Regular Meeting

of the

Building Inspection Commission

October 15, 2025

Agenda Item 6

APPENDIX A – MODIFICATIONS TO FEMA 352 FOR APPLICATION OF THIS BULLETIN

FEMA 352 will form the basis for application of this administrative bulletin. Modifications to the FEMA 352 document have been developed specifically for use with this administrative bulletin and are provided in this appendix for use by engineers and DBI in order to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

Also note that the FEMA 352 document includes guidance on post-earthquake posting of Pre-Northridge Welded Steel Moment Frame Buildings. This bulletin does not address post-earthquake posting, so any reference to that guidance is not a part of the bulletin. At its discretion, DBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status/condition designations or posting categories. Locations in this Appendix that pertain to post-earthquake posting will be identified.

FEDERAL EMERGENCY MANAGEMENT AGENCY FEMA-352 / June, 2000
WITH SF AMENDMENTS

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

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

DISCLAIMER

NOTE FOR USE OF FEMA 352: As a resource for application of City of San Francisco Administrative Bulletin AB-XXX “Post-Earthquake Inspection, Evaluation, Repair and Retrofit Requirements for “Pre-Northridge” Welded Steel Moment Frame Buildings”: FEMA 352 will form the basis for application AB XXX. Modifications to the document are provided for use by engineers and SFDBI to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

Also note that the FEMA 352 document includes guidance on post-earthquake posting of “Pre-Northridge” Welded Steel Moment Frame Buildings. AB-XXX does not address post-earthquake posting, so any reference to that guidance is not a part of the AB. At its’ discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status/condition designations or posting categories. Locations in this Appendix to AB-XXX that pertain to post-earthquake posting will be identified.

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

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

SAC Joint Venture

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

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June, 2000

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.

ACKNOWLEDGEMENTS

Funding for Phases I and II of the SAC Steel Program to Reduce the Earthquake Hazards of Steel Moment-Frame Structures was principally provided by the Federal Emergency Management Agency, with ten percent of the Phase I program funded by the State of California, Office of Emergency Services. Substantial additional support, in the form of donated materials, services, and data has been provided by a number of individual consulting engineers, inspectors, researchers, fabricators, materials suppliers and industry groups. Special efforts have been made to maintain a liaison with the engineering profession, researchers, the steel industry, fabricators, code-writing organizations and model code groups, building officials, insurance and risk-management groups, and federal and state agencies active in earthquake hazard mitigation efforts. SAC wishes to acknowledge the support and participation of each of the above groups, organizations and individuals. In particular, we wish to acknowledge the contributions provided by the American Institute of Steel Construction, the Lincoln Electric Company, the National Institute of Standards and Technology, the National Science Foundation, and the Structural Shape Producers Council. SAC also takes this opportunity to acknowledge the efforts of the project participants – the managers, investigators, writers, and editorial and production staff – whose work has contributed to the development of these documents. Finally, SAC extends special acknowledgement to Mr. Michael Mahoney, FEMA Project Officer, and Dr. Robert Hanson, FEMA Technical Advisor, for their continued support and contribution to the success of this effort.

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1. INTRODUCTION

AB XXX Note: As a resource for application of City of San Francisco Administrative Bulletin AB-XXX "Post-Earthquake Inspection, Evaluation, Repair and Retrofit Requirements for "Pre-Northridge" Welded Steel Moment Frame Buildings": FEMA 352 will form the basis for application AB XXX. Modifications to the FEMA 352 document have been developed specifically for use with this Administrative Bulletin and are provided in this Appendix for use by engineers and SFDBI in order to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

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. (AB-XXX Note: FEMA 350 has been replaced by AISC 341 for use in this AB.)
- *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. (AB-XXX Note: FEMA 351 has been replaced by ASCE 41-23 for use in this AB.)
- *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 post earthquake 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. (AB-XXX Note: FEMA 353 has been replaced by portions of AISC 360, AISC 341 and AWS D1.8 for use in this AB.)

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. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI). Locations in this Appendix to AB-XXX that pertain to post-earthquake posting will be identified.

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.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 ASCE 41-23 Seismic Evaluation and Retrofit of Existing 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.

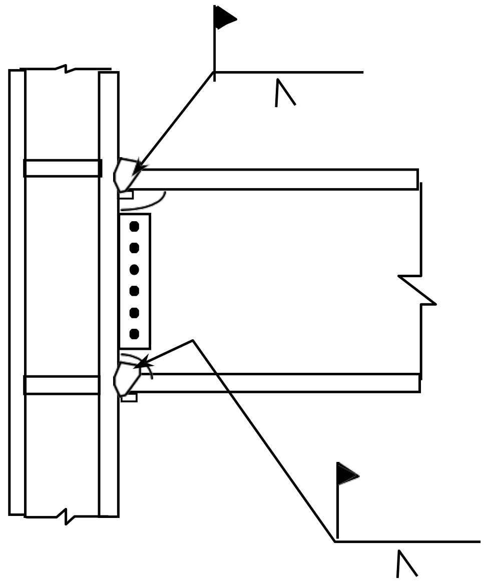


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

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.

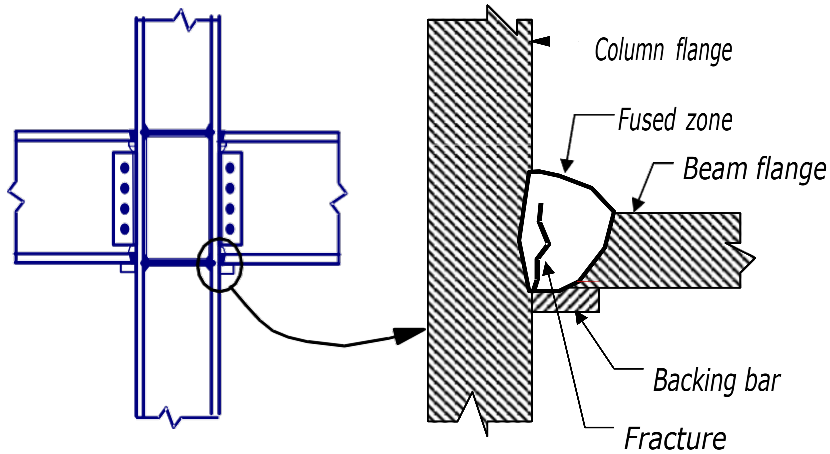
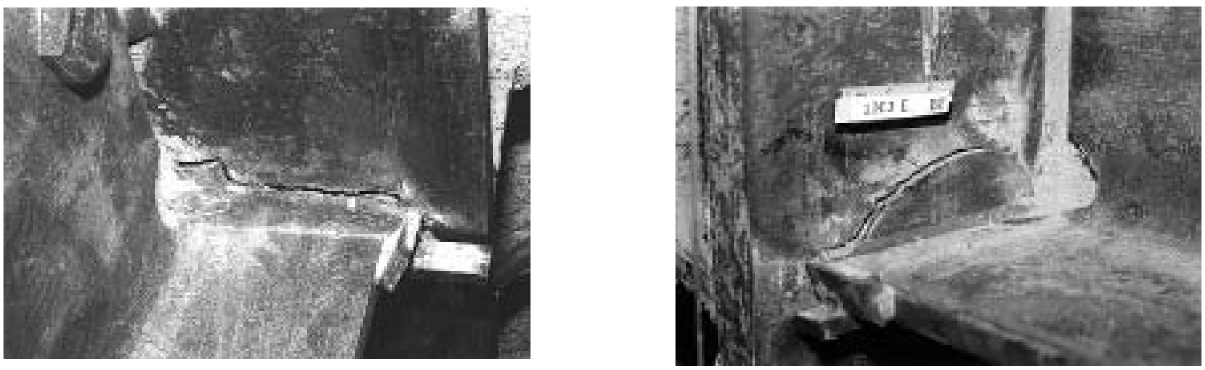


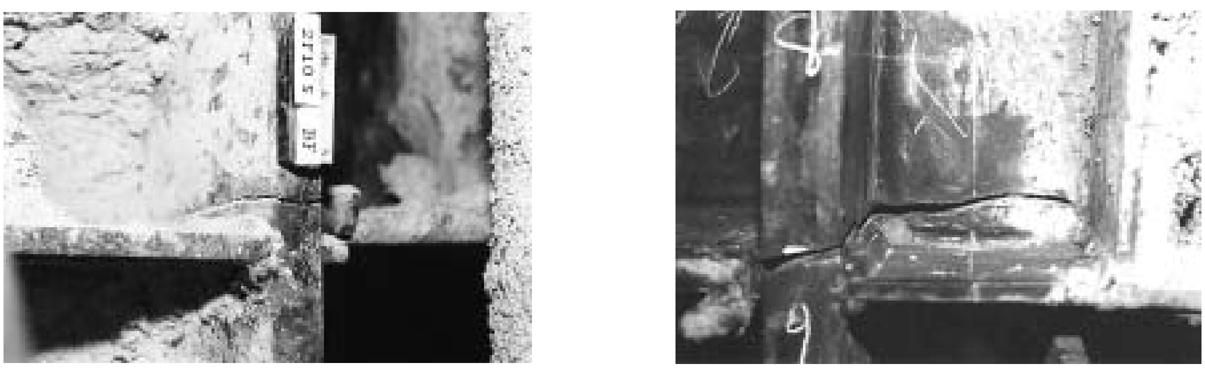
Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection



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 (2000), 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. (AB XXX Note: The documents listed above have been superseded by updated versions of ASCE 7, AISC 360, AISC 341, AWS D1.1, AWS D1.8. The latest versions of these documents should be used in conjunction with this document.

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. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 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, *ASCE 41-23 – Seismic Evaluation and Upgrade of Existing 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.

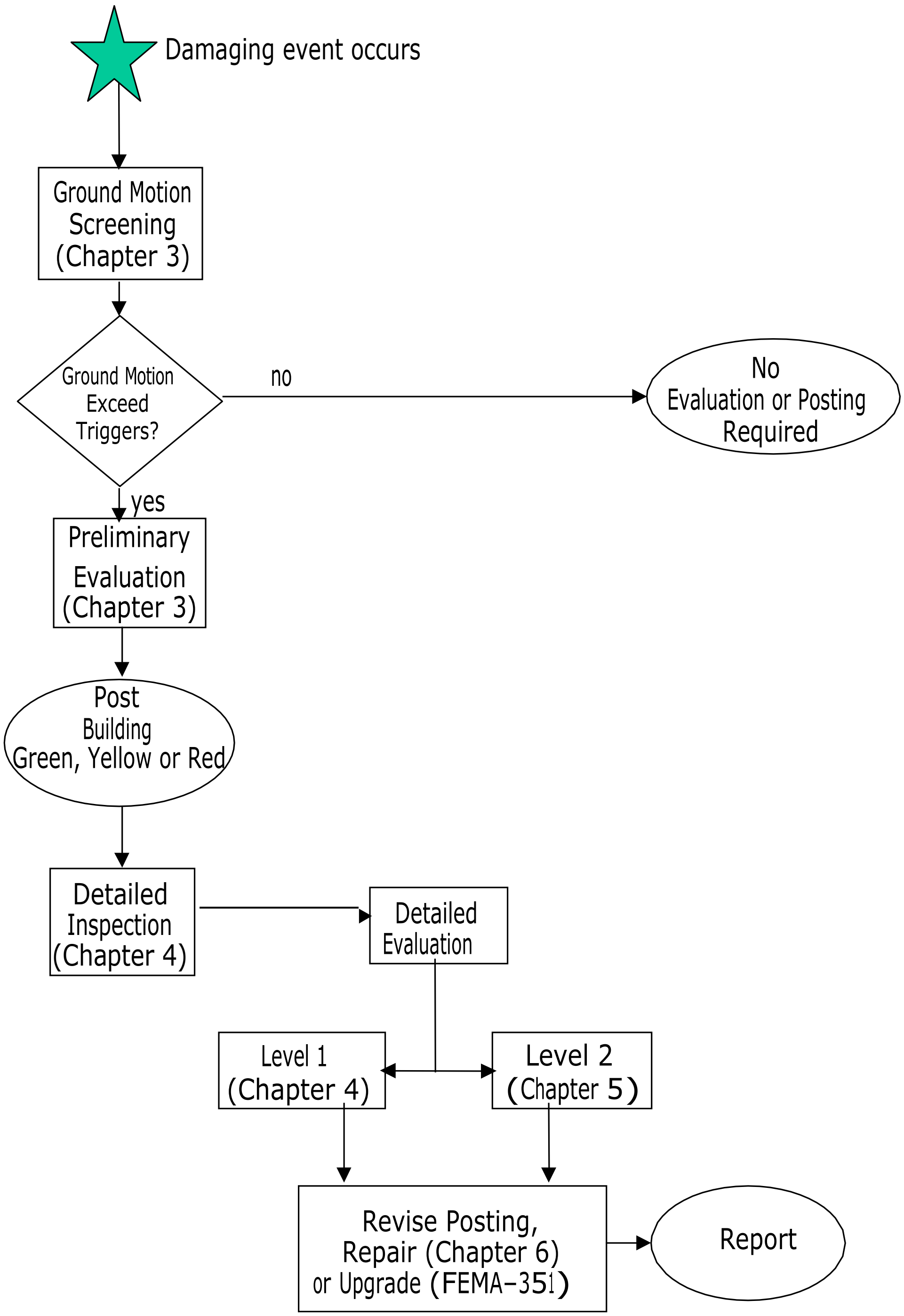


Figure 1-6 Flow Chart for Postearthquake Actions

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 based on ASCE 41-23. 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, *ASCE 41-23*.

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. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

- **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. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).
- **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. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).
- **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 *ASCE 41-23 – Seismic Evaluation and Upgrade of Existing Buildings*.

- **Appendix A: Detailed Procedures for Performance Evaluation. (Not allowed for use in AB XXX without prior approval by SFDBI and engagement with a peer review panel.)** 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. (Not allowed for use in AB XXX without prior approval by SFDBI.)** 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.**

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.

