duplicates the types of conditions that would be encountered on the actual project.
Alternatively, the welder could demonstrate proficient performance on the actual project,
providing this performance is continuously monitored during the construction of at least the first
weld repair. This observation should be made by a qualified welding inspector or engineer.

6.4.4 Joint Mock-Ups

A joint mock-up should be considered as a training and qualification tool for each type of
repair that is more challenging than work in which the welder has previously demonstrated
competence. This will allow the welder to become familiar with atypical welds, and will give the
inspector the opportunity to observe clearly the performance of each welder. An entire mock-up
is recommended for each such case, rather than only a single pass or portion of the weld. In a
complete mock-up, all welding positions and types of weld would be experienced, thus showing
the welder capable of both completing successfully the weld in all required positions, and
applying all heating requirements.

> *Commentary: The structural engineer may, at his or her discretion, require joint
> mock-ups to be performed for specific types of repair work as part of the project
> specifications. This practice is recommended where repair work must be made
> under conditions of unusual or restricted access, under conditions of high
> restraint, or for any joint that is not routinely performed in the industry.*

6.4.5 Repair Sequence

Repair sequence should be considered in the design of repairs, and any sequencing requirements should be clearly indicated on the drawings and WPS. Structural instabilities or high residual stresses could arise from improper sequencing. The order of repair of flanges, shear plates, and fractured columns should be indicated on the drawings as appropriate to guard against structural failure and to reduce possible residual stresses.

6.4.6 Concurrent Work

The maximum number of connections permitted to be repaired concurrently should be indicated on the drawings or in the project specifications.

> *Commentary: Although a connection is damaged, it may still possess significant ability to participate in the structure's lateral-force-resisting system. Consideration should be given to limiting the total number of connections being repaired at any one time, as the overall lateral-force resistance of the structure may be temporarily reduced by some repair operations. If many connections are under repair simultaneously, the overall lateral resistance of the remaining frame connections may not be adequate to protect the structure's stability. Although this appears to fall under the category of means and methods, the typical contractor would have no way of determining the maximum number of connections that can be repaired at any one time without requiring supplemental lateral bracing of the building during construction. Therefore, the structural engineer should take a proactive role in determining this.*

6.5   Execution

6.5.1 General

FEMA-353, Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications, provides recommended general requirements to be included in specifications for repair. The following are of particular importance:

- Strict enforcement of the welding requirements in *AWS D1.1* adopted and modified by the building code.

- Implementation of the special inspection requirements in the *1997 NEHRP Recommended Provisions for New Buildings* and *AWS D1.1,* as well as such other requirements enforced by the local Building Department. Visual inspection means that the inspector inspects the welding periodically for adherence to the approved Welding Procedure Specification (WPS) and *AWS D1.1,* starting with preliminary tack welding and fit-up and proceeding through the welding process. Reliance on the use of nondestructive testing (NDT) at the end of the welding process alone should be avoided. Use visual inspection in conjunction with NDT to improve the chances of achieving a sound weld.

- Require the fabricator to prepare and submit a WPS with at least the information required by *AWS D1.1,* as discussed in Section 6.5.4.

- Welding electrodes for welded joints in severe service and with significant consequences of failures should be capable of depositing weld metal with a minimum notch toughness of 40 ft-lbs at the anticipated service temperature and 20 ft-lbs at 0°F. Refer to *FEMA-353*. Joints in this category include all complete joint penetration groove welds in beam and column flanges and webs, welds of continuity plates to column flanges and similar elements subject to large cyclic stresses at or near plastic levels.

- All welds for the frame girder-column joints should be started and ended on weld run-off tabs where practical. All weld tabs should be removed, the affected area ground smooth and tested for defects using the magnetic particle method. Acceptance criteria should be per *AWS D1.1*, Section 8.15.1. Surface imperfections less than 1/16 inch in dimension should be removed by grinding. Deeper gouges, areas of lack of fusion, and slag inclusions, for example, should be removed by gouging or grinding and rewelding following the procedures outlined above.

- Weld tabs should conform to the requirements of *AWS D1.1*. End dams should not be permitted.

- Steel backing (backing bars), if used, should be removed from new and/or repaired welds at the girder bottom flange, the weld root back-gouged by air arcing and the area tested for defects using the magnetic particle method, as described above. The weld should be completed and reinforced with a fillet weld. Removal of the weld backing at repairs of the top girder flange weld may be considered, at the discretion of the structural engineer.

6.5.2 Removal of Backing

Prior to removing weld backing on existing joints, the contractor should prepare and submit a written WPS for review by the structural engineer. The WPS should conform to the requirements of *AWS D1.1*. In addition the contractor should propose the method(s) that will be used to remove the weld backing, back gouge to sound metal and when during this process preheat will be applied.

Although project conditions may vary, steel backing may be removed either by grinding or by the use of air arc, or oxy-fuel gouging. The zone just beyond the theoretical 90 degree intersection of the beam to column flange should be removed by either air arc or oxy-fuel gouging 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. 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: Only removal of backing from the bottom-beam-flange-to-column joint is recommended. Removal of the weld backing from the top flange may be difficult, particularly along perimeter frames where access to the outer side is*

*restricted. Tests conducted to date have not been conclusive with regard to the
benefit of top flange weld backing removal, and therefore, this is not generally
recommended. There is no direct evidence that removal of weld backing from
continuity plates in the column panel zone is necessary.*

6.5.3 Removal of Weld Tabs

If weld tabs were used and are to be removed in conjunction with the removal of the weld
backing, the tabs should be removed after the weld backing has been removed and fillet added.
Weld tabs may be removed by air arc or oxy-fuel gouging followed by grinding or by grinding
alone. The resulting contour should blend smoothly with the face of the column flange and the
edge of the beam flange and should have a radius of 1/4-3/8 inch. The finished surface should be
visually inspected for contour and any visually apparent indications. This should be followed by
MT. Linear indications found in this location of the weld may be detrimental. They may be the
result of the final residue of defects commonly found in the weld tab area. Linear indications
should be removed by lightly grinding or using a cutting tool until the indication is removed. If
after removal of the defect the ground area can be tapered and is not beyond the theoretical 90
degree intersection of the beam flange edge and column flange, weld repair may not be necessary
and should be avoided if possible.

Existing end dams, if present, should be removed from joints undergoing repair. Prior to
removal of end dams, the contractor should submit a removal / repair plan which lists the method
of dam removal, defect removal, and welding procedure including, process, preheat, and joint
configuration. The tab may be removed by grinding, air arc or oxy-fuel torch.

