FEDERAL EMERGENCY MANAGEMENT AGENCY FEMA XXX/January, 1999

Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Resisting Frame Structures

Program to Reduce the Earthquake Hazards of Steel Moment Frame Structures

50% DRAFT

Working Draft

This document has been produced as a preliminary working draft as part of the SAC Joint Venture's project to develop practice guidelines for design, evaluation, repair, and retrofit of moment-resisting steel frame structures. The purpose of this draft is to permit the project development team and prospective users of the guidelines to explore the basic data requirements and alternative methods of presenting this data in an eventual series of guideline documents. Although portions of the document must necessarily appear in the form of an actual guideline, it is not intended to serve as an interim guideline document. Information contained in this document is incomplete and in some cases, is known to be erroneous or otherwise incorrect. Information presented herein should not be used as the basis for engineering projects and decisions, nor should it be disseminated or attributed.

Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Resisting Frame Structures

Report No. SAC-XX-XX-XX

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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January, 1999

THE SAC JOINT VENTURE

SAC is a joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and California Universities for Research in Earthquake Engineering (CUREe), formed specifically to address both immediate and long-term needs related to solving performance problems with welded steel moment frame connections discovered following the 1994 Northridge earthquake. SEAOC is a professional organization composed of more than 3,000 practicing structural engineers in California. The volunteer efforts of SEAOC's members on various technical committees have been instrumental in the development of the earthquake design provisions contained in the *Uniform Building Code* as well as the *National Earthquake Hazards Reduction Program (NEHRP) Provisions for Seismic Regulations for New Buildings and other Structures*. The Applied Technology Council is a non-profit organization founded specifically to perform problem-focused research related to structural engineering and to bridge the gap between civil engineering research and engineering practice. It has developed a number of publications of national significance including ATC 3-06, which serves as the basis for the *NEHRP Recommended Provisions*. 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. This collection of university earthquake research laboratory, library, computer and faculty resources is 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 subcontractor universities and organizations from around the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of steel moment frame structures.

DISCLAIMER

 These guidelines have been prepared by the SAC Joint Venture with funding provided by the Federal Emergency Management Agency, under contract number EMW-95-C-4770.

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1. INTRODUCTION

1.1 Purpose

The purpose of this *Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Resisting Frame Structures* is to provide engineers and building officials with guidance for performing post-earthquake damage assessments and repairs of welded steel moment-resisting frame (WSMF) structures. It is one of a series publications prepared by the SAC Joint Venture addressing the issue of the seismic performance of moment-resisting steel frame buildings. Companion publications include:

- *Seismic Design Criteria for new Moment-Resisting Steel Frame Construction* - These guidelines provide recommended design criteria and recommendations for new buildings incorporating moment-resisting steel frame construction intended to provide for construction capable of reliably meeting alternative seismic performance objectives.

- *Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Resisting Frame Construction* - These guidelines provide recommendations for methods to evaluate the probable performance of steel frame structures in future earthquakes and to retrofit these structures for improved performance.

- *Quality Assurance Guidelines for Moment-Resisting Steel Frame Construction* - These guidelines provide recommendations to engineers and building officials for methods to ensure that steel frame structures are constructed with adequate construction quality to perform as intended when subjected to severe earthquake loading.

Commentary: When a community is affected by a severe earthquake, many buildings are likely to become damaged and some, as a result of this damage, may pose a significant safety hazard. In many communities affected by past earthquakes, the building official, in fulfillment of his charge to protect the public safety through regulation of building occupancy, has instituted a program 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 large number of buildings present in a community, relative to the number of trained inspection personnel available, it is usually necessary to limit these post-earthquake inspections to those structures most likely to have been severely damaged and to make an assessment of the severity of damage in a very rapid manner.

If the initial rapid post-earthquake assessment reveals the that the structure may have sustained significant damage and may no longer be safe for occupancy, the building is typically tagged with a placard to inform the owner and public of this condition. The building owner is then typically given a period of time during which he 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.

This publication provides guidelines for performing the rapid post-earthquake assessments, typically conducted by the building official; 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 the earthquake event. This document does not specifically provide recommendations for upgrade of a building, to improve its performance in the event of future earthquake ground shaking. It should be noted that in many cases, when a building experiences severe damage in an event, this 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, this should be considered by the Owner at the time structural repairs are conducted. For technical guidelines on evaluating the advisability of upgrades, and methods of designing such upgrades, refer to FEMA-XXX, Evaluation and Upgrade Criteria for Welded Moment-Resisting Steel Frame Construction.

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 that in the process of conducting repairs, conditions of structural irregularity, discontinuity, or strength or stiffness deficiency are not introduced into the structure, or existing such conditions made more severe.

