

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.

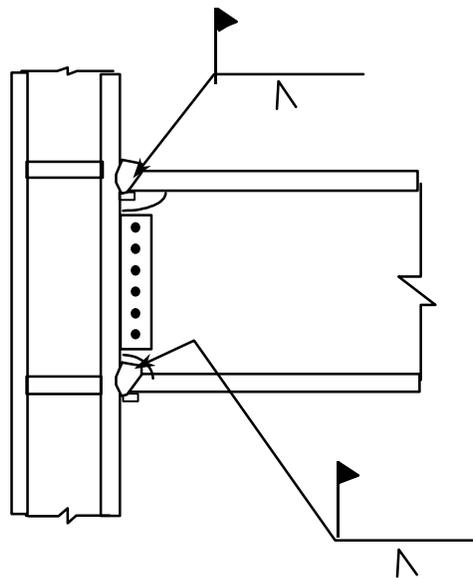


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

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.

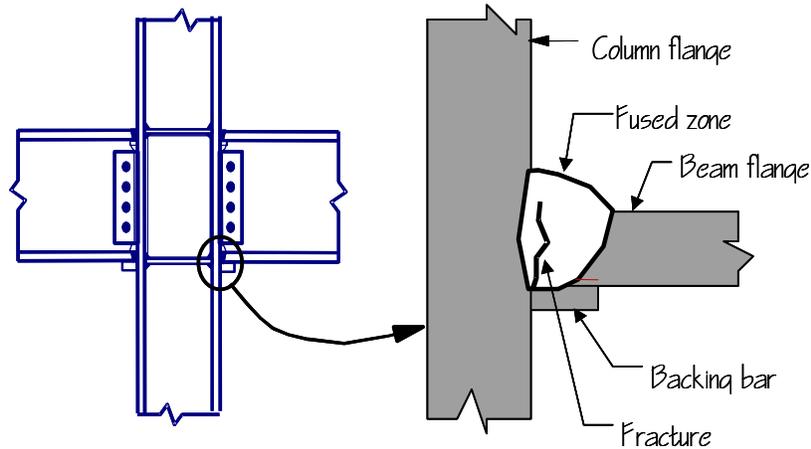


Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection



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.

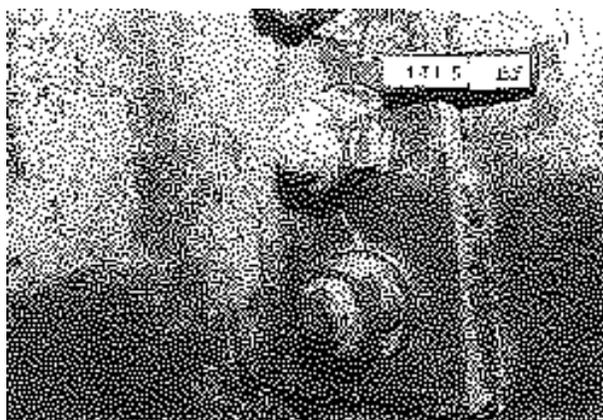


Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

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

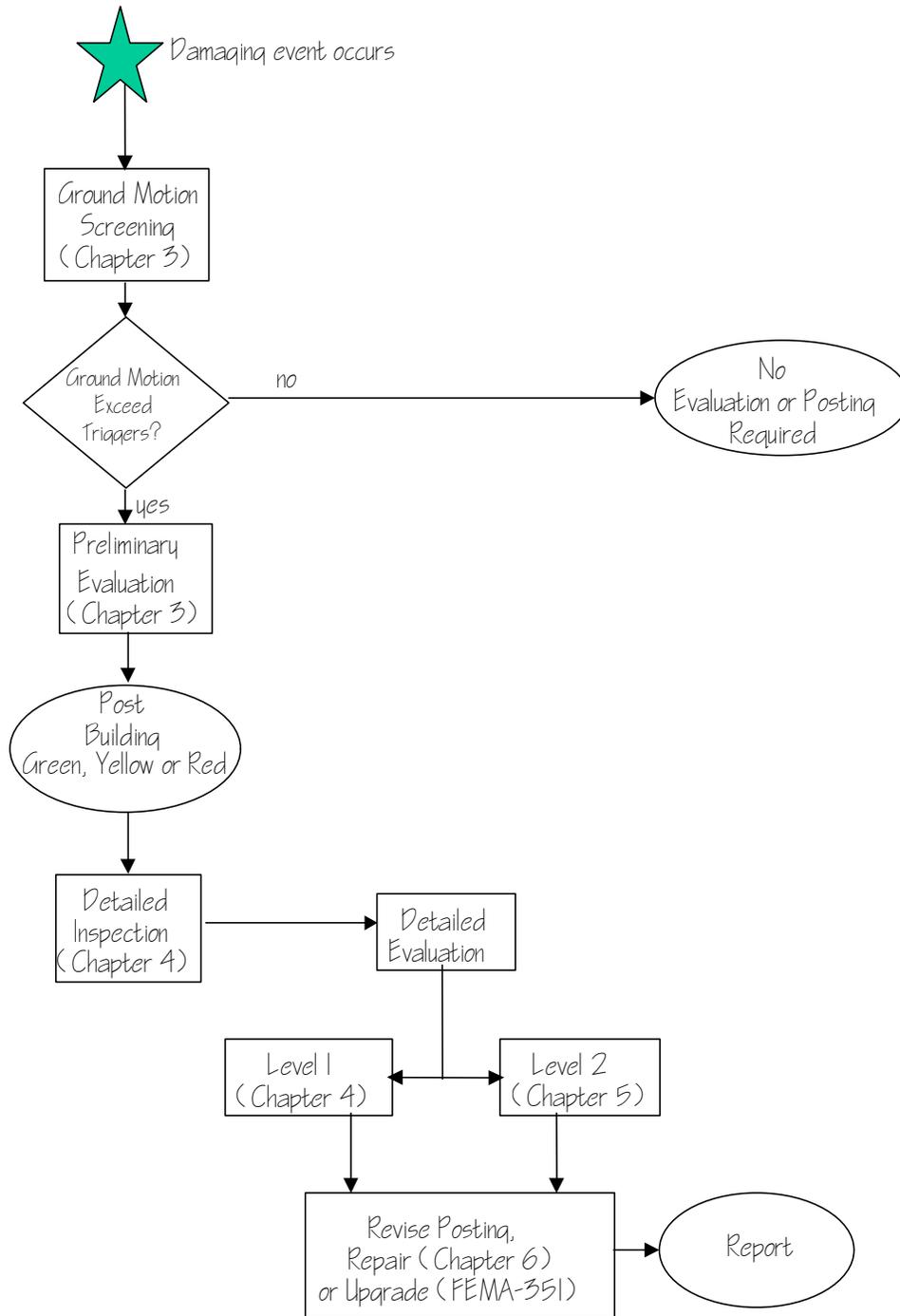


Figure 1-6 Flow Chart for Postearthquake Actions

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.**