6.5.4 Defect Removal

Any rejectable weld defects should be removed by grinding or cutting tools, or by air arc
gouging followed by grinding. The individual performing defect removal should be furnished
the ultrasonic testing (UT) results which describe the location depth and extent of the defect(s).

If defect removal extends into the theoretical weld section, weld repair may be necessary.
The weld repair should be performed in accordance with the contractor's WPS, with strict
adherence to the preheat requirements. The surface should receive a final visual inspection and
MT after all repairs and surface conditioning has been completed.

When the individual removing the defects has completed this operation, and has visually
confirmed that no remnants remain, the surface should be tested by MT. Additional defect
removal and MT may occur until the MT tests reveal that the defects have been removed.

The contour of the surface at this point may be too irregular in profile to allow welding to
begin. The surface should be conditioned by grinding or using a cutting tool to develop a joint
profile that conforms to the WPS. Prior to welding, MT should be performed to determine if any
additional defects have been exposed.

Based upon a satisfactory MT the joint may be prepared for welding. Weld tabs (and backing if necessary) should be added. The welding may begin and proceed in accordance with the WPS. The theoretical weld must be completed for its full height and length. Careful attention should be paid to ensure that individual weld bead size does not exceed that permitted by the WPS.

If specified, the weld tabs and backing should be removed in accordance with the guideline section describing this technique. The final weld should be inspected by MT and UT.

6.5.5 Girder Repair

If at bottom flange repairs back gouging removes sufficient material such that a weld backing is required for the repair, after welding the backing should be removed from the girder. Alternatively, a double-beveled joint may be used The weld root should be inspected and tested for imperfections, which if found, should be removed by back-gouging to sound material. A reinforcing fillet weld should be placed at "T" joints. The reinforcing fillet should have a size equal to one-quarter of the girder flange thickness. It need not exceed 3/8 inch (see Note J, Figure 3.4 of *AWS D1.1*.)

If the bottom flange weld requires repair, the following procedure may be considered:

1. The root pass should not exceed a 1/4 inch bead size.

2. The first half-length root pass should be made with one of the following techniques, at the option of the contractor:

    (a) The root pass may be initiated near the center of the joint. If this approach is used, the welder should extend the electrode through the weld access hole, approximately 1 inch beyond the opposite side of the girder web. This is to allow adequate access for clearing and inspection of the initiation point of the weld before the second half-length of the root pass is applied. It is not desirable to initiate the arc in the exact center of the girder width since this will limit access to the start of the weld during post-weld operations. After the arc is initiated, travel should progress towards the end of the joint (outboard beam flange edge), and the weld should be terminated on a weld tab.

    (b) The weld may be initiated on the weld tab, with travel progressing toward the center of the girder flange width. When this approach is used, the welder should stop the weld approximately 1 inch before the beam web. It is not advisable to leave the weld crater directly in the center of the beam flange width since this will hinder post-weld operations.

3. The half-length root pass should be thoroughly slagged and cleaned.

4. The end of the half-length root pass that is in the vicinity of the center of the beam flange should be visually inspected to ensure fusion, soundness, freedom from slag inclusions and excessive porosity. The resulting bead profile should be suitable for obtaining fusion by the subsequent pass to be initiated on the opposite side of the girder web. If the profile is not conducive to good fusion, the start of the first root pass should be ground, gouged, chipped or otherwise prepared to ensure adequate fusion.

5. The second half of the weld joint should have the root pass applied before any other weld passes are performed. The arc should be initiated at the end of the half-length root pass that is near the center of the beam flange, and travel should progress to the outboard end of the joint, terminating on the weld tab.

6. Each weld layer should be completed on both sides of the joint before a new layer is deposited.

7. Weld tabs should be removed and ground flush to the beam flange. Imperfections less than 1/16 inch should be removed by grinding. Deeper gouges, areas of lack of fusion, and slag inclusions, for example, should be removed by gouging or grinding and rewelding following the procedures outlined above.

6.5.6   Weld Repair (Types W2, or W3 and Defects)

When W2, or W3 cracks are found, the column base metal should be evaluated using UT to determine if fractures have progressed into the flange. This testing should be performed both during the period of discovery and during repair. Similar procedures should be followed when making repairs to defects at weld roots.

When a linear planar-type defect such as a crack or lack of fusion can be determined to extend beyond one-half the thickness of the beam flange, it is generally preferred to use a double-sided weld for repair (even though the fracture may not extend all the way to the opposite surface.) This is because the net volume of material that needs to be removed and restored is generally less when a double-sided joint is utilized. It also results in a better distribution of residual stresses since they are roughly balanced on either side of the center of the flange thickness.

Repair of cracks and defects in welds may warrant total removal of the original weld, particularly if multiple cracks are present. If the entire weld plus some base metal is removed, care must be taken not to exceed the root opening and bevel limits of *AWS D1.1* unless a qualified by test WPS is used. If this cannot be avoided one of two options is available:

1. The beveled face of the beam and/or the column face may be built up (buttered) until the desired root opening and angle is obtained.

2. A section of the flange may be removed and a splice plate inserted.

   Commentary: Building up base metal with welding is a less intrusive technique than removing large sections of the base metal and replacing with new plate. However, this technique should not be used if the length of build-up exceeds the thickness of the plate.

6.5.7   Weld Overlays

An alternative approach that can be considered when repairing or reinforcing pre-Northridge beam-flange to column-flange welded connections is to apply a weld overlay. This procedure is suitable for repairing beam-flange welds that have been classified as having rejectable weld

defects, as opposed to fractures. This application consists of encapsulating the existing low toughness weld metal with material with a higher notch toughness. The high notch toughness weld metal overlay is very resistant to fracture initiation from surface discontinuities within the range of defects that would go undetected, effectively converts surface defects and small fractures in the existing CJP weld into internal defects, and further, provides local reinforcement of the joint for resistance of applied stresses. This repair approach was developed by an independent task group of engineers and researchers in the Los Angeles area, following the 1994 Northridge earthquake. A schematic arrangement of a weld overlay is shown in Figure 6-12.
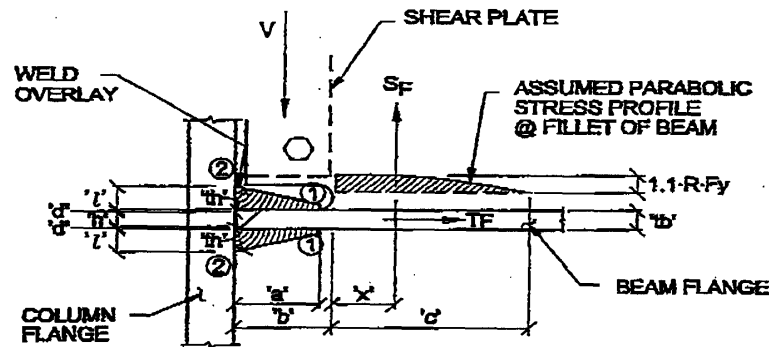
Nomenclature:

F_{yb} = Minimum specified yield stress of beam (force / unit area)

F_{yc} = Minimum specified yield stress of column (force / unit area)

V = Total Shear acting on connection based upon nominal beam flexural strength at the plastic hinge location (i.e., $\dfrac{1.1R_y 2M_p}{L_c} + V_{gravity}$, where L_c = clear distance between column flanges)

R_y = Ratio of Expected Yield Strength F_{yc} to the minimum specified yield strength F_{yb}

S_F = Total shear applied to each flange. (force)

T_F = Total tension applied to each flange. (force)

h = Height of flaw assumed over full width of flange. (dimension)

Figure 6-12 Weld Overlay Repair of Beam Flange to Column Flange Joint

The design of the weld overlay is based on the premise that, in addition to transferring the flange flexural force to the column, the beam shear force must also be transferred through the beam flange weld. Based on physical testing of typical connections, supplemented by finite element analysis the stress distribution across the beam-flange is assumed to be parabolic as shown in Figure 6-13. Design procedures have been developed for two overlay conditions: Class A, which requires that the overlay take the full connection demand; and Class C, which assumes that the remaining existing weld has 50% of its original design capacity remaining. The throat thickness of the weld overlay, which may be regarded as an elongated fillet weld, is determined from geometric considerations similar to standard fillet welds.
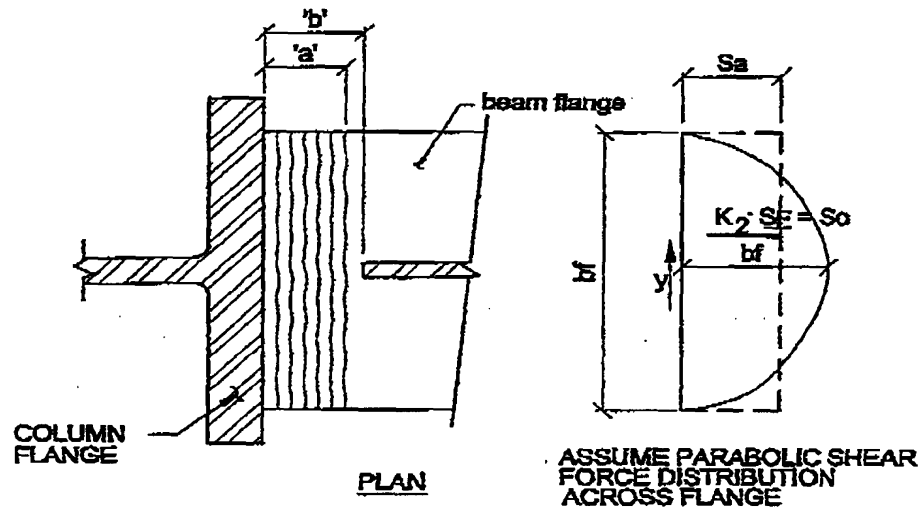
Figure 6-13   Plan View and Assumed Stress Distribution for Weld Overlay Design

Commentary: Independent research into the performance of weld overlay repairs indicates that beam-column connections repaired using this technique are substantially more rugged than typical unreinforced connections employing low toughness weld metals. Refer to Anderson, et al. (2000) for more detailed information on this technique.

6.5.8   Column Flange Repairs - Type C2

Damage type C2 is a pullout type failure of the column flange material. The failure surface should be conditioned to a concave surface by grinding and inspected for soundness using MT. The concave area may then be built up by welding (buttering). The joint contour described in the WPS should specify a "boat shaped" section with a "U" shaped cross section and tapered ends. The weld passes should be horizontal stringers placed in accordance with the WPS. Since stop/starts will occur in the finished weld, care must be taken to condition each stop/start to remove discontinuities and provide an adequate contour for subsequent passes. The final surface should be ground smooth and flush with the column face. This surface and immediate surrounding area should be subjected to magnetic particle testing and ultrasonic testing.

APPENDIX A: DETAILED PROCEDURES FOR PERFORMANCE EVALUATION

A.1   Scope

This appendix provides detailed procedures for evaluating confidence levels associated with the ability of damaged WSMF structures to resist collapse in levels of ground shaking likely to occur in the period immediately following a major earthquake. These detailed procedures are provided as a supplement to the level 2 evaluation procedures in Chapter 5. They may be used to demonstrate enhanced levels of confidence with regard to the ability of a particular damaged structure to resist collapse, relative to the confidence levels that may be derived using the more simplified procedures of Chapter 5. The procedures of this appendix are required as a supplement to the Chapter 5 procedures for structures that are classified as irregular, considering the effects of the damage sustained.

> *Commentary: Chapter 5 provides the basic procedures for a level 2 evaluation, using factored demand-to-capacity ratios to indicate a level of confidence with regard to a damaged building's ability to resist collapse in that level of ground shaking likely to occur in the period immediately following a damaging earthquake. The tabular values of demand and resistance factors and confidence indices contained in Chapter 5 were derived using the procedures presented in this appendix, applied to the performance evaluation of a suite of 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 building configurations, that is, irregular structures not well represented by the model buildings used as the basis for the values contained in Chapter 5.*

A.2   Performance Evaluation Approach

A.2.1 Confidence of Ability to Withstand Collapse

The evaluation procedures contained herein permit estimation of a level of confidence associated with the ability of a damaged building to withstand collapse for the levels of ground shaking likely to be experienced within a year following a damaging earthquake.