1.2 Intent

These guidelines are primarily intended for three different groups of potential users:

a) Engineers engaged in the evaluation and repair of steel frame structures that have been subjected to the effects of strong earthquake ground shaking.

b) Regulators and building departments in localities that have experienced the effects of strong earthquake ground shaking.

c) Organizations engaged in the development of building codes and standards for regulation of the design and construction of steel frame structures that may be subject to the effects of earthquake ground shaking.

The fundamental goal of the information presented in these guidelines is to assist the technical community in implementing effective programs to evaluate steel frame buildings affected by strong earthquake ground shaking and to repair structures that have been damaged by such structures.

1.3 Background

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

WSMF construction is commonly used throughout the United States and the world, particularly for mid- and high-rise construction. Prior to the Northridge earthquake, this type of construction was commonly considered to be very ductile and essentially invulnerable to damage that would significantly degrade structural capacity, due to the fact that severe damage to such structures had rarely been reported in past earthquakes and there was no record of earthquake-induced collapse of such buildings. The discovery of brittle fracture damage in a number of buildings affected by the Northridge Earthquake called for re-examination of this premise. In general, WSMF buildings in the Northridge Earthquake met the basic intent of the building codes, to protect life safety. However, the structures did not behave as anticipated and significant economic losses occurred as a result of the connection damage. 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 some cases, long term loss of use of space within damaged structures.

WSMF 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 WSMF structures 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.

Observation of damage sustained by buildings in the Northridge Earthquake indicates 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 elastic. Typically, but not always, fractures initiated at, or near, the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-1). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

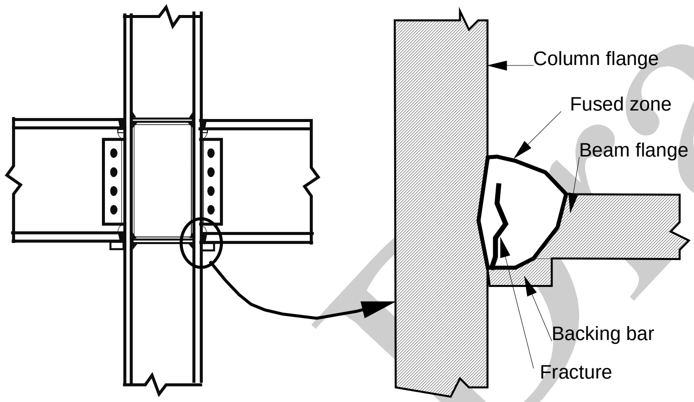
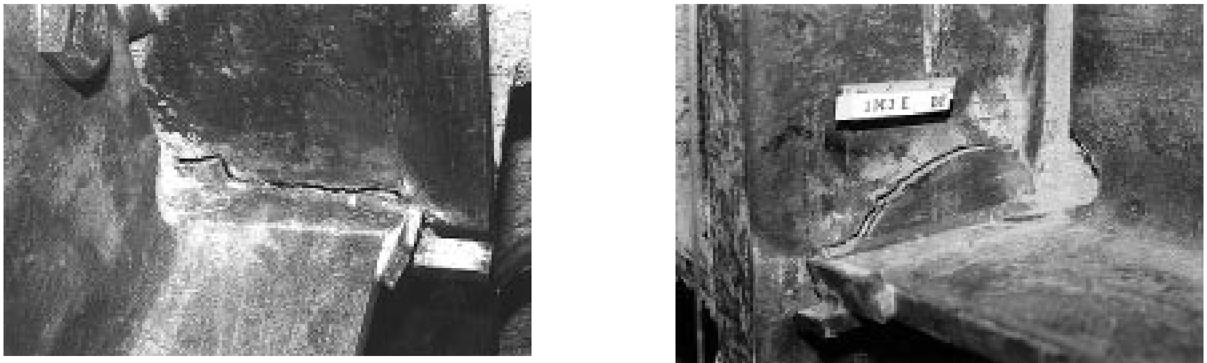


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

In some cases, the fractures progressed completely through the thickness of the weld, and if 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 1-2a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the CJP weld (Figure 1-2b). 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.