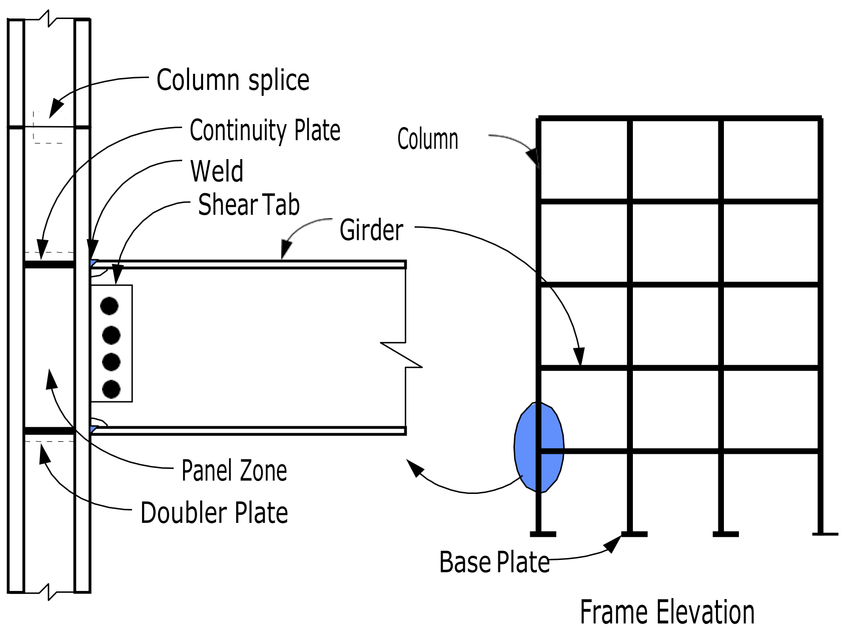


Figure 2-1 Elements of Welded Steel Moment Frame

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.

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

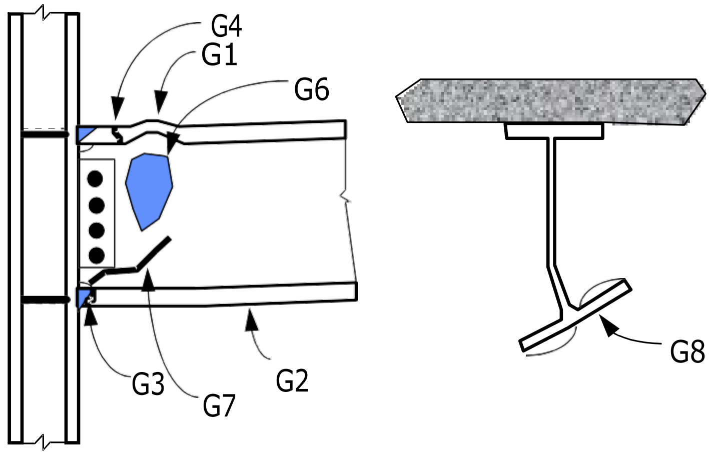


Figure 2-2 Types of Girder Damage

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

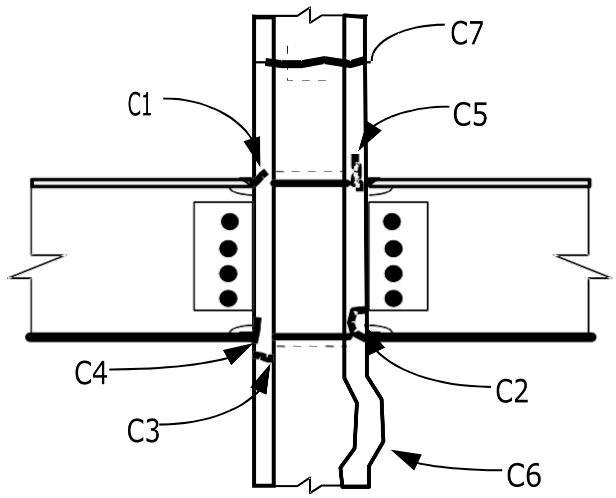


Figure 2-3 Types of Column Damage

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 *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, 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)

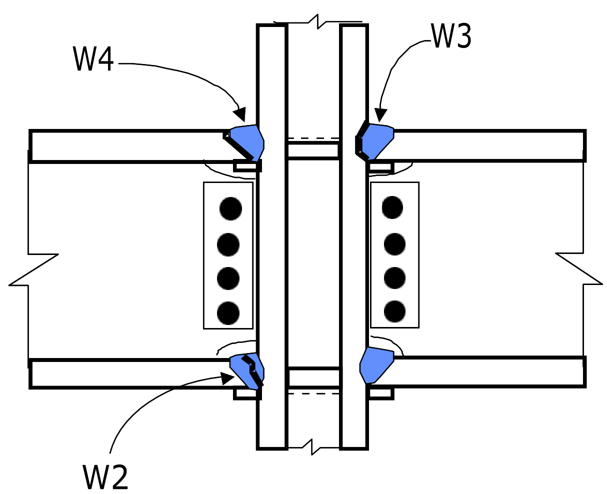


Figure 2-4 Types of Weld Damage

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 *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

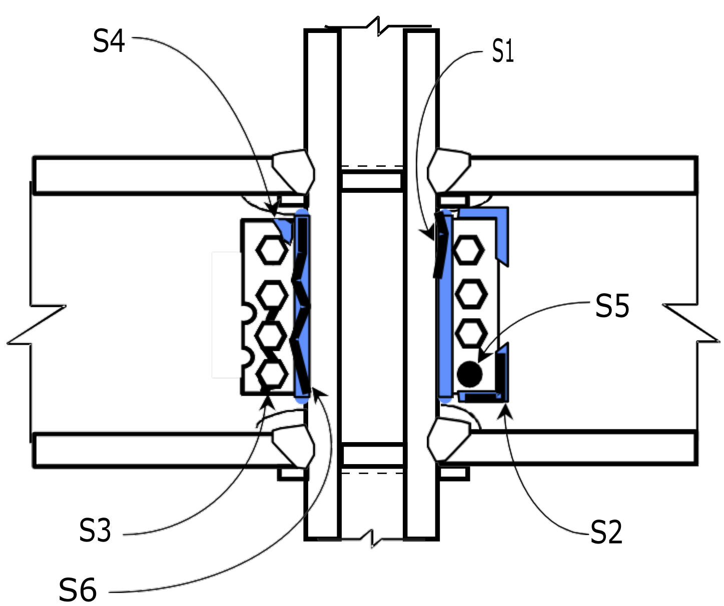


Figure 2-5 Types of Shear Tab Damage

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

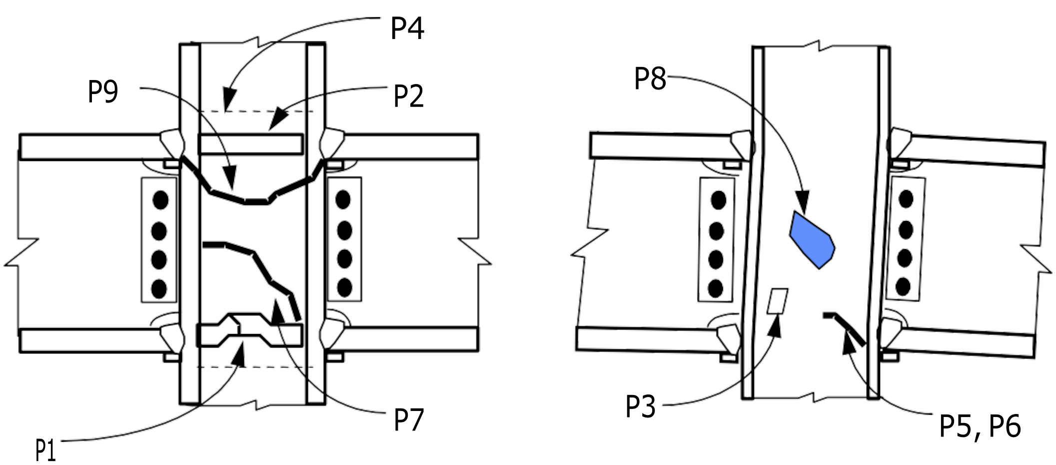


Figure 2-6 Types of Panel Zone Damage

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 POST-EARTHQUAKE 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 or retrofit. 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.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 post-earthquake 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 a 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 post-earthquake 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, ASCE 41-23 Seismic Evaluation and Retrofit of Existing Buildings for performance evaluation and upgrade recommendations for such structures. It is recommended that such performance evaluations be performed when a building has sustained significant 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

Post-earthquake 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 post-earthquake 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 if any of the following apply:

- USGS Shakemap estimate of ground motion at the building site of S1 exceeds 0.20g;
- the estimated maximum peak transient story drift ratio, δ_m, (calculated in accordance with Section 3.4) exceeds 0.007 radians;
- 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 firmer 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 firmer soil profiles;
- prevalent partial collapse of unreinforced masonry buildings, considerable damage to buildings of ordinary construction and/or high levels of nonstructural damage in buildings of any type to the general building stock within 1 kilometer of the building and on sites with similar or firmer soil profiles;
- 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.

If none of the above conditions apply to a building, it may be considered unlikely to have experienced significant damage and need not be evaluated.

Commentary: Preliminary screening is typically performed by the Building Official 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 to coordinate emergency response activities. Shakemap data provided by USGS, created by combining ground motion model estimates based on local soil conditions with spectral accelerations from recorded ground motions, generally represent the best estimate of probable intensities of ground shaking (Wald et al., 2021). The ShakeMap data may be improved in hours and days following an earthquake as additional recorded ground motion data is reviewed and incorporated into the ShakeMap. ShakeMap ground motion contours *can then be used to identify geographic areas within which evaluations should be performed or ordered by the building authority having jurisdiction. The ground motion threshold for inspection of S1 equal to 0.2g or greater is based on recent research to assess the risk of connection weld fracture in nonlinear response history analyses of archetypical models representative of tall pre-Northridge steel moment-frame buildings (Galvis et al., 2024; Valois et al. 2025).*

3.3 Preliminary Evaluation

3.3.1 General

The objective of the 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 3-1 are recommended as posting categories. Section 3.3.4 provides recommended criteria for assignment of a building to the various posting categories.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow, or green placard postings, e.g.) are not included in the scope of the AB. However, at its discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations, or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

Table 3-1 Post-earthquake Condition Designations

Condition		Finding	Description
GREEN	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.
YELLOW	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.
RED	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 post-earthquake evaluation process to discover that although a building has relatively little damage, it has significant structural deficiencies relative to current building code requirements and may, as a result, be structurally unsafe. The condition assessments indicated in Table 3-1 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 a post-

earthquake evaluation, the engineer should notify the Owner and Building Official of these conditions.

3.3.2 Building Construction Characteristics

To make a meaningful assessment of a building's post-earthquake 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 the use of particular weld metals,
- identification of materials and typical details of connections and elements for areas of 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 of concrete or masonry.

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 post-earthquake 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 story 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 story 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 story 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 story drift.

   Preliminary evaluation of the building for permanent story 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 as outlined below in Sections 4A and 4B. 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.

   (A) Where visual observation of building exterior or interior indicates the conditions in the zone or zones indicated in Table 3-2, then perform selective removal of architectural finishes to expose framing in the locations indicated. Observe for indications of yielding or buckling of framing or damage to connections. Exposures and observations should be made of at least one beam-column connection per framing line per story in the regions indicated. 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 the minimum number of exposures per story as specified in Table 3-2.

(B) 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.

Table 3-2 Post-earthquake Connection Inspection Exposures

Case	Indicator for Zone(s) of Interest	Locations for selective removal of finishes and observations	Minimum number of connection exposures per story for highly redundant structures
1	Large permanent story drift	Affected zone(s)	two exposures
2	Concentrated transient story drift	Affected zone(s)	four exposures
3	No indications of permanent story drift or concentrated transient story drift	Throughout structure	two exposures

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.

(AB-XXX Note: The disruption or removal of asbestos-containing materials is not allowed without express approval of the Authority Having Jurisdiction. Consult with SFDBI prior to any such actions.)

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 personal protection. The engineer should not personally attempt to remove fireproofing suspected of being an asbestos-containing material unless trained in the appropriate hazardous materials handling procedures and wearing appropriate protective equipment.

(AB-XXX Note: The disruption or removal of asbestos-containing materials is not allowed without express approval of the Authority Having Jurisdiction. Consult with SFDBI prior to any such actions.)

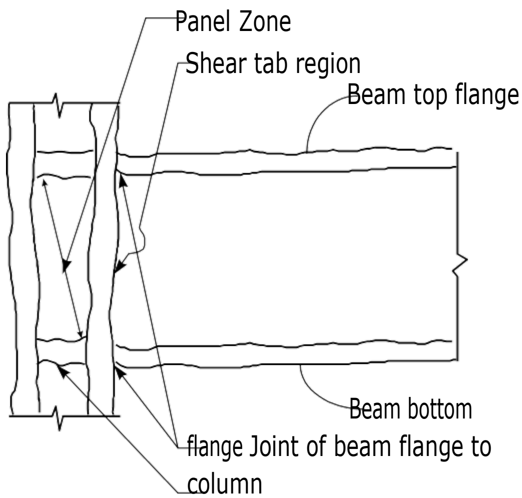


Figure 3-1 Observation Zones for Fire-Proofed Beam-Column Connections

- ❒ 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 the 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).
  - along 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).
  - 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).

- 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 top and bottom beam flanges for loosened, spalled or cracked material, indicative of fractures, buckled or yielded sections (damage types C1, C3, C4, C6, Section 2.2.6).
- in the area immediately above the top and 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).
- in the area immediately above and below the column splices for loosened, spalled or cracked material indicative of column-splice fractures (damage type C7, 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 fireproofing 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. Inspections should also look for damage or spalling to concrete above the beam, which could be indicative of column damage above the beam. However, as column damage or fractures above the beam are less likely than the other areas noted, concrete need not be removed unless significant damage to the concrete is observed.

(AB-XXX Note: The disruption or removal of asbestos containing materials is not allowed without express approval of the Authority Having Jurisdiction. Consult with SFDBI prior to any such actions.)

In many buildings constructed before 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 to both the person performing the work and others 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.

(AB-XXX Note: The disruption or removal of asbestos-containing materials is not allowed without express approval of the Authority Having Jurisdiction. Consult with SFDBI prior to any such actions.)

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.

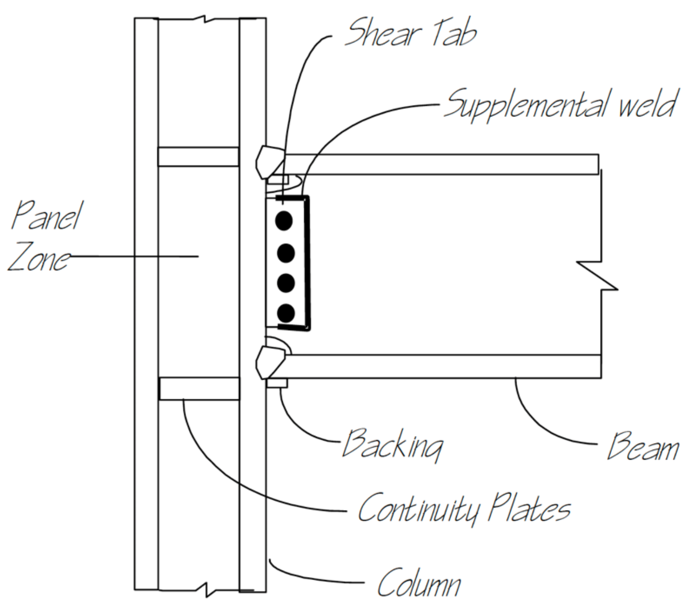


Figure 3-2 Components of Moment Connection

Note that visual inspection should not be performed casually. After a fracture occurs in steel framing, it can close under further loading of the building. Such "closed" fractures, though obscure, can typically be detected by careful observation, sometimes aided by 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 cannot normally be performed as part of a preliminary evaluation.