> *Commentary: The probability that a damaged building may experience collapse as a result of ground shaking likely to occur in the year following a damaging event is calculated as the integral over a year's time of the probability that damage will exceed the collapse capacity of the structure. Mathematically, this may be expressed as:*

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

where:

$P(D>PL)$ = *Probability of damage exceeding the collapse level in a period of "t" years, taken as 1 year*

$P_{D>PL}(x)$ = *Probability of damage exceeding a collapse 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 1 year following the first damaging event*

Vulnerability may be thought of as the capacity of the structure to resist collapse. 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 inelastic deformations. Ground accelerations associated with the seismic hazard, and the resulting 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 the probability that damage could exceed a collapse 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 expressed 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 evaluation procedures of Chapter 5 are based on the typical characteristics of standard buildings. Since they are based on the capacity characteristics of typical rather than specific structures, the procedures contained in Chapter 5 inherently 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 procedures of Chapter 5, it may be found that for a specific structure, there is only a 30% level of confidence that the structure is capable of resisting collapse for the levels of ground shaking likely to be experienced in a period of a year following a damaging event. This rather low level of confidence may be more a function of the uncertainty inherent in the procedures used to estimate the probability of collapse than the actual inadequate capacity of the building to resist collapse. 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 an 80 % or 90% level of confidence, in resisting collapse. The difference in such findings can mean the difference between deciding that a building must be vacated or that it can continue to be occupied.

In both the procedures of this appendix and Chapter 5, the uncertainties associated with estimation of the intensity of ground motion have been neglected. These uncertainties can be high, on the order of those associated with structural performance. Thus, the confidence estimated using these procedures is really a confidence with regard to structural performance, given an assumed seismicity, dominated by a single event, consisting of a repeat of the original damaging event, within a period of a year following the initial damaging shock. It is believed that this assumed seismicity is conservative, but credible.

A.2.2 Basic Procedure

As indicated in Chapter 5, a demand and resistance factor design (DRFD) format is used to associate a level of confidence with the probability that a building will be able to resist collapse in the level of ground shaking anticipated in the year following a damaging earthquake. The basic approach is to determine a confidence parameter, λ, which may then be used, with reference to Table 5-7, 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 the tables of Chapters 5, or by more rigorous direct calculation of capacity using the procedures of this appendix.

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

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

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

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

λ =   a confidence index parameter from which a level of confidence can be obtained by reference to Table 5-7.

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 $P\text{-}D$ instability and collapse. It is also closely related to plastic rotation demand, or drift angle demand, on individual beam-column connection assemblies, and therefore a good predictor of the performance of beams, columns and connections. Other parameters used in these guidelines include column axial compression and column axial tension. In order to determine a level of confidence with regard to the ability of a building to resist collapse for the level of ground shaking likely to occur in the year immediately following an earthquake, the following steps are followed:

1. **A best estimate of the ground shaking intensity that caused the initial damage in the building is developed.** This can be done by reference to instrumental recordings of ground motion at the building or nearby sites, the use of standard attenuation relations, or ground shaking contour maps. For the purpose of this evaluation, it is assumed that an event of similar intensity at the building site is likely to occur within a one year period. Ground shaking intensity should be characterized by a 5% damped elastic response spectrum.

2. **A mathematical structural model is developed to represent the damaged building structure.** Note that since damage can result in unsymmetrical structural response, it may be necessary to develop several models. The model(s) are then subjected to a structural analysis, using any of the methods contained in Chapter 5. This analysis predicts the median estimates of maximum interstory drift demand, maximum column compressive demand, and maximum column-splice tensile demand, for the assumed repeat of the original damaging earthquake.

3. **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 5 for regular structures, or the detailed

procedures of Section A.6 of this appendix may be used. These detailed procedures are required for irregular structures or for regular structures that have been made irregular by the damage they have sustained. Interstory drift capacity for moment-resisting connections may be estimated using the default values of Chapter 5, for typical connection types, or 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 5.

4. **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 5 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 that chapter. If the procedures of this appendix are used to determine median estimates of demand or capacity, then the corresponding demand and resistance factors should be determined in accordance with the applicable procedures of this appendix.

5. **Evaluate confidence.** The confidence obtained with regard to the ability of the structure to meet the performance objective is determined using the lowest of the λ values determined in accordance with Step 4 above, back-calculated from the equation:

$$\lambda = e^{-b\beta_{UT}(K_X - k\beta_{UT}/2)}$$ (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 variation 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. This is taken as having a value of 5, representative of the assumed seismicity for the year following a damaging earthquake,

K_X = standard Gaussian variate associated with the 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 5-7 provides a solution for this equation, for various values of the parameters λ and β_{UT}.

The values of the parameter b_{UT} used in Equation A-3 and Table 5-7 are used to account for the uncertainties inherent in the estimation of demands and capacities. Uncertainty enters the process through a variety of assumptions that are made in the performance evaluation process, including for example assumed values of damping, structural period, properties used in structural modeling, and strengths of materials. Assuming that the amount of uncertainty introduced by each of the assumptions can be characterized, the parameter b_{UT} can be calculated using the equation:

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

where b_{u_i} 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.5 and A.6 indicate how to determine b_{u_i} values associated with demand estimation, beam-column connection assembly behavior, and building global stability capacity prediction respectively.

A.3   Determination of Hazard Parameters

In order to implement these postearthquake evaluation procedures, it is necessary to obtain an estimate of the 5% damped, linear response spectrum for the original damaging earthquake, and to obtain from that response spectrum, an estimate of the spectral response acceleration, S_{aT1} at the fundamental period of the damaged building.

A.4   Determination of Demand Factors

The demand variability factor, g, and analytical uncertainty factor, g_a, are used to adjust the calculated interstory drift, column axial load and column splice tension demands to their mean values, considering the variability and uncertainty inherent in drift demand prediction and probable intensity of ground shaking during the year following the initial damaging earthquake.

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 demands 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. From these analyses, statistics are developed for the median value and standard deviation of the natural logarithm of the various demand parameters including maximum interstory drift, column axial load, and column-splice tension. These standard deviations of the natural logarithms of these response parameters are denoted b_{DR}.

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

$$g = e^{\frac{k}{2b}b_{DR}{}^2}$$ (A-5)

where:

- k is the logarithmic slope of the hazard curve, taken as having a value of 5.

- 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 b_{DU}.

This parameter is used to calculate the analytical uncertainty factor, g_a.

In addition to uncertainty in demand prediction, the analytical uncertainty factor g_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:

$$C_B = \frac{\text{demand predicted by nonlinear time history analysis}}{\text{demand predicted by analysis method}}$$ (A-6)

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 b_{DU}.