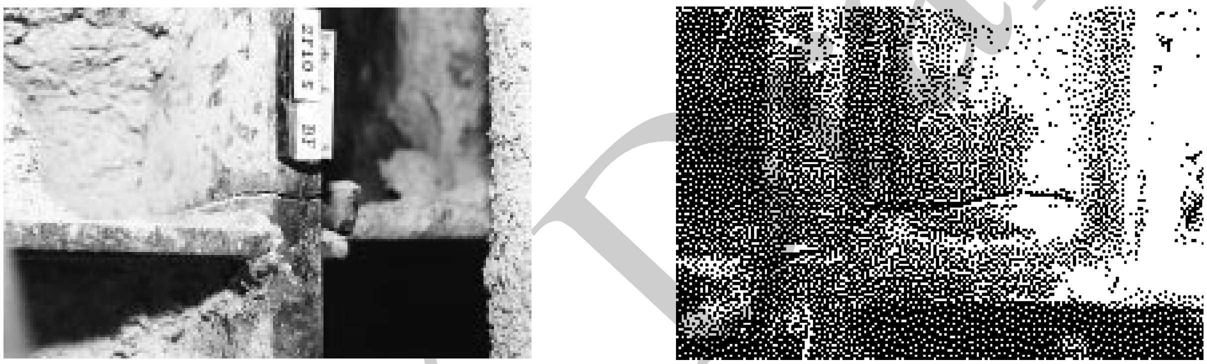
A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-3b). Investigators have reported some instances where columns fractured entirely across the section.



a. Fracture at Fused Zone

b. Column Flange “Divot” Fracture

Figure 1-2 - Fractures of Beam to Column Joints



a. Fractures through Column Flange

b. Fracture Progresses into Column Web

Figure 1-3 - Column Fractures

Once such fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and strength to resist loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining 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, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-4).

Figure 1-4 - Vertical Fracture through Beam Shear Plate Connection

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 post-earthquake damage evaluations difficult. Until news of the discovery of connection fractures in some buildings began to spread through the engineering community, it was relatively common for engineers to perform cursory post-earthquake evaluations of WSMF buildings and declare that they were undamaged. Unless a building exhibits overt signs of damage, such as visible permanent inter-story-drifts, in order to reliably determine if a building has sustained connection damage it is often 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 WSMF buildings sustained so much connection damage that it was deemed more practical to demolish the structure rather than to repair it.

In response to concerns raised by this damage, the Federal Emergency Management Agency (FEMA) entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused study of the seismic performance of welded steel moment connections and to develop recommendations for professional practice. Specifically, these recommendations were intended to address 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 more definitively explore 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 (SAC 1995c, SAC 1995d, SAC 1995e, SAC 1995f, SAC 1995g, SAC 1996) formed the basis for the development of FEMA 267 - *Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures* (SAC, 1995b), 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 Northridge earthquake.

In the time since the publication of *FEMA-267*, SAC has continued to perform problem-focused study of the performance of moment resisting steel frames and connections of various configurations. This work has included detailed analytical evaluations of buildings and connections, parametric studies into the effects on connection performance of connection configuration, base and weld metal strength, toughness and ductility, as well as additional large scale testing of connection assemblies. As a result of these studies, as well as independent research conducted by others, it is now known that a large number of factors contributed to the damage sustained by steel frame buildings in the Northridge earthquake. These included:

- design practice that favored the use of relatively few frame bays to resist lateral seismic demands, resulting in much larger member and connection geometries than had previously been tested;

- standard detailing practice which resulted in large inelastic demands at the beam to column connections;

- detailing practice that often resulted in large stress concentrations in the beam-column connection, as well as inherent stress risers and notches in zones of high stress;

- the common use of welding procedures that resulted in deposition of low toughness weld metal in the critical beam flange to column flange joints;

- relatively poor levels of quality control and assurance in the construction process, resulting in welded joints that did not conform to the applicable quality standards;

- excessively weak and flexible column panel zones that resulted in large secondary stresses in the beam flange to column flange joints;

- large increases in the material strength of rolled shape members relative to specified values;

1.4 Post-earthquake Evaluation and Repair

Post-earthquake evaluation of a welded moment-resisting steel frame is a multi-step process. Its intent is to identify buildings that have sustained structural damage, determine the extent and severity of this damage, assess the general implications of the damage with regards to building safety and determine appropriate actions regarding building occupancy and repair. Once a building has been determined to be significantly damaged, the structural engineer should conduct a more detailed evaluation of the residual structural integrity and safety of the structure and develop a detailed plan for repair, upgrade, demolition, or other action, as appropriate. Currently, most building codes only require repair when a structure has been damaged. As such, the focuse of this document is the identification and repair of damage. ; However, the extent, severity or characteristics of the damage may be sufficiently severe that the owner may wish to consider upgrading or modifying the structure to improve probable performance in future events. Prediction of structural performance during future seismic events and selection of appropriate upgrades to achieve desired performance is the subject of the companion documents, *FEMA-XXX Evaluation and Upgrade Criteria for Welded Moment-Resisting Steel Construction*.