Column splice welds should also be inspected for evidence of C7 damage.

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 (3.3.3.3 to 3.3.3.5) provide recommendations in this regard. The structural engineer, based on the evaluated data or personal engineering judgment, may make a more conservative assessment.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 story drift at any level of 0.7% 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.

If any of the above conditions are detected, the building should be assessed on a preliminary basis as conforming to damage condition Red-1 of Table 3-1. 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.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations, or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 by processing the strong motion recording from the damaging earthquake or aftershocks. In buildings without instruments, the observation of unexpectedly 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 3-1. Appropriate precautions should be taken to limit access to hazardous areas.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

- 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 the floor framing have 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 3-1. 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.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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 the screening criteria outlined in Section 3.2, 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 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).

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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.

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

3.3.5.2 Private Consultants

If post-earthquake 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).

(AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may

choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

In addition, a formal report should be prepared indicating the scope of the evaluation and its findings, including a description of any damage encountered, the appropriate post-earthquake 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.

3.4 Procedure for Estimating Peak Transient Story Drift

This section outlines a method to estimate the probable peak transient drift story drift experienced during the earthquake, based on (1) an estimate of the building's fundamental period of vibration, or a more exact one, if this is available, and (2) an estimate of the associated spectral acceleration. As per the screening criteria in Section 3.2, if the estimated peak transient story drift ratio exceeds 0.007 radian, then further inspection is required. The steps to estimate the transient building story drift are as follows:

Step 1 - Determine Estimated Building Period, T_E

Determine the approximate building first mode period, T_E, from the following:

$$T_E = 0.035 h_n^{0.8} \tag{1}$$

where:

h_n is the height of the building above-grade (measured in ft)

Exception: Where modal analysis or strong motion instrumentation for the building indicates that another value of T_E is more appropriate, this analytically or instrumentally determined period may be used.

Commentary: For steel moment-frame structures, ASCE 7-22 specifies a formula to approximate the lower-bound period to conservatively determine the seismic response coefficient for design. As illustrated in Figure 1, Equation 1 is an adaptation of the ASCE 7-22 equation to approximate the mean (expected) period of steel frame structures. The period given by Equation 1 has also been shown to provide a reasonable approximation to first-mode periods from frame analysis models of a set steel moment-frame models representative of tall steel buildings in San Francisco (Valois et al. 2025).

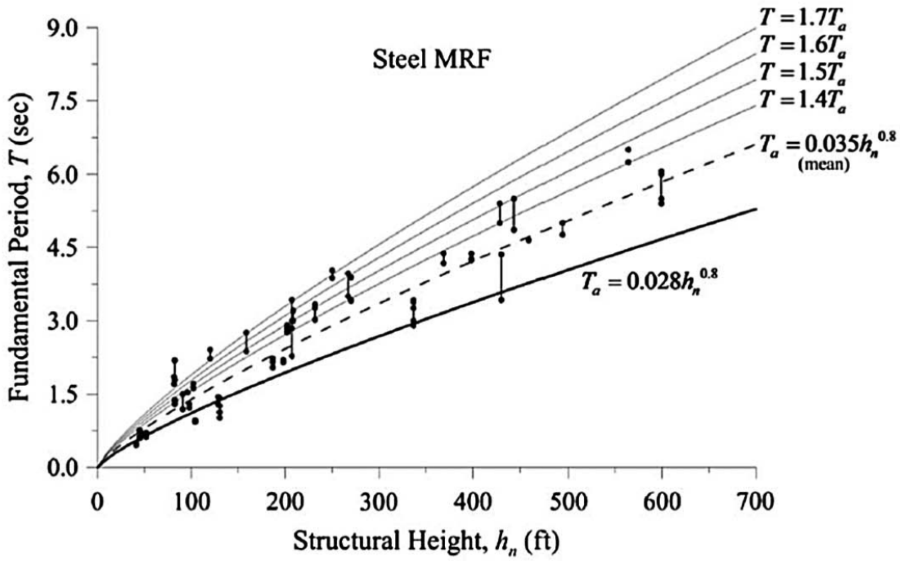


Figure 1 – Approximate Period formula and measured SMF building period from ASCE 7-22 Figure C12.8-2, based on work by Chopra, et. al.

Step 2 - Estimate the Spectral Acceleration Demand

Using the building site location, determine the estimated first mode spectral acceleration at the building site, S_{aE}, from the Shakemap data of spectral acceleration for the damaging earthquake. Where the Shakemap data are available only for short-period spectral acceleration S_s and 1-second period acceleration S_1, the acceleration demand shall be determined from equations 2 or 3, as applicable.

$$S_{aE} = S_s \qquad T_E < T_S \qquad (2)$$

$$S_{aE} = {}^{S_1}/_{T_E} \qquad T_E \geq T_S \qquad (3)$$

where:

S_S Is the value of short-period spectral acceleration obtained at the site location from the Shakemap, using linear interpolation of plotted contours

S_1 Is the value of 1-second spectral acceleration obtained at the site location from the Shakemap, using linear interpolation of plotted contours

T_S Is the transition period as determined from equation (4).

$$T_s = {}^{S_1}/_{S_s} \qquad (4)$$

Where the available Shakemap data are available at sufficient building response periods to define a smoothed response spectrum at the site in the period range of interest for the building, or strong motion recordings are obtained from the building that enable construction of a site-specific, 5% damped, smoothed response spectrum the values of S_{aE} may be obtained directly from this smoothed response spectrum at period T_E.

Step 3 – Estimate the Peak Transient Building Drift

Estimate the peak transient building roof displacement, Δ_R, from Equation 5.

$$\Delta_R = C_0 \frac{{T_E}^2}{4\pi^2} S_{aE} \quad (5)$$

where:

C_0 Is the modal shape factor obtained from ASCE 41-23, Table 7-5, for the number of stories, "other building types" and "any load pattern."

Step 4 – Estimate the maximum peak transient story drift

Estimate the maximum peak transient story drift ratio, δ_m, from Equation 6 or 7, as applicable, for building frames with regular geometries. These simplified equations should not be used for building frames with setbacks, soft stories, or other irregularities.

$$\delta_m = \frac{\Delta_R}{h_n} \qquad \text{For regular buildings of 3 stories or less in height} \quad (6)$$

$$\delta_m = (1 + \left(\frac{N-3}{12}\right))\frac{\Delta_R}{h_n} < \frac{2.0\Delta_R}{h_n} \qquad \text{For regular buildings with greater than 3 stories} \quad (7)$$

Commentary: Equations 6 and 7 estimate peak transient story drift ratio as a function of the roof drift ratio, calculated as the peak roof displacement divided by the building height. The peak roof drif ratio is based on the target drift equation in ASCE 41-23, and the conversion of roof to story drift ratio is based on a study of steel moment frames by Medina and Krawinkler (2004). The equations apply to buildings responding at near elastic response, since they are applied to evaluate the inspection threshold for moderate story drift ratios. The equations to estimated story drift ratios have also been substantiated in a study of steel moment frames by Valois, et al. (2025). These factors apply to buildings without setbacks, soft stories, or other irregularities that would concentrate story drifts. For buildings with irregularities, the story drifts should be calculated by frame analyses that represent the building geometry.

Step 5- Determine If Further Evaluation is Recommended

As specified in Section 3.2, if the estimated maximum peak transient story drift ratio, δ_m, exceeds 0.007 radians, building-specific evaluation is recommended.

Commentary: Analyses of buildings that were damaged in the Northridge earthquake indicate that fractures of welded connections occurred at story drift ratios as low as 0.006 radians (SAC 1995). A more recent study of archetype models of tall pre-Northridge frames indicated onset of significant connection fractures at story drift ratios of 0.005 to 0.010 (Valois et al. 2025). The threshold drift ratio of 0.007 is considered a reasonable value to screen for potential connection fractures.

NEW REFERENCES FOR CHAPTER 3 FOR AB XXX

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4. LEVEL 1 DETAILED POST-EARTHQUAKE EVALUATIONS

4.1 Introduction

Detailed evaluation is the second step of the post-earthquake 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. Prior to performing a Level 2 evaluation, it is recommended that a complete inspection of all fracture-susceptible connections be performed, to determine their condition. However, Chapter 5 also includes an alternative procedure for use when only a sample of connections have been inspected. AB-XXX note: A procedure for performing a Chapter 5 level analysis with less than a complete inspection has been added to that chapter.

Commentary: The Level 1 evaluation approach in this chapter is based upon a methodology originally presented in FEMA-267, modified to account for experience gained in applying the FEMA-267 guidelines to real buildings and 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 the 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 post-earthquake 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 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 significant 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

Post-earthquake 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. All column splices that 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 column in tension shall also be considered as "connections" in establishing the inspection plan.

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. For column splice connections remove any fireproofing or other obscuring finished to allow direct visual observation of the splice area. Refer to Chapter 3 for cautions regarding the removal of asbestos containing fireproofing materials.

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, then 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.

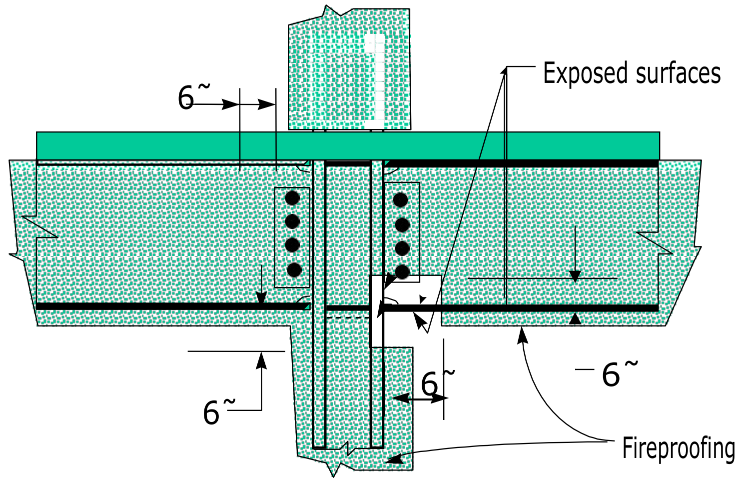

Figure 4-1 Fireproofing Removal for Initial Connection Inspection

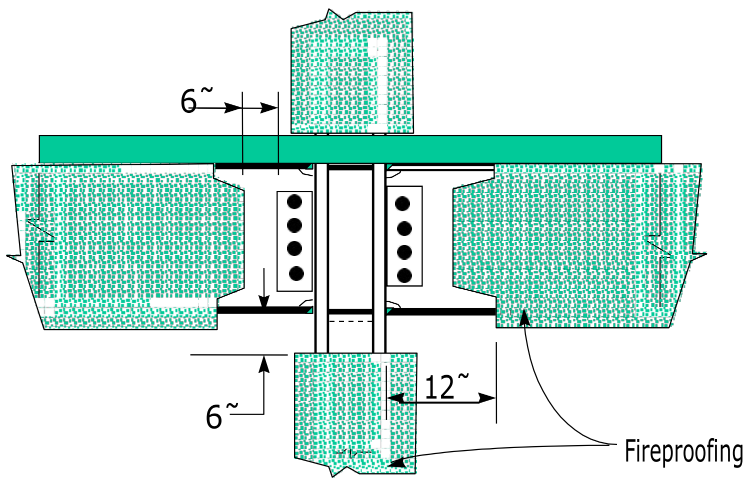

Figure 4-2 Fireproofing Removal for Complete Connection Inspection

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-torsional buckling (damage type G8, Section 2.2.1).
- ❐ Observe the 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).
- ❐ Observe the condition of the fireproofing in the region of the column splice for loosened, cracked or spalled material indicative of potential damage to the splice.

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 the 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 surfaces 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).
- ❒ Observe the column splice area for damage (damage type C7, Section S.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 combinations of two or more individual classes of damage.

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 a very severe impact. In particular, complete fractures of column flanges at beam-column connections or column splices are of particular concern as they impact both the lateral and gravity force-resisting systems.

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-P-delta instability and collapse. However, if several connections develop fractures at both flanges of the beam-column connection, a 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 that 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. Two additional groups, one for each direction, should be established at floors that have column splices in the story above. Column splices and beam-column connections should not be combined into a group, i.e., a floor with both column splices and beam-column connections should have four groups.

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

$$D_i = \frac{1}{n}\sum_{j=1}^{n}\frac{d_j}{4} \tag{4-1}$$

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 j^{th} connection or column splice 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 Section 4.4.1.3.

4.4.1.5 Determine Damage Classifications and Recommended Recovery Strategies for the Building

Recommended post-earthquake repair, damage classifications, and recovery strategies are indicated in Table 4-2. They are based on the maximum damage index, D_{max}, determined in accordance with Section 4.4.1.4. The three damage classifications are based on criteria and definitions in FEMA P-2335.