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

$$g_a = C_B e^{\frac{k}{2b}b_{DU}{}^2}$$ (A-7)

The analysis uncertainty factors presented in Chapter 5 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, b_{DU}, used as a basis for the analysis uncertainty factors presented in Chapter 5 are summarized in Table A-1. The bias factors C_B used in Chapter 5 are summarized in Table A-2. It is recommended that these default values for b_{DU} and C_B be used for all buildings.

Table A-1   Default Logarithmic Uncertainty b_{DU} for Various Analytical Methods

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

Table A-2   Default Bias Factors C_B

	Analysis Procedure			
	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Type 1 Connections				
Low Rise (<4 stories)	0.65	0.80	0.85	1.00
Mid Rise (4 – 12 stories)	0.85	1.15	0.95	1.00
High Rise (> 12 stories)	1.0	1.0	0.85	1.00
Type 2 Connections				
Low Rise (<4 stories)	0.90	1.20	1.25	1.00
Mid Rise (4 – 12 stories)	1.00	1.30	1.35	1.00
High Rise (> 12 stories)	0.70	1.20	1.30	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, contained in Chapter 5 be used for most structures. The procedures*
> *contained in this appendix are most useful for calculating capacities and capacity*
> *factors.*

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 deformations 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 is the preferred demand parameter for reporting laboratory data. This parameter is less subject to interpretation by various testing laboratories and also has the advantage that it is approximately equal to the interstory drift angle predicted by linear structural analyses. Refer to *FEMA 350*, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings* for additional information on laboratory testing protocols and parameters for reporting test behavior.

Median drift angle capacities *C* and resistance factors *f* for common connection types are presented in Chapter 5. These values were determined from cyclic tests of full-size connection assemblies using the testing protocols indicated below. 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. degradation of moment-resisting capacity of the assembly to a value below the nominal moment-resisting capacity,

2. initiation of fracture of bolts, welds, or base metal that results in significant strength degradation of the assembly, and

3. complete failure of the connection, characterized by an inability of the connection to maintain integrity of the beam to column connection under gravity loading.

Based on this data, drift angle statistics are obtained for a state of incipient collapse, q_u. The quantity q_u, the *ultimate* capacity of the connection, which occurs at damage state 3, above, is used to evaluate the acceptability of local connection behavior.

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 (Krawinkler et al., 1997) 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 proximate 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.

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

Median drift angle capacities for the quantity q_U, should be taken directly from available laboratory data. The median value should be taken as that value from all of the available tests that is not exceeded by 50% of the tests. The value of the quantity f should be determine by the following procedure.

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

$$b = \sqrt{\frac{\sum_{i=1}^{n}\left(\ln x_i - \overline{\ln x_i}\right)^2}{n-1}} \tag{A-8}$$

where:

b =   the standard deviation of the natural logarithms of the test data

x_i =   individual test data value

n =   the number of tests from which data is available

$\overline{\ln x_i}$ =   the mean of the logarithms of the x_i values,

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

$$f_R = e^{-2.5b^2} \tag{A-9}$$

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

$$f = f_U f_R = 0.9 f_R \tag{A-10}$$

where:

f_R = the resistance factor accounting for random behavior

f_U = the resistance factor accounting for uncertainty in the relationship between laboratory findings and behavior in real buildings and assumed in these recommended criteria to have a logarithmic standard deviation b_U of 0.2.

A.6   Global Stability Capacity

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

The global stability limit is determined using the Incremental Dynamic Analysis (IDA) technique (Cornell, 1999). 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 non-linear behavior of the structure. Perform a nonlinear time history analysis of the building for this scaled accelerogram. Determine the S_{aT1} for this scaled accelerogram and the maximum predicted interstory drift from the analysis. Plot this point on the graph. Call this point D_1.

4. Increase the scaling amplitude of the accelerogram slightly and repeat Step 3. Plot this point as D_2. Draw a straight line between points D_1 and D_2.

5. Repeat Step 4 until the straight line slope between consecutive points D_i and D_{i+1}, is less than $0.2\,S_e$. When this condition is reached, D_{i+1} is the global drift capacity for this accelerogram. If $D_{i+1} \geq 0.10$ then the drift capacity is taken as 0.10. Figure A-1 presents a typical series of plots obtained from such analyses.

6. Repeat Steps 2 through 5 for each of the accelerograms in the suite selected as representative of the site and hazard and determine an interstory drift capacity for the structure for each accelerogram.

7. Determine a median interstory drift capacity for global collapse C as the median value of the calculated set of interstory drift capacities, determined for each of the accelerograms. Note that the median value is that value exceeded by 50% of the accelerograms.
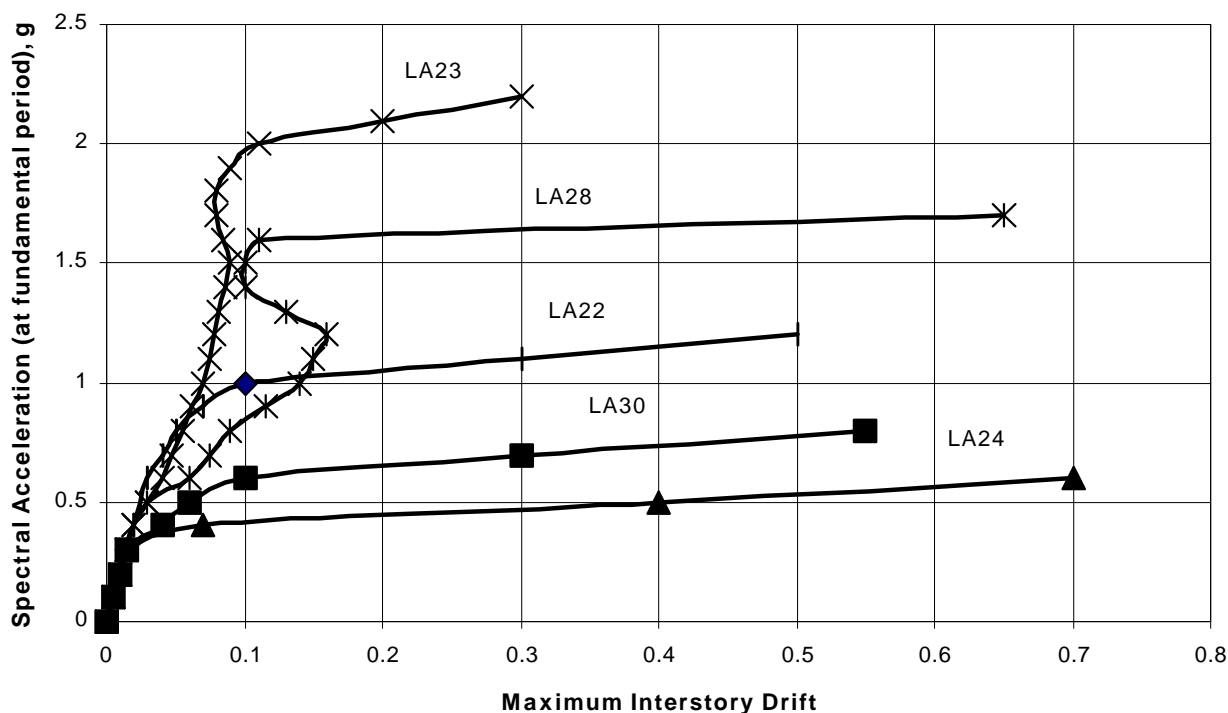
Figure A-1 Representative Incremental Dynamic Analysis Plots