A series of simple steps are outlined in Chapter 3 that allow rapid identification of buildings having actual or potential damage requiring more detailed evaluation and repair. This screening process incorporates a preliminary on-site inspection and review of design documents. Based on an assessment of this and other data regarding the severity of ground shaking at the building site and the damage to nearby structures, recommendations are developed regarding the probability the structure has sustained damage, the severity of damage, and the nature of appropriate actions. A report on these findings is prepared for the owner and others, as appropriate.

If a finding of probable significant damage is made, a more detailed evaluation of the structure is required. 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 procedure used for structural performance assessment in other guideline and criteria documents prepared by the FEMA/SAC project. Both of these procedures contain recommendations for inspection of some or all WSMF 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 guidelines 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. Guidelines for the assessment of seismic performance of the repaired building and recommendations for improved performance may be found in the companion publication, *FEMA-XXX*.

Figure 1-5 presents an overview of the post-earthquake assessment and repair process.

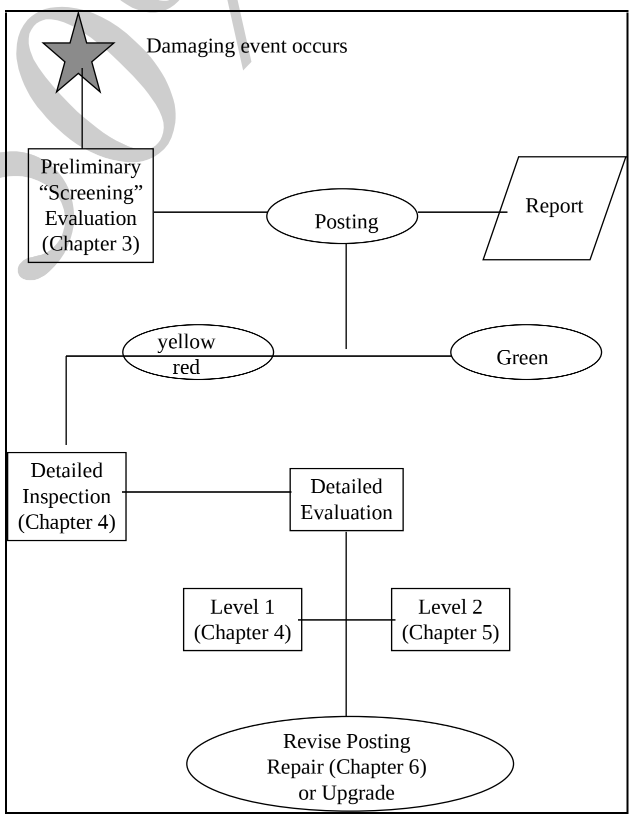


Figure 1-5 Flow Chart for Post-earthquake Actions

1.5 Application

This publication supersedes the post-earthquake evaluation and repair guidelines contained in FEMA-267, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, and the *Interim Guidelines Advisory*, FEMA-267a. It is intended to be used in coordination with and in supplement to the locally applicable building code and those national standards referenced by the building code. Building codes are living documents and are revised on a periodic basis. This document has been prepared based on the provisions contained in the *1997 NEHRP Recommended Provisions for the Regulation of New Buildings and Other Structures* (BSSC, 1997), the 1997 *AISC Seismic Specification* (AISC, 1997) and the 1996 *AWS D1.1 Structural Welding Code - Steel*, as it is anticipated that these documents will form the basis for the 2000 edition of the International Building Code. Users are cautioned to carefully consider any differences between the aforementioned documents and those actually enforced by the building department having jurisdiction for a specific project and to adjust the recommendations contained in these guidelines, accordingly.

1.6 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 the problem of the welded steel moment frame (WSMF) connection. SEAOC is a professional organization comprised of more than 3,000 practicing structural engineers in California. The volunteer efforts of SEAOC's members on various technical committees have been instrumental in the development of the earthquake design provisions contained in the *Uniform Building Code* as well as the *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings*. The Applied Technology Council is a non-profit organization founded specifically to perform problem-focused research related to structural engineering and to bridge the gap between civil engineering research and engineering practice. It has developed a number of publications of national significance including ATC 3-06, which served as the basis for the *NEHRP Recommended Provisions*. CUREe's eight institutional members are: the University of California at Berkeley, the California Institute of Technology, 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, the University of Southern California, and Stanford University. This collection of university earthquake research laboratory, library, computer and faculty resources is the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by subcontractor universities and organizations from around the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems in earthquake engineering.