Table 4-2 Damage Classification and Recommended Repair

Values of D_{max} [3] and residual drift	**Damage Classification and Recommended Repair**	**Note**
$D_{max} > 0$	**Performance Critical Damage:** Damage Class (DC 2) in FEMA P-2335 (2025). Repair all connections discovered to have $d_j > 1$	1
$D_{max} \geq 0.2$, or permanent residual story drift > 0.006 radians.	**Disproportionate Earthquake Damage:** As defined in FEMA P-2335 (2025) for buildings in Seismic Design Categories D, E, or F that have been damaged by ground motions whose 0.3-second spectral acceleration at the building site is less than 30% of the mapped acceleration parameter S_S. Inspect all connections in the building. Repair all connections discovered to have $d_j > 1$ and evaluate need for structural upgrade.	1
$D_{max} > 0.5$, or permanent residual story drift > 0.006 radians.	**Substantial Structural Damage:** As defined in FEMA P-2335 (2025). A potentially unsafe condition should be deemed to exist (1) unless a Level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure, or (2) until repairs have been completed. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections discovered to have $d_j > 1$ and evaluate need for structural upgrade.	1,2

Notes to Table 4-2:

1. In addition to the D_{max} criterion, Performance Critical Damage, Disproportionate Earthquake Damage, or Substantial Structural Damage conditions may occur due to damage to the gravity essential components as described in Section 4.4.1.6.
2. 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 $d_j \geq 2$.
3. See Section 4.4.1.4

*Commentary: The three damage classifications are based on criteria and definitions in FEMA P-*2335 (2025*), which are aligned with policies in the International Existing Building Code (IEBC, 2024). Performance Critical Damage pertains to any damage that may impact the structural collapse safety of a building. In steel moment-frame systems, essentially any significant member buckling, steel fractures, or bolt failures are considered Performance Critical Damage and should be repaired or otherwise mitigated (e.g., by retrofitting the moment frame with an alternate lateral force resisting system, such as the addition of steel braces or a concrete shear wall). Substantial Structural Damage is defined as damage that has either reduced the lateral load-carrying capacity by more than 33% or significantly reduced the gravity load-carrying system. Disproportionate Earthquake Damage is defined as damage that has either reduced the lateral load-carrying capacity by more than 10% or significantly reduced the gravity load-carrying system. Disproportionate Earthquake Damage or Substantial Structural Damage classifications are also triggered if the permanent residual story drift ratio in any story is larger than 0.006 radians (0.6% story drift). This threshold is three times the maximum permissible story out-of-plumb for frame erection and 0.6 times the limit permitted under MCE-level ground motion demand for performance-based seismic design of new code-conforming tall buildings (PEER TBI 2017, LATBSDC 2024). The criteria for loss in lateral load-carrying capacity based on* D_{max} *and residual drift are based on a combination of judgment and nonlinear dynamic analyses of archetypical pre-Northridge steel moment-resisting frames by Valois et al. (2025).*

Per FEMA P-2335 and the IEBC, classifications of Disproportionate Earthquake Damage or Substantial Structural Damage require additional assessment to determine whether the building may require structural subgrade (i.e., retrofit) in addition to repair. This is particularly the case when damage is severe 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

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. As indicated in Table 4-2, building damage classified as Substantial Structural Damage is potentially unsafe for re-occupancy following an earthquake until they have either been repaired or judged to be safe based on a more detailed evaluation.

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.

ASCE 41 and AISC 342 provide procedures for assessing the probable performance of steel moment-frame buildings and for designing upgrades to improve this performance.

4.4.1.6 Inspect and Characterize Damage to Gravity Essential Components

The building's Gravity Essential Components should also be inspected and checked for post-earthquake repair, damage classification, and recovery strategies. The inspection, classification, and recovery strategy for the steel moment frame's gravity essential components are considered to be covered by requirements in Sections 4.4.1.1 to 4.4.1.5.

Components of the gravity system framing, including columns, beams, and connections, should be included in the detailed inspection and assessment if either (1) damage to the gravity system framing is suspected based on preliminary post-earthquake assessments, or (2) if the building is classified with either Disproportionate Earthquake Damage or Substantial Structural Damage according to the requirements of Sections 4.4.1.1 to 4.4.1.5. In such cases, the components of the gravity system framing should be inspected at the beam-column connections and classified for the following damage indices in Table 4-1a: G1, G2, G6, G7, G8; C1 to C7, and S1 to S6.

If damage to gravity system framing is observed, the gravity framing damage index for each floor should be calculated using Equation 4-1, where all gravity framing components are included to calculate one index per floor. The damage should then be classified in accordance with Sections 4.4.1.4 and 4.4.1.5.

Commentary: In general, gravity system framing is not expected to be especially vulnerable to earthquake damage in pre-Northridge moment frame buildings. Nearly all the damage to steel moment frame buildings in the Northridge earthquake occurred to the welded beam-column connections, which occurred at induced drift levels that were well below the drifts that are expected to cause damage to gravity framing systems. For example, tests of well-detailed shear-tab gravity framing connections showed that while damage may initiate (typically local crushing of the floor slab against the column) at story drift ratios of about 0.04 radians, the connections could sustain story drift ratios up to about 0.10 radians (10% story drift) without loss of their gravity resising capacity (Liu and Astaneh-Asl, 2000). The required inspection and damage indices for the gravity framing are primarily intended to capture damage to non-typical shear connections at moderate drifts or where the building has experienced large story drifts (e.g., more than 0.04 radians).

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 classify damage and 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. Two additional groups, one for each building direction, should be established for floors that have column splices in the story above the floor level.

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

Step 2: Determine the minimum number of connections and column splices 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 and column splices using the procedures of Section 4.4.1 and determine connection damage indices, d_j, for each inspected connection.

Step 4: If inspected connections or splices 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 or splices 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 or splices 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 its as-damaged state to improve understanding of its residual condition, confirm that the recommended strategies are appropriate, or suggest alternative strategies.

However, for such analyses to be meaningful, full inspections of all connections and splices are reccomended. 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 and column splices 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. Two additional groups, one for each building direction, should be established for floors that have column splices in the story above.

In buildings with 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 for 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. Method B is required for buildings over 240 feet in height. 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 that provide a favorable economic result of building disposition. The building official need not perform any action regarding 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. If 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 or splices 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 or splices shown by analysis to have the highest demand/capacity ratios or at locations experiencing the largest drift ratios.
    - Connections or splices 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 or splices incorporating the largest size framing elements.

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

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 the 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 are often practical incentives 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 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. As discussed above, not more than 10% of the total connections inspected should be selected based on convenience.

There are several 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 spreadsheet 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 (Required for buildings over 240 feet in height)

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

1. The minimum number of connections or splices 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 or splices 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 or splices may be selected based on structural analysis results as those most likely to be damaged.
3. The remaining connections or splices in the group to be inspected are selected so that the sample contains connections or splices distributed throughout the building, including the upper, middle, and lower stories. The rules of Section 4.4.2.2.1 should be followed in a general way.

Prior to initiating the inspections, the structural analysis and list of connections or splices to be inspected should be subjected to a qualified independent third-party peer review. The peer review should consider the basis for the analysis and the consistency of the assumptions employed. The review should assure that overall, the resulting list of connections or splices to be inspected provides an appropriate sampling of the building's connections or splices. For this purpose, an elastic modal or response history analysis of the structure should be considered acceptable. The ground motion input for the model should be based on USGS Shakemap data for the damaging earthquake or strong motion instrument recordings made in or immediately adjacent to the building.

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

Commentary: In analyses conducted of damaged buildings following the Northridge earthquake, there has not always been strong correlation between locations of connection damage and the highest demands 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. Nevertheless, structural 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 reflect the spectral characteristics of the damaging ground motion experienced at the site. USGS Shakemaps are created using a combination of ground motions estimated based on the earthquake rupture, ground motion models that account for site characteristics, and recorded seismogram. Short of performing a more detailed assessment of ground motions, the Shakemaps are considered to provide a best estimate of the site-specific spectral characteristics experienced at the site (Wald et al., 2021). Current Shakemaps generally report PGA, Ss, and S1 values of the geomean values of acceleration; however, USGS has plans to enhance Shakemaps to provide 20-period spectral quantities. The spectral values can also be adjusted to account for directivity bias in nearby recorded ground motions (Bantis et al. 2024). 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. A recent study by nonlinear dynamic analyses of archetypical pre-Northridge steel moment-resisting frames by Valois et al. (2025) has indicated that tall building response *characteristics (higher mode effects, e.g.) are substantial enough that fully random selection of connection inspections is not recommended for tall buildings. For the purposed of this document, "tall" should be considered as at least 240 feet in height.*

4.4.2.3 Step 3 — Inspect the Selected Samples of Connections

4.4.2.3.1 Inspection

The moment-resisting connections and column splices 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 or splice as described in Section 4.4.1.1 An individual data sheet (Appendix C) should be filled out for each connection or splice 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 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 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, 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 several 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 Recommended Criteria have de-emphasized, relative to the recommendations of FEMA-267, the importance of employing NDT in the post-earthquake inspection process.

4.4.2.3.2 Damage Characterization

The observed damage at each of the inspected connections or splices 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 $1 < d_j < 2$, 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. When a column splice is found to be damaged (Damage Type C7), check adjacent column splices consistent with Figure 4-4.

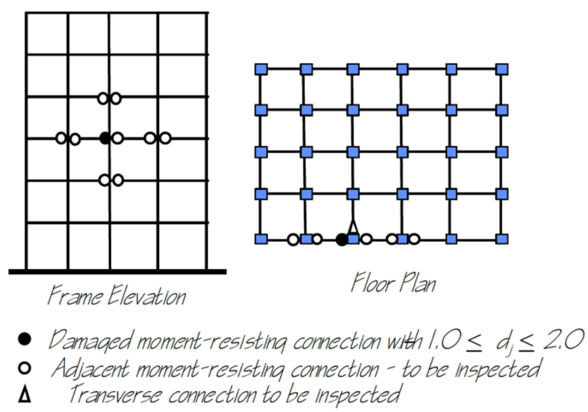

Figure 4-3 Inspection of Connections Adjacent to Damaged Connection ($1 \leq d_j \leq 2$)

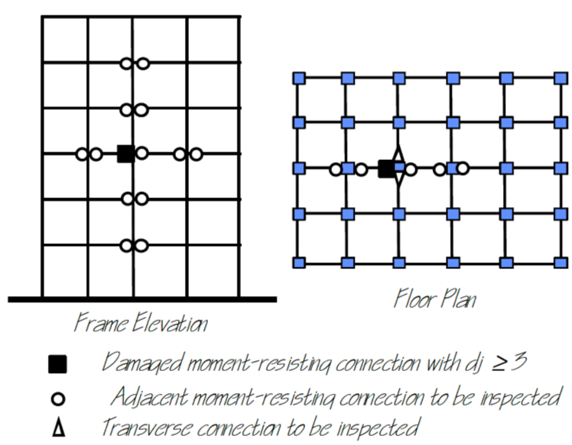

Figure 4-4 Inspection of Connections Adjacent to Damaged Connection ($d_j \geq 3$)

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

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

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

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 j^{th} inspected connection in the original sample

The additional connections or splices 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 Substantial Structural Damage

In this procedure, the probable maximum floor damage index at a floor D_{max} is estimated from the damage indices determined for all the connections actually inspected, including those additional connections inspected in accordance with the requirements of Section 4.4.2.4. In addition, the probability $P_{0.5}$ that 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, and

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

Second, determine the probability $P_{0.5}$ 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 $b_{0.5}$, which represents the number of multiples of the standard deviation of a normal distribution above the mean that would be required to exceed 0.5. The factor $b_{0.5}$ is calculated from the equation:

$$b_{0.5} = (0.5 - D)/S \quad (4\text{-}6)$$

Using the value of $b_{0.5}$ calculated from equation 4-6, determine $P_{f0.5}$, from Table 4-4. $P_{f0.5}$ 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 $P_{f0.5}$ is high, this strongly suggests the possibility that there has been a significant reduction in seismic resisting capacity.

Next, determine the probability $P_{0.5}$ 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:

$$P_{0.5} = 1 - \left(1 - P_{f0.5}\right)^q \qquad (4\text{-}7)$$

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:

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

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 $P_{f0.5}$ or $P_{f0.2}$ as a Function of Parameter $b_{0.5}$ or $b_{0.2}$

$b_{0.5}$ or $b_{0.2}$	$P_{f0.5}$ or $P_{f0.2}$	$b_{0.5}$ or $b_{0.2}$	$P_{f0.5}$ or $P_{f0.2}$
-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 0.5, in equation form, this is expressed as :

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

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

In spite of the approximations associated with setting the 0.50 damage index criterion (Valois et al., 2025) and the suggested way of testing whether that criterion 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. In anomalous or marginal cases, such additional checks based on engineering judgment are strongly encouraged.*

4.4.2.7 Step 7 – Determine the Probability that the Connections in a Group at a Floor Level Sustained Disproportionate Earthquake Damage

If the estimated ground motion at the site has a value of the 0.3 second spectral acceleration that is less than 30% of the mapped acceleration parameter Ss, using the value of $b_{0.2}$ calculated from equation 4-10, determine $P_{f0.2}$, from Table 4-4. $P_{f0.2}$ is the probability that if all connections had been inspected, the cumulative damage index at any floor would have been found to exceed 0.20. If the probability $P_{f0.2}$ is high, this strongly suggests the possibility that there has been a disproportionate reduction in seismic resisting capacity.

$$b_{0.2} = (0.2 - D)/S \qquad (4\text{-}10)$$

Next, determine the probability $P_{0.2}$ 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 0.02 or more from the equation:

$$P_{0.2} = 1 - (1 - P_{f0.2})^q \qquad (4\text{-}11)$$

4.4.2.8 Step 7-- Determine Damage Classifications and Recommended Recovery Strategies for the Building

Recommended post-earthquake repair, damage classifications, and recovery strategies are indicated in Table 4-5. They are based on the maximum damage index, D_{max}, determined in the previous steps. The three damage classifications are based on criteria and definitions in FEMA P-2335.

Table 4-5 Recommended Repair and Damage Classifications

Values of D_{max} and P	Recommended Repair and Damage Classifications	Note
$D_{max} > 0$	**Performance Critical Damage:** Damage Class (DC 2) in FEMA P-2335 (2025). Repair all connections discovered to have $d_j \geq 1$	1,2
$P_{0.2}$ > 50 %, or $D_{max} > 0.2$, or permanent residual story drift > 0.006 radians.	**Disproportionate Earthquake Damage:** As defined in FEMA P-2335 (2025) for buildings in Seismic Design Categories D, E, or F that have been damaged by ground motions whose 0.3-second spectral acceleration at the building site is less than 30% of the mapped acceleration parameter S_S. Inspect all connections in the building. Repair all connections	1

	with $d_j \geq 1$ and evaluate need for structural upgrade.	
$P_{0.5}$ > 25 %, or D_{max} > 0.5, or permanent residual story drift > 0.006 radians.	**Substantial Structural Damage:** As defined in FEMA P-2335 (2025). A potentially unsafe condition should be deemed to exist (1) unless a Level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure, or (2) until repairs have been completed. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j \geq 1$ and evaluate need for 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.

4.4.2.9 Inspect and Characterize Damage to Gravity Essential Components

The building's Gravity Essential Components should also be inspected and checked for post-earthquake repair, damage classification, and recovery strategies. The inspection, classification, and recovery strategy for the steel moment frame's gravity essential components are considered to be covered by requirements in Sections 4.4.2.1 to 4.4.2.7.