8. Determine a logarithmic standard deviation for random differences in ground motion accelerograms, b_R, using Equation A-8 of Section A.5.2. In this equation, x_i is the interstory

drift capacities predicted for the ith accelerogram, and n is the number of accelerograms contained in the analyzed suite.

9. Calculate the global resistance factor $\boldsymbol{f}_R$ due to randomness in the predicted global collapse capacity for various ground motions from the equation:

$$\boldsymbol{f}_R = e^{-\frac{k}{2b}\boldsymbol{b}^2} \qquad\qquad (A-10)$$

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

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

$$\boldsymbol{f} = \boldsymbol{f}_U \boldsymbol{f}_R = e^{-\frac{k}{2b}\boldsymbol{b}_U^{\;2}}\boldsymbol{f}_R \qquad\qquad (A-11)$$

where:

$\boldsymbol{f}_R$ is the global resistance factor due to randomness determined in Step 9.

$\boldsymbol{b}_U$ is the logarithmic standard deviation related to uncertainty in the 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 steel moment 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 structures. 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-$\boldsymbol{D}$ instabilities to develop. Prediction of the onset of P-$\boldsymbol{D}$ instability due to ground shaking is quite complex. It is possible that an acceleration record will displace a structure to a point where static P-$\boldsymbol{D}$ instability would initiate, only to bring the structure back again before collapse can occur, due to a reversal in ground shaking direction.*
>
> *The basic effect of P-$\boldsymbol{D}$ instability is that a negative stiffness is induced in the structure. That is, P-$\boldsymbol{D}$ effects produce a condition in which increased displacement can occur at 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
(IDA) 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. Increasing the amount of
dead load on the structure, to produce artificially the appropriate negative
stiffness, can account approximately for these effects.*

APPENDIX B: SAMPLE PLACARDS

B.1   "Inspected" Placard

Title:
(Iplacard.eps)
Creator:
Adobe Illustrator(R) 8.0
Preview:
This EPS picture was not saved
with a preview included in it.
Comment:
This EPS picture will print to a
PostScript printer, but not to
other types of printers.

Notes:

1. Recommended placard color: green.

2. Use for posting conditions Green-1, Green-2, and Green-3 of Table 3-2.

3. Placard is from ATC (1995)

B.2 "Restricted Use" Placard

Title:
(Rplacard.eps)
Creator:
Adobe Illustrator(R) 8.0
Preview:
This EPS picture was not saved
with a preview included in it.
Comment:
This EPS picture will print to a
PostScript printer, but not to
other types of printers.

Notes:

1. Recommended placard color: yellow.

2. Use for posting conditions Yellow-1 and Yellow -2 of Table 3-2.

3. Placard is from ATC (1995)

B.3   Modified "Restricted Use" Placard
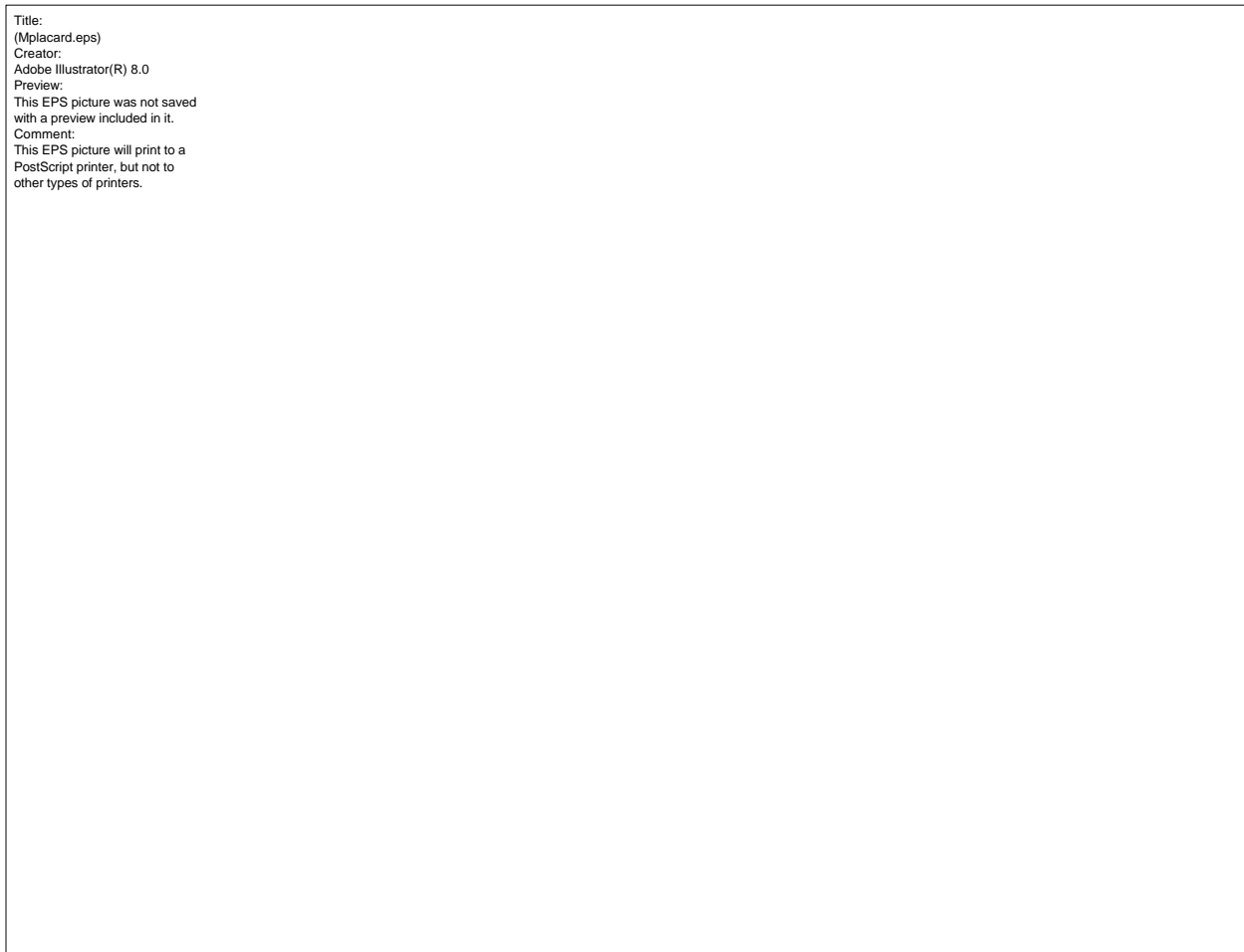
Title:
(Mplacard.eps)
Creator:
Adobe Illustrator(R) 8.0
Preview:
This EPS picture was not saved
with a preview included in it.
Comment:
This EPS picture will print to a
PostScript printer, but not to
other types of printers.