The SAC Joint Venture developed a two phase program to solve the problem posed by the discovery of fractured steel moment connections following the Northridge Earthquake. Phase 1 of this program was intended to provide guidelines for the immediate post-Northridge problems of identifying damage in affected buildings and repairing this damage. In addition, Phase 1 included dissemination of the available design information to the professional community. It included convocation of a series of workshops and symposiums to define the problem;

development and publication of a series of Design Advisories (SAC-1994-1, SAC-1994-2, SAC-1995); limited statistical data collection, analytical evaluation of buildings and laboratory research; and the preparation of the *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures, FEMA-267*. The Phase 2 project was comprised of a longer term program of research and investigation to more carefully define the conditions which lead to the premature connection fractures and to develop sound guidelines for seismic design and detailing of improved or alternative moment resisting frame systems for new construction, as well as reliable retrofitting concepts for existing undamaged WSMF structures. Detailed summaries of the technical information that forms a basis for these guidelines are published in a separate series of State-of-Art reports (SAC, 1999a), (SAC, 1999b), (SAC, 1999c), (SAC, 1999d), and (SAC, 1999a).

1.7 Sponsors

Funding for Phases I and II of the SAC Steel Program 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 co-funding, 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.

1.8 Guideline Overview

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

- **Chapter 2 - Damage Classification**. This chapter provides an overview of the different types of structural damage that may be anticipated to occur in welded moment-resisting steel frame structures, 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 Evaluation**. This chapter provides a screening criteria that can be used to determine if there is sufficient likelihood that a welded steel moment-resisting frame structure has experienced damage to warrant further investigation. These Guidelines may be used by building officials to determine which buildings should be subjected to rapid post-earthquake damage assessments and how to conduct such assessments. The guidelines of this chapter may also be useful to engineers responding to requests by individual clients to assess the post-earthquake condition of a structure. In many cases, the preliminary damage assessment will lead to a recommendation to conduct more detailed evaluations. Chapters 4 and 5 provide guidelines for such evaluations.

- **Chapter 4 - Level 1 Detailed Evaluations**. Except for those structures that have experienced partial or total collapse, or that exhibit significant permanent inter-story drift, the results of a preliminary evaluation, conducted in accordance with Chapter 3 are likely to be inconclusive, with regard to the post-earthquake condition of the structure. This Chapter provides guidelines for conducting more detailed evaluations of the building to confirm its post-earthquake condition and develop recommendations for occupancy and repair of the structure as appropriate. It includes performing inspections of the fracture-critical 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 value of the damage index. This level of evaluation is too lengthy to be conducted as part of the rapid post-earthquake assessments typically conducted by building departments and is anticipated to be implemented by engineers engaged by the building owner, following the performance of a preliminary assessment.

- **Chapter 5 - Level 2 Detailed 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 the damaged building that can be used to supplement occupancy decisions suggested by the evaluation procedures of Chapter 4.

- **Chapter 6 - Repair**. This chapter provides guidelines for repair of the most common types of damage encountered in welded moment-resisting steel frame construction. Note that 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. Guidelines for performing structural upgrade may be found in a companion publication, *FEMA-XXX, Upgrade and Evaluation Criteria for Existing Welded Moment-Resisting Steel Frame Construction*.

2. INSPECTION AND CLASSIFICATION OF DAMAGE

2.1 Introduction

This chapter defines a uniform system for inspection, classification and reporting of damage to WSMF structures that have been subjected to strong earthquake ground shaking.

Structural damage observed in WSMF buildings following strong ground shaking can include yielding, buckling and excessive 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.

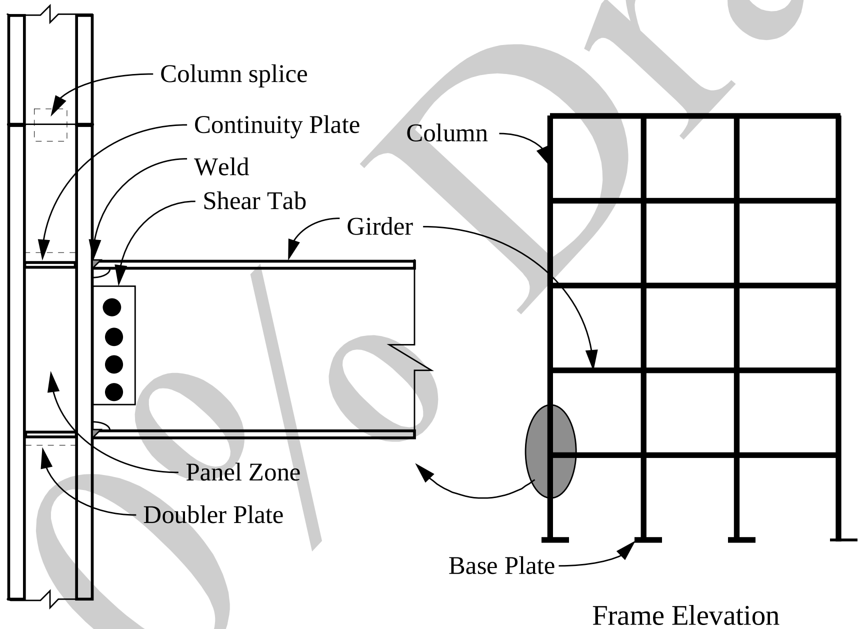


Figure 2-1 - Elements of Welded Steel Moment Frame

2.2 Damage Types

Damage to framing elements of WSMFs 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 WSMF structures that is utilized throughout these Guidelines. The damage types indicated below are not mutually exclusive. A given girder-column connection may experience several types of damage simultaneously. In addition to the individual element damage types, a damaged WSMF may also exhibit global effects, such as permanent inter-story drifts.