Components of the gravity system framing, including columns, beams, and connections, should be included in the detailed inspection and assessment if either (1) damage to the gravity system framing is suspected based on preliminary post-earthquake assessments, or (2) if the building is classified with either Disproportionate Earthquake Damage or Substantial Structural Damage according to the requirements of Sections 4.4.2.1 to 4.4.2.7. In such cases, the components of the gravity system framing should be inspected at the beam-column connections and classified for the following damage indices in Table 4-1a: G1, G2, G6, G7, G8; C1 to C7, and S1 to S6. The sampling to inspect components of the gravity system should follow the sampling procedure covered by the by requirements in Sections 4.4.2.1 to 4.4.2.7.

If damage to gravity system framing is observed, the gravity framing damage index for each floor should be calculated using Equation 4-8, where all sampled gravity framing components are included to calculate one index per floor. The damage should then be classified in accordance with Sections 4.4.2.3 and 4.4.2.4.

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.

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5. LEVEL 2 DETAILED POSTEARTHQUAKE EVALUATIONS

5.1 Introduction

Detailed evaluation is the second step of the post-earthquake 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 decrease, if any, of the 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, and those for any alterations or repairs, 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. , *ASCE 41-23 – Seismic Evaluation and Retrofit of Existing Buildings* and *AISC 342-22 Seismic Provisions for Evaluation and Retrofit of Existing Steel 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 permit 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 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 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. Following publication and adoption of the International Building Code, these criteria became nationally applicable in structures assigned to Seimsic Design Categories D, E and F. However, fracture-susceptible connections may exist in some post-1994 buildings, particularly those constructed in zones of lower seismicity.

5.2.1 Procedure for Developing Damaged Model from Limited Data

The procedures of this section shall be used to develop models of the damaged structure when only a portion of the building's vulnerable connections have been inspected for damage.

Follow the procedures of FEMA 352 Section 4.4.2 and determine the quantities, D_i for each floor with inspected connections for each connection group and d_{avg}:

1. For each connection group, and each floor in the group at which inspection has been performed, determine the floor damage index, D_i per formula 4-8.

2. Determine the average damage index d_{avg} for each group per formula 4-2.

3. At each floor level, in a direction of response, assign damage indices, d_{kl}, to each uninspected connection, such that the computed damage index D'_i, for the floor determined in accordance with formula 4-1, as if all connections at the floor had been inspected, equals or exceeds D_i for the floor. For floors at which no connections have been inspected, take $D_i = d_{avg}/4$. Note that it is permissible to assign a value of d_{kl}=0 to some connections.

4. Construct a "damaged" model by modifying the model of the undamaged structure per the criteria of Section 5.9.11, for each connection with an assigned damage index d_i greater than 0.

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 post-earthquake disposition for the building. (AB XXX note: Post-earthquake occupancy status decisions (red, yellow or green placard postings, e.g.) are not included in the scope of the AB. However, at its' discretion, SFDBI may choose to allow engineers to make use of this material in assigning post-earthquake occupancy status, condition designations or posting categories. As a result, language related to occupancy decisions was not removed from the document to help facilitate such decisions by SFDBI).

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.

The basic tool used to implement a level 2 evaluation is a structural analysis of the building in 1) the undamaged state and 2) the damaged state.

Commentary: ASCE 41-23 – Seismic Evaluation and Retrofit of Existing Buildings and *AISC 342-22 Seismic Provisions for Evaluation and Retrofit of Existing Steel Buildings will be used in conjunction with the provisions of this Chapter to establish whether the threshold has been exceeded for either Substantial Structural Damage or Disproportionate Earthquake Damage, as defined in the IEBC and FEMA 2335 – Guidelines for Post-Earthquake Assessment, Repair, and Retrofit of Buildings.*

5.4 Field Inspection

Prior to performing 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 potential damage that cannot be quantified by visual means alone. Beam- column connections and column splices 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 *ASCE 41*and should be considered as part of a post-earthquake 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.

Determine the required material properties, including base, bolt, and weld metal strength and toughness in accordance with AISC 342, Section A5.

5.6 Structural Analysis

The basic process of post-earthquake evaluation, as contained in these procedures, is to develop a mathematical model of both the damaged and undamaged structure, and by performing structural analysis, to determine the reduction, if any, in the value of the spectral acceleration at the building's first mode fundamental period, $S_{a\text{-}CP}(\underline{T_{1U}})$ at which the structure provides Collapse Prevention performance, per ASCE-41.

Notation

T_{1U} – First mode translational period of the structure in the undamaged condition, sec.

T_{1D}- First mode translational period of the structure in the damaged condition, sec.

$S_D(T_{1U})$- Design spectral acceleration at the undamaged building's first mode period, sec.

$S_D(T_{1D})$- Design spectral acceleration at the damaged building's first mode period, sec.

$S_{au\text{-}CP}(T_{1U})$- Spectral acceleration at the first mode undamaged period of the building, T_{1U}, at which the structure just meets Collapse Prevention performance, as defined in *ASCE 41, g*.

$S_{au\text{-}CP}(T_{1D})$- Spectral acceleration at the first mode period of the damaged building, T_{1D}, at which the undamaged structure just meets Collapse Prevention performance, as defined in *ASCE 41, g*.

$S_{ad\text{-CP}}(T_{1D})$- Spectral acceleration at the first mode period of the damaged building, T_{1D}, at which the damaged structure just meets Collapse Prevention performance, as defined in *ASCE 41, g*.

$S_{ad\text{-CP}}(T_{1U})$- Spectral acceleration at the first mode undamaged period of the building, T_{1U}, at which the damaged structure just meets Collapse Prevention performance, as defined in *ASCE 41, g*.

S_{1E} – Estimated spectral acceleration at a period of 1-second, at the building site during the damaging earthquake, g.

> *Commentary: The values of $S_{au}(T_{1U})$ and $S_{ad}(T_{1D})$ are determined directly from analysis, in accordance with ASCE 41. These values are respectively determined as the largest value of the first mode acceleration at the structure's fundamental period, that the structure can resist and still meet the Collapse Prevention performance level. The values of $S_{au}(T_{1D})$ and $S_{ad}(T_{1U})$ are derived from the design response spectrum for the building, as determined under the building code.*

5.7 Ground Motions Spectra

5.7.1 Design Earthquake Spectrum

To determine the spectral acceleration parameters $S_{au\text{-CP}}(T_{1D})$ and $S_{ad\text{-CP}}(T_{1U})$ for the building, it is necessary to obtain the 5%-damped, design earthquake acceleration response spectrum for the building under the current building code. This may be obtained using the ASCE 7 hazards tool and either the multi-period or 2-parameter spectrum of ASCE 7. Alternatively, a site-specific design earthquake spectrum may be used. Regardless of the procedure used, the same design spectrum shall be used for both the damaged and undamaged structures. The values of the two parameters, $S_{au\text{-CP}}(T_{1D})$ and $S_{ad\text{-CP}}(T_{1U})$, shall be determined as the spectral accelerations from the design spectrum at periods T_{1D} and T_{1U}, respectively.

5.7.2 Damaging Earthquake

The value of the 1-second, 5%-damped spectral acceleration at the building site, S_{1E}, during the damaging earthquake is used to determine if Disproportionate Earthquake Damage has occurred. S_{1E} shall be determined from one of the following sources:

1. Actual accelerogram obtained from a strong motion recording instrument located at or in the building either at or below ground level
2. Actual accelerogram obtained from a strong motion recording instrument located on a site within 1 kilometer of the building, on a site with similar site conditions as those present at the building, and located at or below ground level
3. A value determined for the USGS Shakemap tool

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 ASCE 7-22.

When instrumental recordings of the damaging ground shaking are not available, an estimated value of S_{IE} for this ground shaking should be constructed based on ShakeMap information published by the USGS for the building site in the damaging causing event.

5.7.3 Acceleration Time Histories

Acceleration time histories, if required, should be constructed in accordance with the recommendations of ASCE 41, for the BSE-1N hazard level. These histories shall be amplitude-scaled as necessary to determine the values of the parameters $S_{au\text{-CP}}(T_{1U})$ and $S_{ad\text{-CP}}(T_{1D})$.

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 both the undamaged and damaged structures, representing their strength and deformation characteristics and to conduct an analysis to predict the values of ground motion intensity at which the structures marginally meet the Collapse Prevention performance goal, as defined in ASCE 41. 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 *ASCE 41*. This section provides supplementary guidelines on the applicability of the *ASCE 41* 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.

5.8.1 Procedure Selection

Section 7.3 of *ASCE 41* 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 *ASCE 41*, 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. The same analysis procedure shall be used for the building in both the undamaged and damaged states.

Commentary: The four analysis procedures contained in ASCE 41 each have limitations as to their ability to reasonably predict a building's response, and also differing levels of conservatism, depending on the perceived accuracy of the approach. Use of different procedures can result in differing assessments as to the performance level a structure will achieve, when subjected to a given ground motion. Since the primary purpose of analytical evaluation in these Guidelines is to indicate the percent that building damage has degraded the building's earthquake performance capability, it is important to use the same analytical procedure to evaluate both the damaged and damaged building, so that procedural differences do not affect the assessed impact of the damage.

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 should generally be conducted in accordance with *ASCE 41*, except as specifically noted herein.

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.

Commentary: Some earthquake damage may affect the building's behavior differently in one direction of response than it does in the reverse direction. For example, a moment-resisting beam-column connection with a bottom flange fracture, will significantly degrade the stiffness of the connection under loading that tends to place the bottom flange in tension at the connection, but will not have significant impact on stiffness in the reverse loading. Similarly, column splice failures will eliminate a columns ability to resist tensile loads, and under low levels of compressive load, will limit the columns flexural capacity at the splice, but may not affect the column's compressive capacity. In a damaged structure, it may be difficult to demonstrate the direction of response in which damage has the most affect, without performing analysis in both directions. Therefore, these guidelines require that when LSP is performed, both directions of response (positive and negative) be evaluated.

5.8.2.2.1 Period Determination

Fundamental periods T_{1U} and T_{1D} for the undamaged and damaged building respectively shall be calculated for each of two orthogonal directions of building response, using standard methods of modal analysis. For the damaged case, the model should account for the damage sustained as determined by field investigation. 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 ASCE 7-22 for this purpose, may be inaccurate for damaged structures.

In many WSMF buildings, only a rather small percentage of the building's beams and columns are intended and detailed to provide lateral resistance. However, in such buildings, the presence of the gravity framing, particularly the columns, can add substantial stiffness to the building, affecting both its natural period of vibration, and also its lateral resistance. It is permissible to include the gravity framing in the analytical models used to determine building period, or not. However, if the gravity framing is incorporated for the damaged building period determination, it shall also be used for the undamaged building.

5.8.2.3 Determination of Actions and Deformations

5.8.2.3.1 Pseudo Lateral Load

A pseudo lateral load, given by Equation 7-21 of ASCE 41, 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.

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 7.4.1.3.2 of *ASCE 41*.

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 Column Demands

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.

5.8.2.3.5 Controlling Analysis

The value of $S_{au\text{-CP}}(T_{1U})$ and $S_{ad\text{-CP}}(T_{1D})$ shall be separately determined for each orthogonal axis and each direction of response (positive and negative).

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 *ASCE 41*, except as specifically noted herein.

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. 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 7.4.2.2 of *ASCE 41*. 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 same damping ratio shall be used for both the damaged and undamaged building models.

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 and concurrent effects should be accounted for as noted in Section 7.2.6 of *ASCE 41*.

5.8.3.3 Determination of Actions and Deformations

5.8.3.3.1 Determination of Column Demands

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.

5.8.3.3.2 Controlling Analysis

The value of $S_{au\text{-CP}}(T_{1U})$ and $S_{ad\text{-CP}}(T_{1D})$ shall be separately determined for each orthogonal axis and each direction of response (positive and negative).

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 structure is displaced to a target displacement, and resulting internal deformations and forces are determined. This process is followed independently for each of two orthogonal response directions and both the positive and negative direction of response in each, for both the undamaged and damaged structure. 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. 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.

For each analysis performed, the spectral acceleration for the undamaged structure $S_{au\text{-CP}}(T_{1U})$ and damaged structure $S_{ad\text{-CP}}(T_{1D})$ at which the structure marginally conforms to the Collapse Prevention performance level, per *ASCE 41* is determined.

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 to determine the performance level. See Section 7.4.3 of *ASCE 41*.

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 δ_t, given by Equation 7-29 of*ASCE 41*. 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 δ_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 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 7.4.3.2.4 of *ASCE 41*.

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 7.4.3.2.6 of *ASCE 41*.

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 7.2.6 in *ASCE 41*. 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 7.2.4.2.3 of ASCE 41.

5.8.4.3 Determination of Actions and Deformations

5.8.4.3.1 Target Displacement

The target displacement δ_t for buildings with rigid diaphragms at each floor level shall be estimated using the procedures of Section 7.4.3.3.2 of ASCE 41, for both the undamaged and damaged conditions. 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. The spectral accelerations $S_{au\text{-CP}}(T_{1U})$ and $S_{ad\text{-CP}}(T_{1D})$ at which the damaged and undamaged models just meet Collapse Prevention performance shall be determined.

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.

5.8.5 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.

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 *ASCE 41*.

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 2.3.4 of *ASCE 41* 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. Ground motions shall be amplitude scaled to determine the values of $S_{au\text{-CP}}(T_{1U})$ and $S_{ad\text{-CP}}(T_{1D})$ at which the undamaged and damaged structures, each just meet Collapse Prevention performance.

Consideration of multidirectional excitation effects required by Section 7.2.6 of *ASCE 41* 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 7.2.4.2 of *ASCE 41*.

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.

If any of the ground motions produces instability or collapse, the structure shall be considered not to meet Collapse Prevention performance for that level of ground motion.

5.9 Mathematical Modeling

5.9.1 Modeling Approach

In general, a damaged or undamaged steel frame building should be modeled, analyzed and evaluated as a three-dimensional assembly of elements and components. Although two-dimensional models may provide adequate evaluation 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 *ASCE 7*.

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 angles or single clip plates can provide significant secondary stiffness to a structure and should in general be modeled when performing post-earthquake 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, which can be neglected.

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 post-earthquake 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 δ_{max} from this effect, at any point on any floor diaphragm, exceeds the average displacement δ_{avg} by more than 10%.

Commentary: Accidental torsion is an artificial device used by the building codes to account for actual torsion that can occur, but is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation. Such torsion can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner in the structure. For example, if the frames on the east and west sides of a structure have similar elastic stiffness, the structure may not have significant torsion during elastic response. However, if the frame on the east side of the structure yields significantly sooner than the framing on the west side, then inelastic torsion will develop. Rather than requiring that an accidental torsion be applied in the analysis, as do the building codes, these Recommended Criteria 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. Foundation flexibility assumptions should be the same for damaged and undamaged buildings unless damage is such that the stiffness of foundations has changed as a result of the earthquake.