Notes:

1. Recommended placard color: yellow.

2. Use for posting conditions Yellow-1 and Yellow -2 of Table 3-2.

3. Placard is from ATC (1995)

B.4   "Unsafe" Placard

Title:
(Uplacard.eps)
Creator:
Adobe Illustrator(R) 8.0
Preview:
This EPS picture was not saved
with a preview included in it.
Comment:
This EPS picture will print to a
PostScript printer, but not to
other types of printers.

Notes:

4.   Recommended placard color: red.

5.   Use for posting conditions Red-1 and Red –2 of Table 3-2.

6.   Placard is from ATC (1995)

APPENDIX C: SAMPLE INSPECTION

Figure C-1   Inspection Form – Major Axis Column Connection

SUPPLEMENTAL SKETCH FOR LARGE DISCONTINUITIES

NOTE: LOCATE BY SKETCH, WITH DIMENSIONS, **ALL** CRACKS
AND DISCONTINUITIES FOUND BY U.T. INSPECTION.

BEAM FLANGE

CONTINUITY PLATES
AS OCCURS

BEAM WEB

BEAM TOP FLANGE – (TOP FACE)

COLUMN FLANGE

BEAM TOP FLANGE – SECTION

BEAM FLANGE

CONTINUITY PLATES
AS OCCURS

BEAM WEB

BEAM BOTTOM FLANGE – (TOP FACE)

COLUMN FLANGE

BEAM BOTTOM FLANGE – SECTION
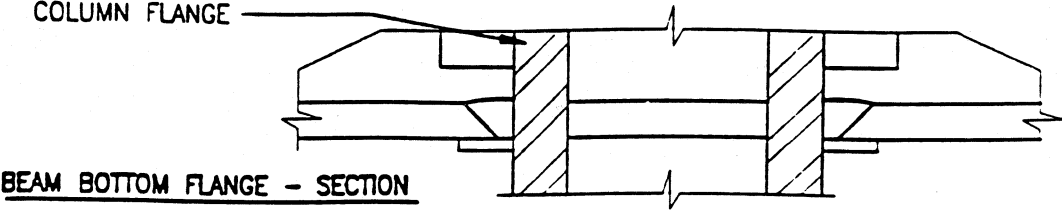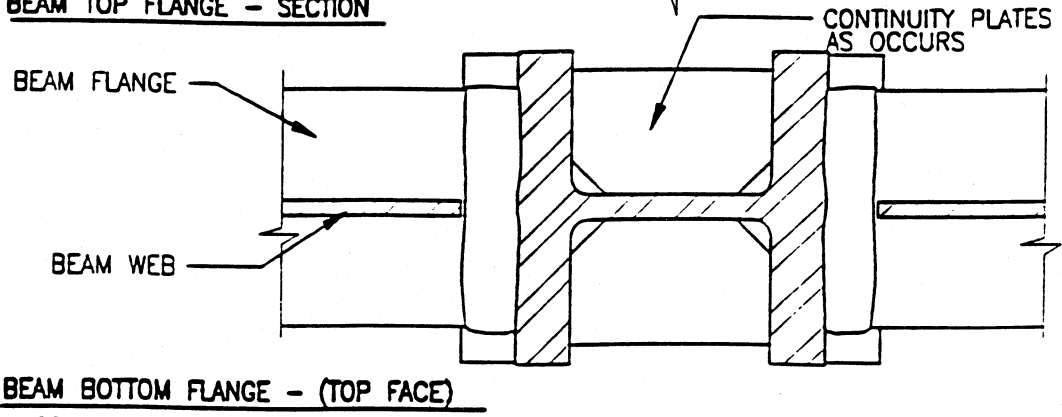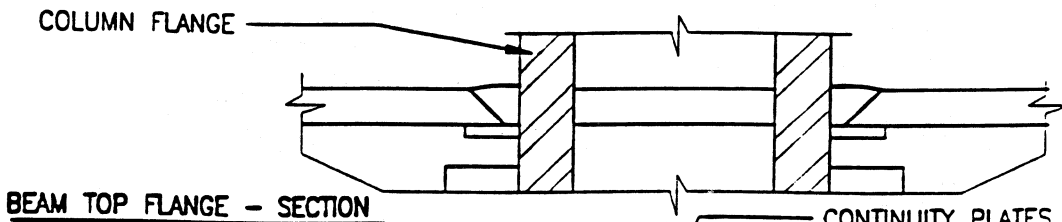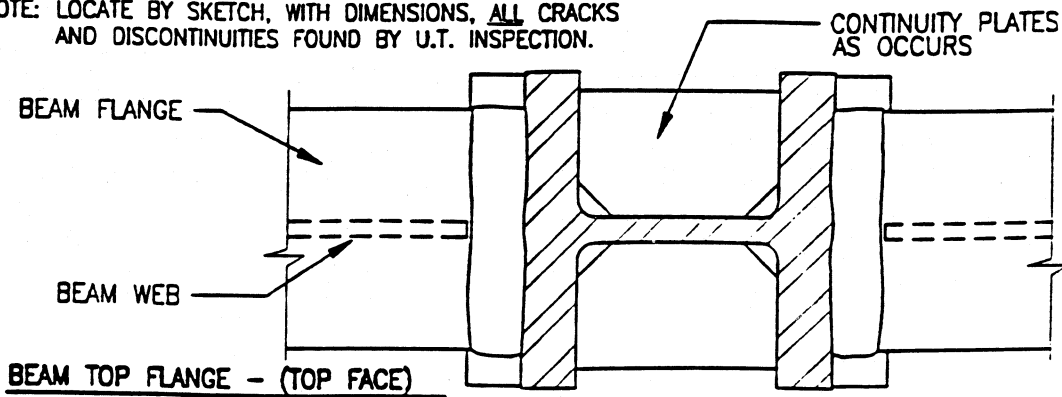
ADDITIONAL COMMENTS: _____