Following a detailed post-earthquake 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 WSMF structure. In a level 1 evaluation, conducted in accordance with Chapter 4 of these guidelines 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 guidelines, 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 post-earthquake period. Chapter 6 addresses specific techniques and design criteria recommended for the repair and modification of the different types of damage, keyed to these damage classifications.

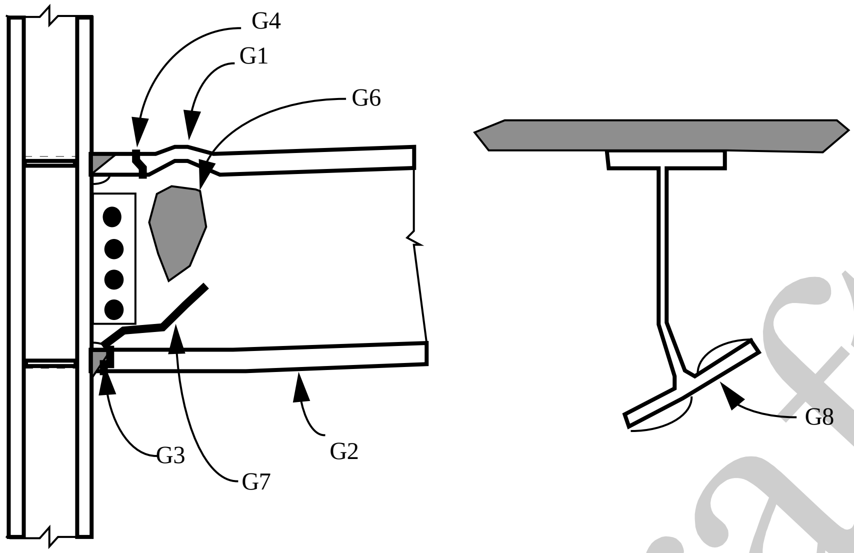
Commentary: The damage types contained in this Chapter are based on a system first defined in a statistical study of damage reported in NISTR-5625 (Yourself et. al.- 1994). The original classes contained in that study have been expanded somewhat to include some conditions not previously identified.

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. Eight separate types are defined in Table 2-1. Figure 2-2 illustrates these various types of damage. See section 2.2.3 and 2.3.4 for damage to adjacent welds and shear tabs, respectively.

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 HAZ (top or bottom)
G4	Flange fracture outside HAZ (top or bottom)
G5	Flange fracture top and bottom
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsion buckling of section



Note: condition G5 consists of types G3 and/or G4 damage occurring at both the top and bottom flanges.

Figure 2-2 - Types of Girder Damage

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.

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength. 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 a 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 loose tensile capacity under continued or reversed loading, however, it may retain some capacity in compression. Visually evident girder flange buckling should be repaired.

With the conventional structural steels used in WSMF buildings, girder flange cracking within the HAZ (type G3) is most likely to occur at connections in which improper welding procedures were followed, resulting in local embrittlement of the HAZ. Like the visually similar type G4 damage, 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. Little G4 or G5 *damage was actually seen in buildings following the Northridge Earthquake. In some cases, this damage was found to extend from the weld access hole in the web of the girder, a metallurgically complex area, into the flange.*

In the Northridge Earthquake girder damage has most commonly been detected at the bottom flanges, although some instances of top flange failure have also been reported. There are several apparent 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 greatly reduce the chance of local buckling of the top flange. The bottom flange, however, being less restrained can experience buckling relatively easily.

There are a number of other factors that could lead to the greater incidence of bottom flange fractures observed in the field. The location of the weld backing is one of the most important of these. At the bottom flange joint, the backing is located at the extreme tension fiber, while at the top flange, it is located at a point of lesser stress and strain demand, both due to the fact that it is located on the inside face of the flange and because the floor slab tends to alter the section properties. Therefore, any notch effects created by the backing are more severe at the bottom flange. Another important factor is that welders can typically make the CJP weld at the girder top flange without obstruction, while the bottom flange weld must be made with the restriction induced by the girder web. Also the welder typically has better and more comfortable access to the top flange joint. Thus, top flange welds tend to be of higher quality, and have fewer stress risers, which can initiate fracture. Finally, studies have shown that UT inspection during construction of the top flange weld is more easily achieved than at the bottom flange, contributing to the better quality likely to occur in top flange welds.