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 ψ_i should be calculated for each direction of response, as follows:

$$\psi_i = \frac{P_i \delta_i}{V_{yi} h_i} \qquad (5\text{–}5)$$

where:

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

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

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

δ_i = lateral 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 ψ_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity ψ_i in a story exceeds 0.1, the analysis of the structure should explicitly consider the geometric nonlinearity introduced by P-Δ effects. When ψ_i in a story exceeds 0.3, the structure shall be considered unstable, unless a detailed global stability capacity evaluation for the structure, considering P-Δ effects, is conducted.

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-Δ effects (FEMA-355F). For a given structure, it is believed that if the value of ψ is less than 0.3 the effects of P-Δ have been adequately considered by these general procedures. For values of ψ greater than this limit the statistics on frame interstory drift capacities contained in Section 5.10 are inappropriate. For such frames explicit determination of interstory drift capacities, considering P-Δ effects using the detailed performance evaluation procedures 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:

$$V_{yi} = \frac{2\sum_{j=1}^{n} M_{pGj}}{h_i} \tag{5-6}$$

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 Framing Properties

Elastic and nonlinear framing properties should be taken in accordance with the requirements of *AISC 342*.

5.9.8 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 post-earthquake 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.

5.9.9 Undamaged Connection Modeling

Undamaged connections should be modeled in accordance with the requirements of *AISC 342*.

5.9.10 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.

5.9.10.1 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.
- 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.
- 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 5-1 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.

5.9.10.2 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.

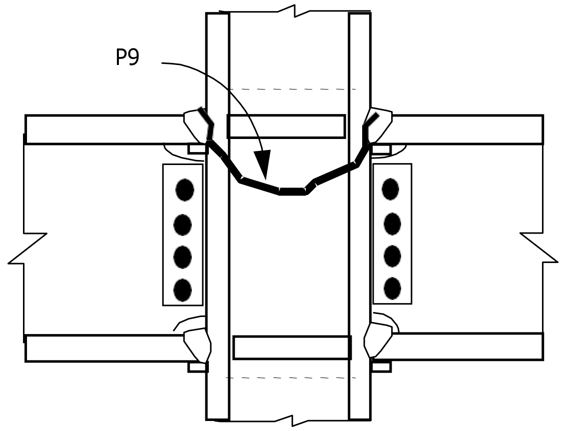


Figure 5-1 Type P9 Panel Zone Damage

- 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. See also Section C5 of AISC 342.
- 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.

5.9.10.3 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.

5.9.10.4 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

For each of the damaged and undamaged models, determine the quantity $S_{au\text{-}CP}(T_{1U})$ from the formula:

$$S_{ad-CP}(T_{1U})_i = \frac{S_D(T_{1U})}{S_D(T_{1D})} S_{ad-CP}(T_{1D})_i \qquad (5\text{-}8)$$

where "i" is the direction of loading.

Also, determine the ratio R_i as:

$$R_i = \frac{S_{ad-CP}(T_{1u})_i}{S_{au-CP}(T_{1u})_i} \tag{5-9}$$

Determine $R_{i\text{-}min}$ as the minimum of the R_i.

If S_{1E} is less than or equal to $0.3S_{D1}$ and $R_{i\text{-}min}$ is less than 0.9, the structure shall be considered to have experienced Disproportionate Earthquake Damage.

If $R_{i\text{-}min}$ is less than 0.7, the structure shall be considered to have experienced substantive damage.

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 and any analysis that was performed, and provide an interpretation of the results of the inspection program and analysis, and a statement as to whether the structure has experienced either Disproportionate Earthquake Damage or Substantial Structural Damage. 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 indicating the reduction in capacity ($R_{i\text{-}min}$).
- 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, *ASCE 41 Seismic Evaluation and Retrofit of Existing 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, ASCE 41 Seismic Evaluation and Retrofit of Existing 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 future 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. ample repair details for various types of damage are included in these recommended criteria, for reference, only. Sections 6.3 through 6.5 contain numerous references to the AWS D1.1/D1.1M Structural Welding Code. Since the publication of FEMA 352 and FEMA 353, AWS has published AWS D1.8/D1.8M – Structural Welding Code – Seismic Supplement that incorporates new information on seismic welding procedures that should be considered by the engineer in the implementation of repair details in these sections.

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.

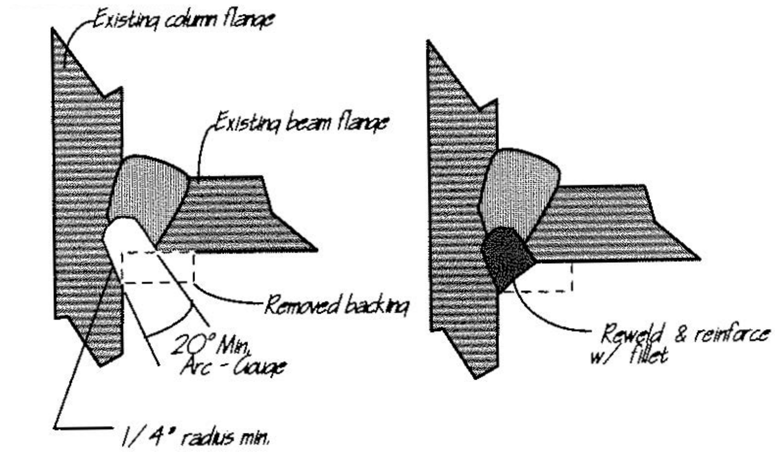
6.3.2 Weld Fractures - Type W Damage

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*, sixteenth edition, page 16.1-133.). 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

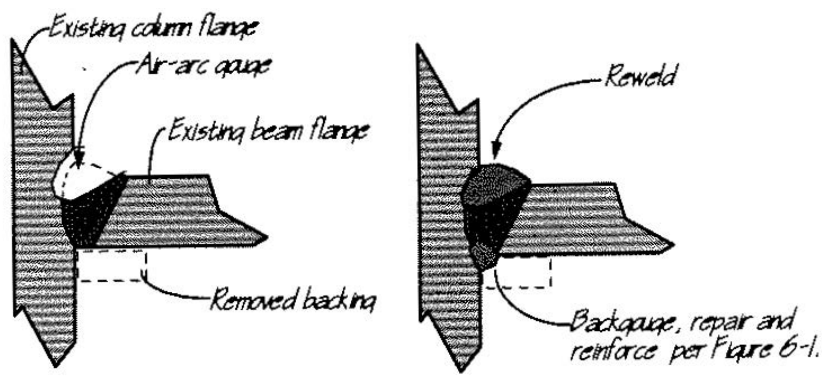
Damage or Defect Class	Figure
Rejectable defects at weld root	Figure 6-1, Figure 6-2
W2	Figure 6-3
W3	Figure 6-3
W4	Figure 6-3
W5	Figure 6-3



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.

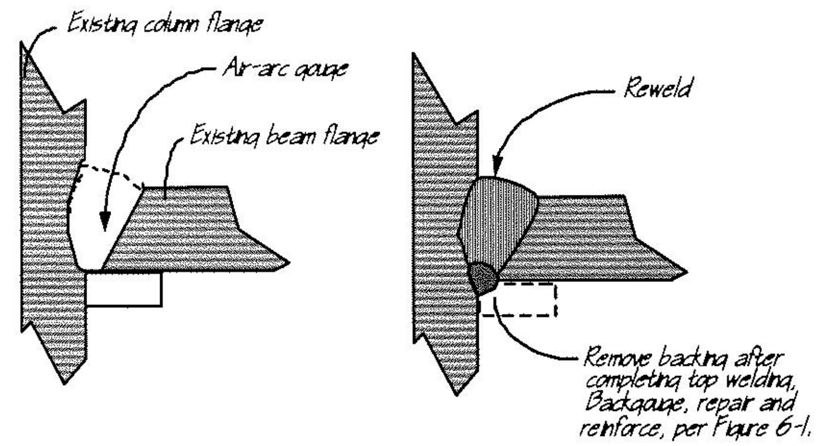
Figure 6-1 Gouge and Re-weld of Root Defect or Damage



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- 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.

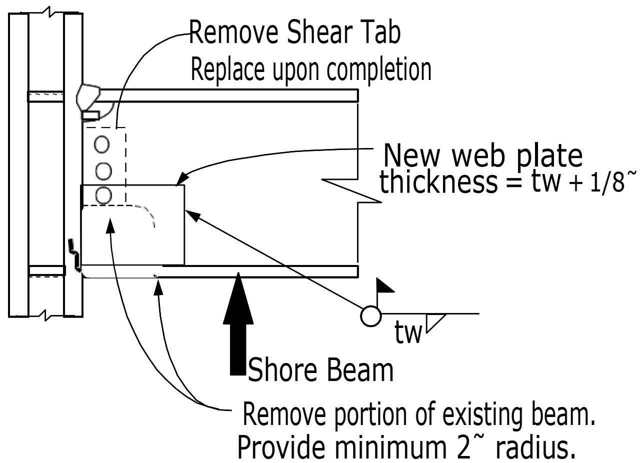


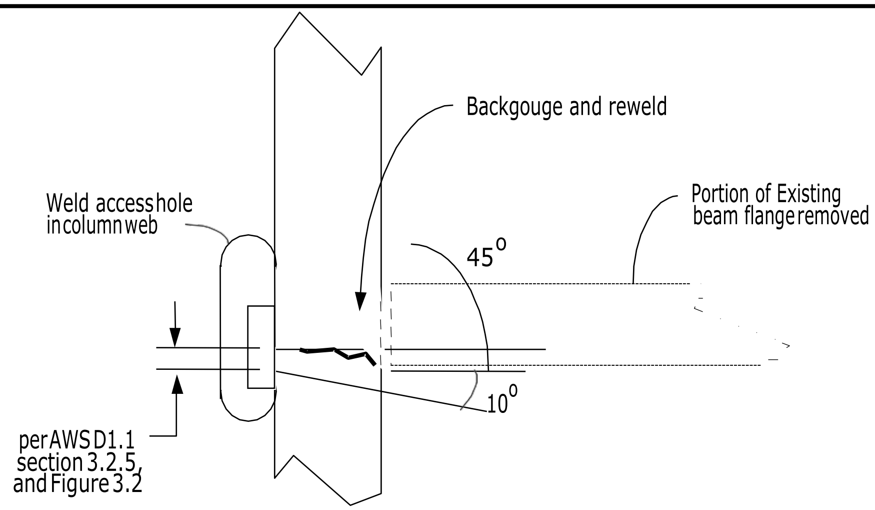
Figure 6-4 Temporary Removal of Beam Section for Access

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 or Special Applications, in conjunction with AWS K6. 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 A ccess	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

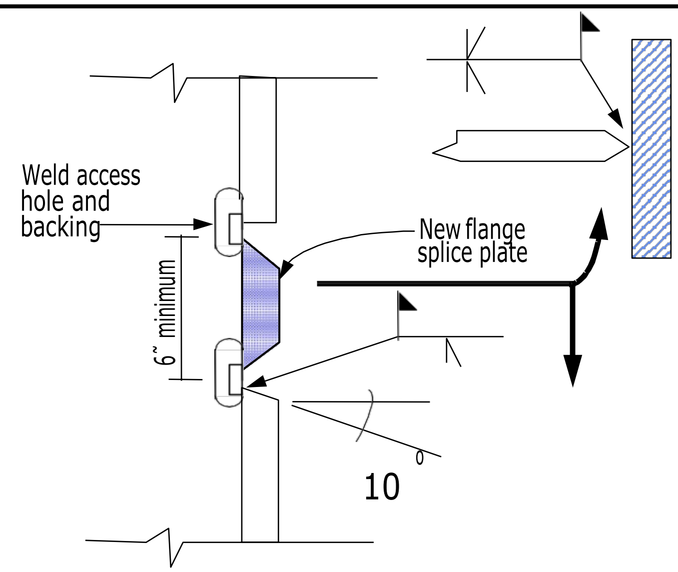
Commentary: Special attention should be given to conditions where more than 20% of the column cross section will be removed at one time, as special temporary shoring may be warranted. In addition, care should be taken when applying heat to a flange or web containing a fracture, as fractures have been 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:

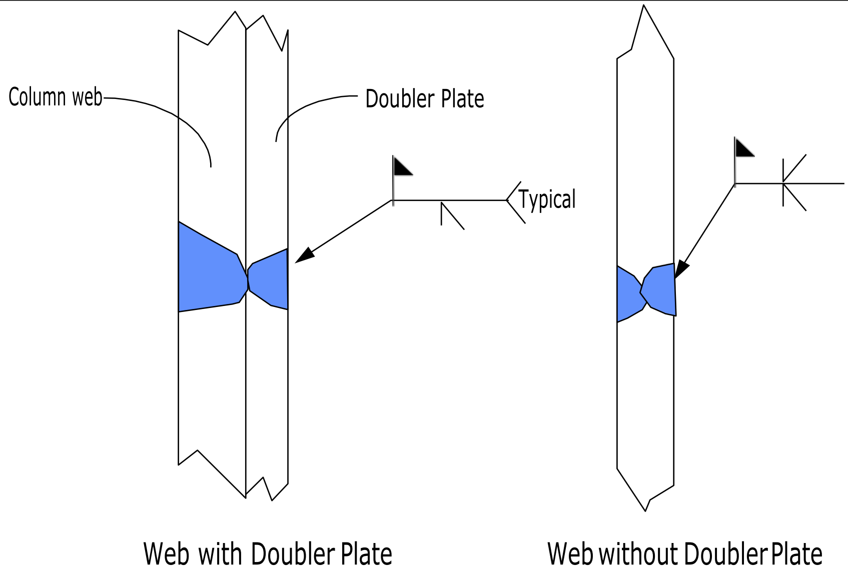
1. Investigate extent of fracture by UT to confirm that fractures are contained with the 4 degree angle zone of a standard pre-qualified JP groove weld as defined in Figure 2.4, AWS D1.1, Joint Designation B-U4a-G
2. Provide 10° bevel on lower flange plate, to channel slag out of joint.
3. Grind all 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"/+¼". 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.