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Figure C-2   Inspection Form – Large Discontinuities – Major Axis

Figure C-3   Inspection Form – Minor Axis Column Connection

SUPPLEMENTAL SKETCH FOR LARGE DISCONTINUITIES

NOTE: LOCATE BY SKETCH, WITH DIMENSIONS, ALL CRACKS
AND DISCONTINUITIES FOUND BY U.T. INSPECTION.

CONTINUITY PLATES
AS OCCURS

BEAM FLANGE

BEAM WEB

BEAM TOP FLANGE – TOP FACE

COLUMN FLANGE

BEAM TOP FLANGE – SECTION

CONTINUITY PLATES
AS OCCURS

BEAM FLANGE

BEAM WEB

BEAM BOTTOM FLANGE – TOP FACE

COLUMN FLANGE

BEAM BOTTOM FLANGE – SECTION
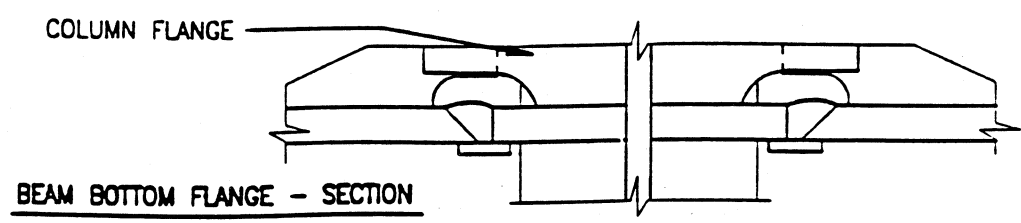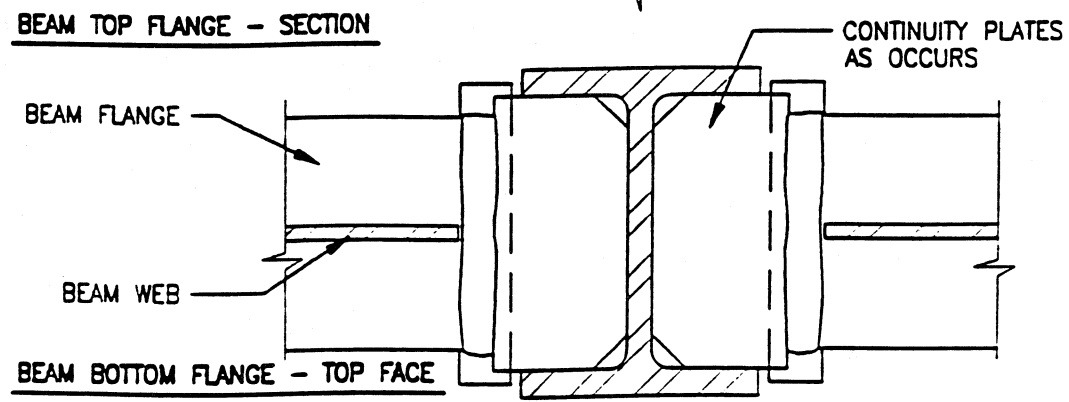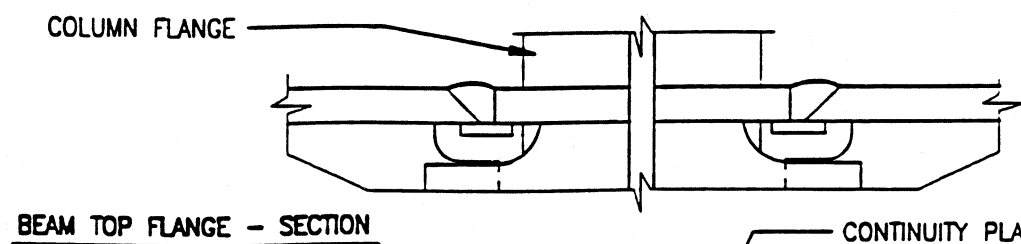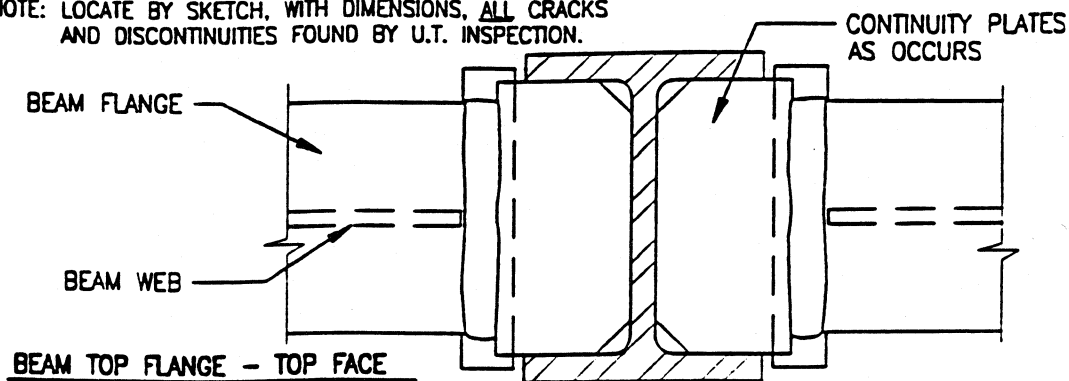
ADDITIONAL COMMENTS: _____

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_____

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Figure C-4   Inspection Form – Large Discontinuities – Minor Axis

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.

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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

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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 –
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