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 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 5-2 - Types of Column Damage

Type	Description
C1	Incipient flange crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside HAZ
C4	Full or partial flange crack in HAZ
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column Splice Failure

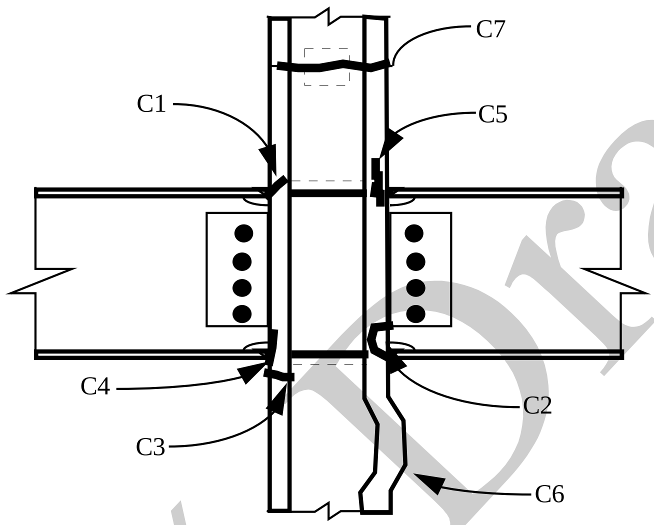


Figure 2-3 - Types of Column Damage

Commentary: Column flange damage includes types C1 through C7. Type C1 damage consists of a small crack within the column flange thickness, typically at the location of adjoining girder flange. C1 damage does not go through the thickness of the column flange and can be detected only by NDT, such as UT. 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 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 stepped shape failure surface within the thickness of the column flange and aligned parallel to it. This damage is often detectable only with the use of NDT.

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

Type C5 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 has been known for a number of years and 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. 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.

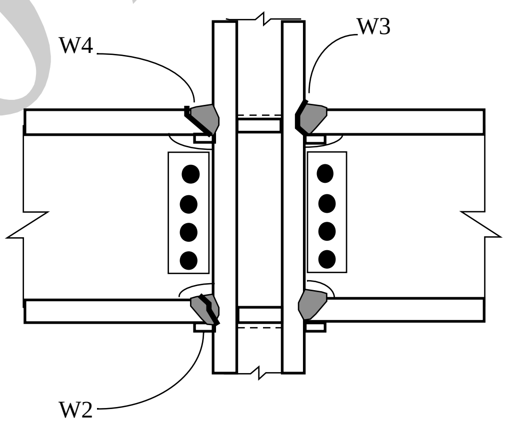
As a result of the potential safety consequences of complete column failure, all column damage should be considered as significant and repaired accordingly.

2.2.3 Weld Damage

Three types of weld discontinuities, defects and damage are defined in Table 2-3 and illustrated in Figure 2-4. All apply to the CJP welds between the girder flanges and the column flanges.

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

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



Note: See Figure 5-6 for related girder damage and Figure 5-7 for related column damage

Figure 2-4 - Types of Weld Damage

Commentary: In addition to the types of damage indicated in Table 2-3 and Figure 2-4, the damage classification system presented in FEMA-267 also included conditions at the root of the complete joint penetration weld that did not propagate through the weld or into the surrounding base metal and could be detected only by removal of the weld backing or through use of 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, which permit small discontinuities in welds. FEMA-267 recognized that W5 conditions were likely to be the result of acceptable flaws introduced during the initial building construction, but included this classification so that it could be reported in the event that it was detected in the course of the ultrasonic testing that FEMA-267 required. There was no requirement to repair such conditions. Since. these guidelines do not require UT as a routine part of the inspection protocol, W5 conditions are unlikely to be detected and have been omitted.

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 the FEMA-267 strongly supports the position that most, if not all W1 damage are pre-existing defects, rather than construction damage. This research also shows that W1 conditions are very difficult to reliably detect, even with the use of UT. In a number of case studies, it has been demonstrated that when W1 conditions are detected 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 guidelines, 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 conditions were W1's.

Although these guidelines 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 MT or 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

Eight types of damage to girder web to column flange shear tabs are defined in Table 2-4 and illustrated in Figure 2-5. Table 5-4 also provides guidance on analytical modeling of girders incorporating this damage. Severe damage to shear tabs is often an indication that other damage has occurred to the connection including column, girder, panel zone, or weld damage.