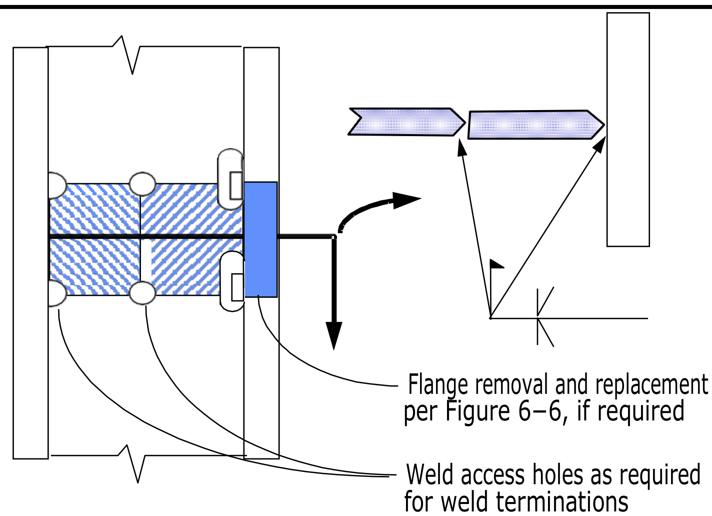
Figure 6-6 Replacement of Column Flange Repair



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



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"/+⅛").

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.

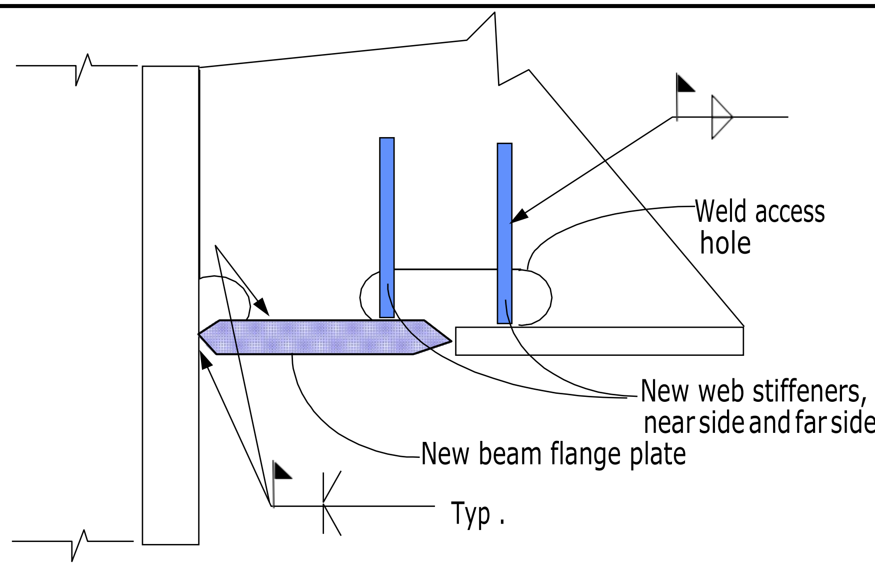
6.3.5 Girder Flange Fractures - Type G3 to G5

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 o 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

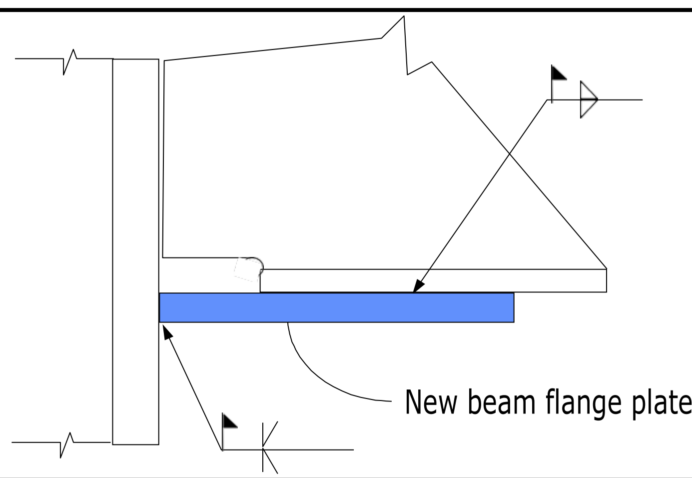
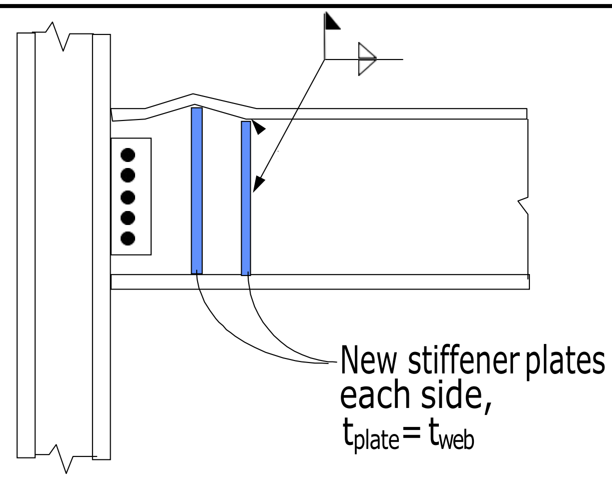


Figure 6-10 Alternative Beam Flange Plate Replacement

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 *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, and AISC 341-22). 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 *RCSC Specification for Structural Joints using High Strength Bolts (RCSC,* 2020), ASTM A490 bolts and galvanized bolts should not be re-tightened and re-used under any circumstances. Other 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 or 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 or 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 or joint preparation based on material property and thickness, as well as welding parameters. eld 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. Sections 6.3 through 6.5 contain numerous references to the AWS D1.1/D1.1M Structural Welding Code. Since the publication of FEMA 352 and FEMA 353, AWS has published AWS D1.8/D1.8M – Structural Welding Code – Seismic Supplement that incorporates new information on seismic welding procedures that should be considered by the engineer in the implementation of repair details in these sections.

> *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 success fully making the repair welds required. All welders should be qualified to the *AWS D1.1* requirements or 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 or all the configurations that may be encountered. Specific additional training and/or experience may be required or 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 or 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 or repair. Sections 6.3 through 6.5 contain numerous references to the AWS D1.1/D1.1M Structural Welding Code. Since the publication of FEMA 352 and FEMA 353, AWS has published AWS D1.8/D1.8M – Structural Welding Code – Seismic Supplement that incorporates new information on seismic welding procedures that should be considered by the engineer in the implementation of repair details in these sections.

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 California Building Code 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 or adherence to the approved Welding Procedure Specification (WPS) and *AWS D1.1,* starting with preliminary tack welding and it-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 or welded joints in severe service and with significant consequences of failures should be capable of depositing weld metal with a minimum notch toughness of 40ft-lbs at the anticipated service temperature and 0 ft-lbs at 0ºF. Refer to AWS D1.8 for requirements for Demand Critical welds. 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 or 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 8.1.1. 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- /8 inch. The finished surface should be visually inspected or contour and any visually apparent indications. This should be followed by magnetic particle testing (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 def ect(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 PS. 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 or 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 W PS.

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 or 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. 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 or clearing and inspection of the initiation point of the weld be ore 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. A fter 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 or 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 lush to the beam flange. Imperfections of 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 or 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.

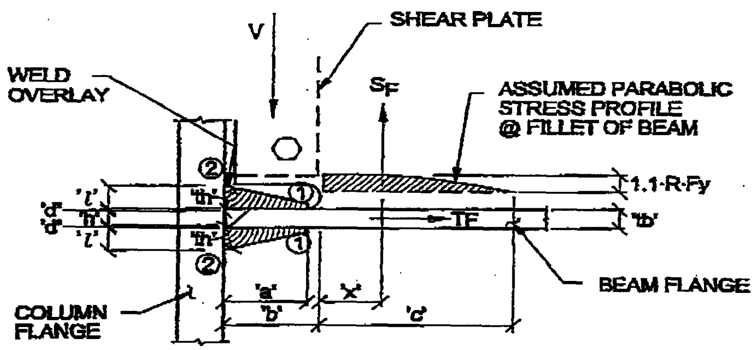
Repair of cracks and defects in welds may warrant total removal of the original weld, particularly if multiple cracks are present. If the entire weld plus some base metal is removed, care must be taken not to exceed the root opening and bevel limits of *AWS D1.1* unless a qualified by test PS is used. 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 rom surface discontinuities within the range of defects that would go undetected, effectively converts surface defects and small fractures in the existing C JP weld into internal defects, and further, provides local reinforcement of the joint or resistance of applied stresses. This repair approach was developed by an independent task group of engineers and researchers in the Los A ngeles 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., $\frac{1.1R_yM_p}{L_c} + V_{gravity}$, *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 trans erred 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 0% 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.

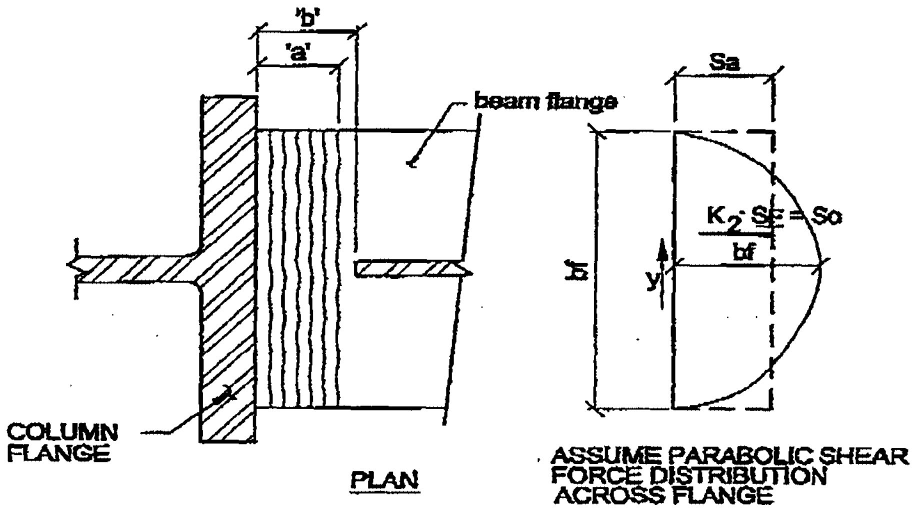


Figure 6-13 Plan View and Assumed Stress Distribution for Weld Overlay Design

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 is a pullout type failure of the column flange material. The failure surface should be conditioned to a concave surface by grinding and inspected or 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 or subsequent passes. The final surface should be ground smooth and lush 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

NOTE FOR USE OF FEMA 352: As a resource for application of City of San Francisco Administrative Bulletin AB-XXX "Post-Earthquake Inspection, Evaluation, Repair and Retrofit Requirements for "Pre-Northridge" Welded Steel Moment Frame Buildings": FEMA 352 will form the basis for application AB XXX. Modifications to the document are provided for use by engineers and SFDBI to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

Appendix A of FEMA 352 is not adopted for use as part of AB-XXX. Any use of Appendix A of FEMA 352 may only be done with prior approval of SFDBI.

APPENDIX B: SAMPLE PLACARDS

NOTE FOR USE OF FEMA 352: As a resource for application of City of San Francisco Administrative Bulletin AB-XXX "Post-Earthquake Inspection, Evaluation, Repair and Retrofit Requirements for "Pre-Northridge" Welded Steel Moment Frame Buildings": FEMA 352 will form the basis for application AB XXX. Modifications to the document are provided for use by engineers and SFDBI to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

Appendix B of FEMA 352 is not adopted for use as part of AB-XXX. Any use of Appendix B of FEMA 352 may only be done with prior approval of SFDBI.

APPENDIX C: SAMPLE INSPECTION FORMS

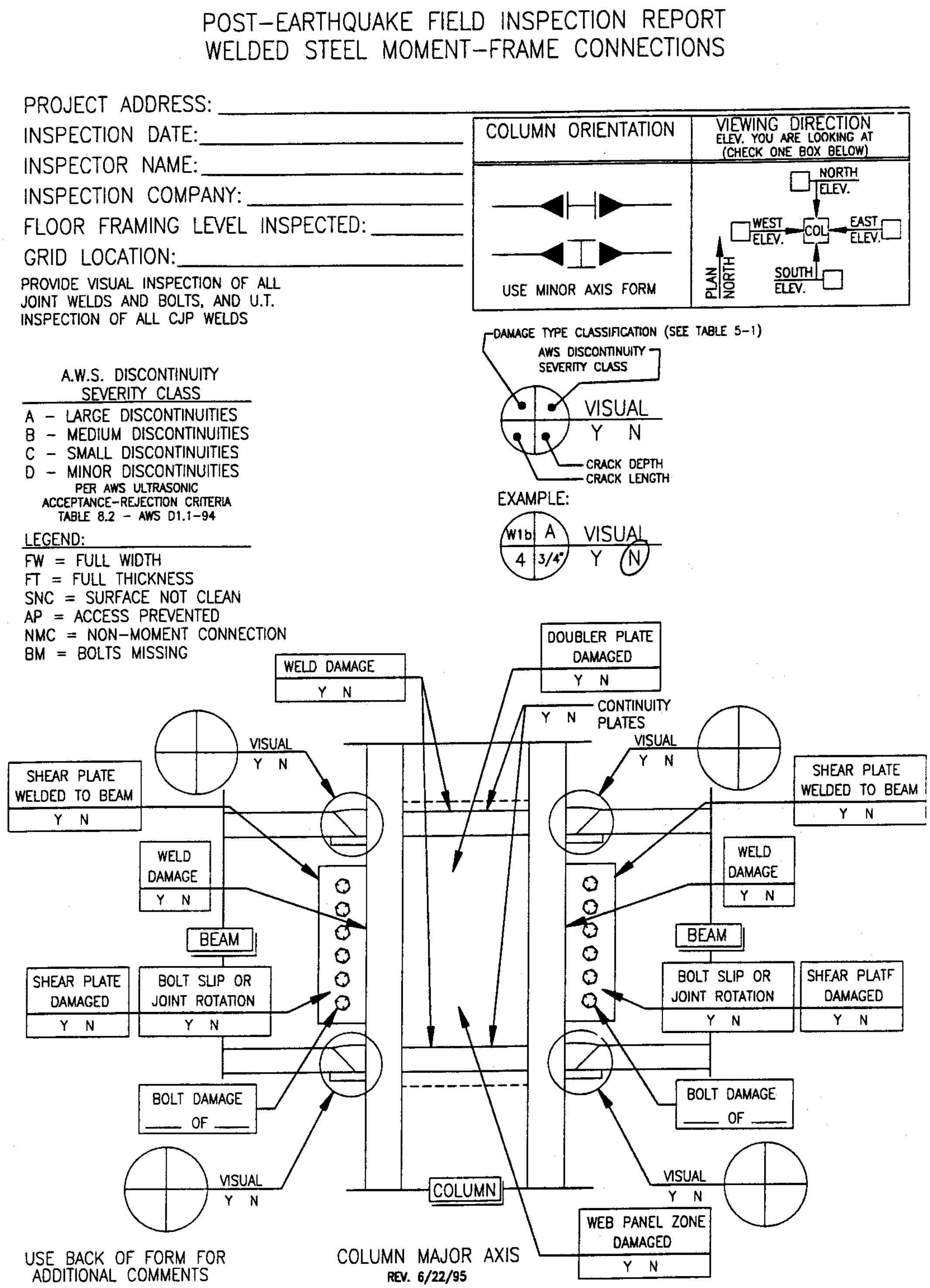


Figure C-1 Inspection Form – Major Axis Column Connection

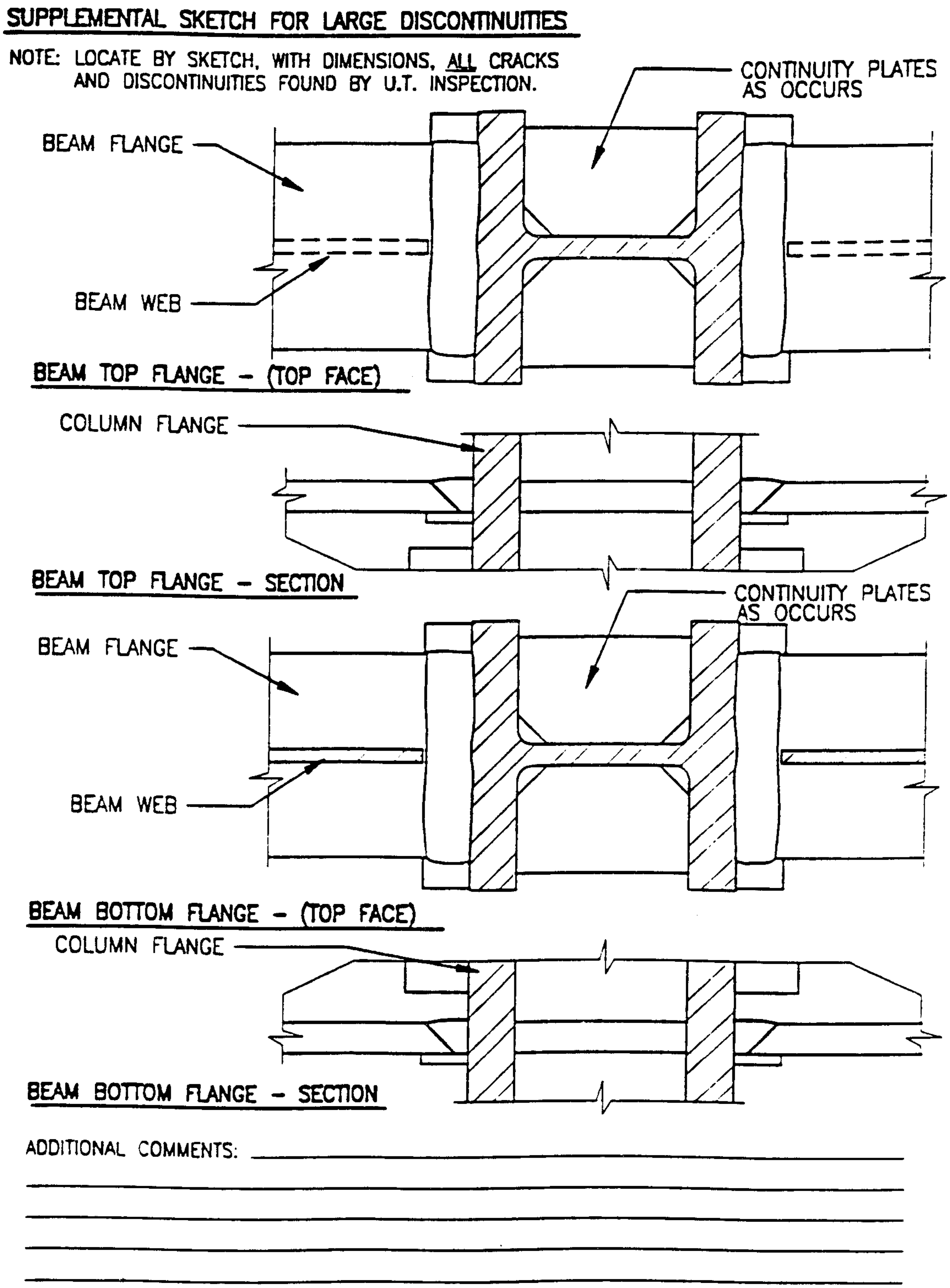


Figure C-2 Inspection Form – Large Discontinuities – Major Axis

POST-EARTHQUAKE FIELD INSPECTION REPORT
WELDED STEEL MOMENT-FRAME CONNECTIONS

PROJECT ADDRESS: ____________________

INSPECTION DATE: ____________________

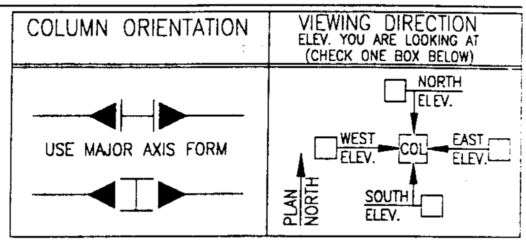
INSPECTOR NAME: ____________________

INSPECTION COMPANY: ____________________

FLOOR FRAMING LEVEL INSPECTED: ________

GRID LOCATION: ____________________

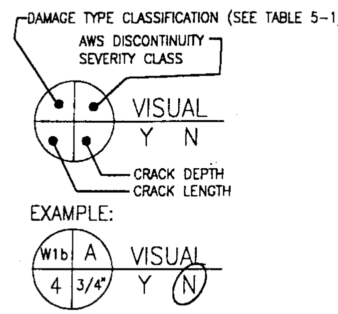
PROVIDE VISUAL INSPECTION OF ALL JOINT WELDS AND BOLTS, AND U.T. INSPECTION OF ALL CJP WELDS



A.W.S. DISCONTINUITY SEVERITY CLASS

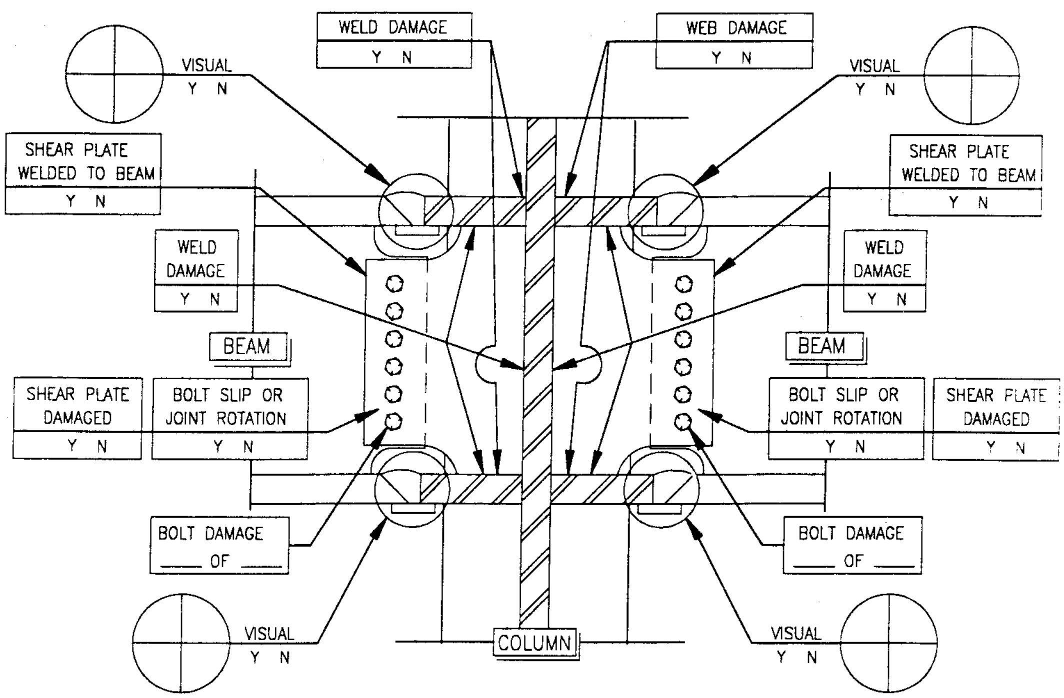
A – LARGE DISCONTINUITIES
B – MEDIUM DISCONTINUITIES
C – SMALL DISCONTINUITIES
D – MINOR DISCONTINUITIES

PER AWS ULTRASONIC ACCEPTANCE-REJECTION CRITERIA TABLE 8.2 – AWS D1.1-94



LEGEND:

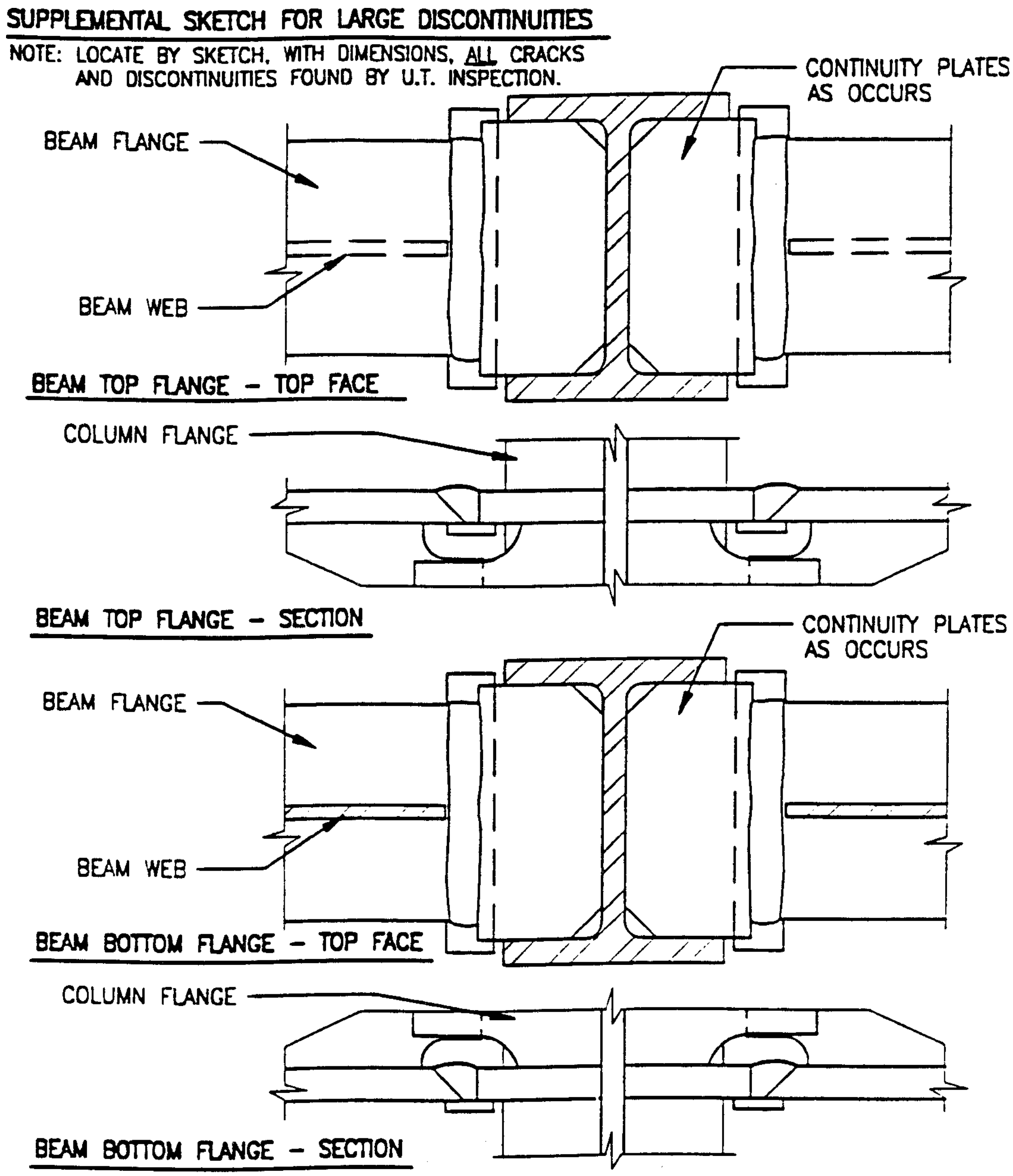
FW = FULL WIDTH
FT = FULL THICKNESS
SNC = SURFACE NOT CLEAN
AP = ACCESS PREVENTED
NMC = NON-MOMENT CONNECTION
BM = BOLTS MISSING



USE BACK OF FORM FOR ADDITIONAL COMMENTS

COLUMN MINOR AXIS
REV. 6/22/95

Figure C-3 Inspection Form – Minor Axis Column Connection



ADDITIONAL COMMENTS: ____

____

____

____

____

Figure C-4 Inspection Form – Large Discontinuities – Minor Axis

REFERENCES, BIBLIOGRAPHY, AND ACRONYMS

NOTE FOR USE OF FEMA 352: As a resource for application of City of San Francisco Administrative Bulletin AB-XXX "Post-Earthquake Inspection, Evaluation, Repair and Retrofit Requirements for "Pre-Northridge" Welded Steel Moment Frame Buildings": FEMA 352 will form the basis for application AB XXX. Modifications to the document are provided for use by engineers and SFDBI to refer to current technical documents and the results of research investigations carried out since the publication of FEMA 352 in 2000.

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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A36, Specification for Carbon Structural Steel

A325, Specification for Structural Bolts, Steel, Heat-Treated, 120/105 ksi Minimum Tensile Strength

A435, Straight Beam Ultrasonic Examination of Steel Plates

A490, Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength

A563, Specification for Carbon and Alloy Steel Nuts

A572, Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel

A898, Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes

A913, Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process

A992, Standard Specification for Steel for Structural Shapes for Use in Building Framing

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E994, Standard Guide for Laboratory Accreditation Systems

E1212, Standard Practice for Establishment and Maintenance of Quality Control Systems for Nondestructive Testing Agencies

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F959, Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners

F1554, Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength

F1852, Specification for “Twist-Off” Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

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ANSI/AWS A5.5 Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding

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ANSI/AWS A5.29, Specification for Low-Alloy Steel Electrodes for Flux-Cored Arc Welding

AWS D1.1-, Structural Welding Code – Steel

AWS D1.3, Structural Welding Code – Sheet Steel

AWS D1.4, Structural Welding Code – Steel Reinforcing Bars

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Acronyms.

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

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

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