Table 2-4 - Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S1a	girder flanges sound
S1b	girder flange cracked
S2	Fracture of supplemental weld
S2a	girder flanges sound
S2b	girder flange cracked
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

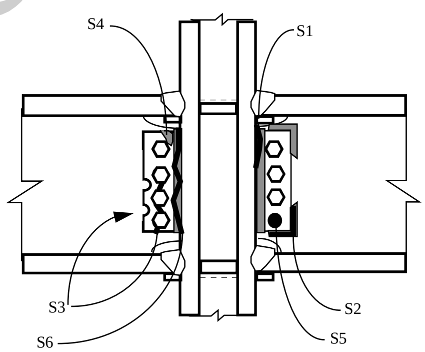


Figure 2-5 - Types of Shear Tab Damage

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, than 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 excessive shear demands do occur, than 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

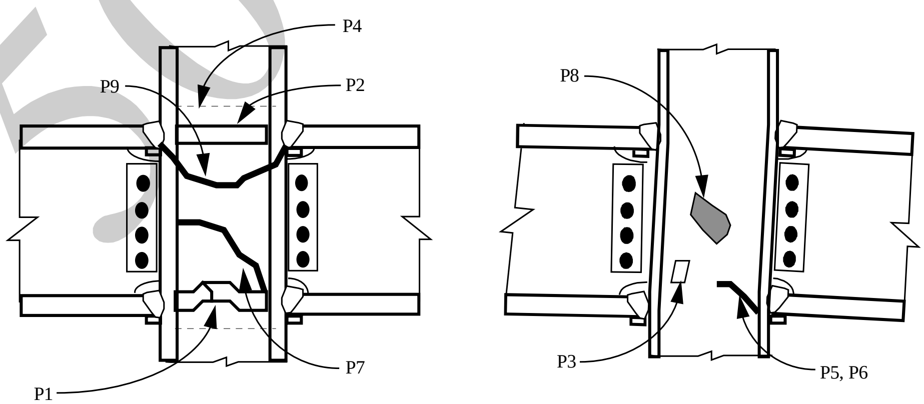


Figure 2-6 - Types of Panel Zone Damage

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 into the girder flange to column flange connection. In recent SAC Phase 1 testing at the University of California at Berkeley, excessive deformation of the column panel zone was identified as a contributing cause to the initiation of type W2 fractures at the top girder flange. It is reasonable to expect that such damage could also be initiated in real buildings, under certain circumstances.

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

Panel zone web buckling (type P8) may result in rapid loss of shear stiffness of the panel zone with potential negative effects as described above. Such buckling is unlikely to occur in connections which 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 WSMF buildings. Other framing elements which may experience damage include column base plates, beams, columns, and their connections that were not intended in the original design to participate in lateral force resistance, and floor and roof diaphragms. In addition, large permanent inter-story drifts may develop in the structures. Based on observations of structures affected by the 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 guidelines.

3. PRELIMINARY POST-EARTHQUAKE EVALUATION

3.1 Introduction

3.1.1 General

The fist step in the evaluation of a welded moment-resisting steel structure following a potentially damaging earthquake is to conduct a rapid preliminary evaluation, or screening, to determine the likelihood of significant structural damage, the implications of this damage with regard to building safety and occupancy and the need for a more detailed evaluation. As indicated in Section 1.3 and Chapter 2, structural damage was detected in many WSMF buildings following the Northridge and other recent earthquakes where there was little outward signs of structural distress. Detailed post-earthquake evaluations involve rigorous inspection of structural condition and analytical assessment of structural integrity. These more detailed evaluations can be quite costly and unnecessary for buildings that have not sustained significant structural damage. Therefore, the initial screening (preliminary evaluation) process is intended to identify those buildings most likely to have sustained significant damage and that should be subject to more detailed evaluations, as well as to determine those buildings in which dangerous conditions may exist and for which immediate restrictions on occupancy should be placed. Based on the findings of the preliminary evaluation a report should be prepared for the owner, and others as appropriate.

Commentary: The intent of the preliminary evaluation is to quickly identify buildings which are likely undamaged and those that are likely damaged to the extent that their pre-earthquake capacity has been significantly impaired. This evaluation is not intended as a means for determining the conformance of the building to code requirements or as a predictor of probable performance of the structure in future earthquakes, including aftershocks. The preliminary evaluation should provide a basis for making recommendations regarding immediate post-earthquake occupancy, the need for additional more detailed evaluations and repairs. Details regarding the nature of more detailed inspections and computation of structural integrity are described in Chapters 4 and 5.

The procedures contained in this Chapter are based in large part on general observations made from the inspection reports of buildings subjected to the Northridge earthquake. Because of the non-specific empirical nature of these evaluations, individual owners may be justified in conducting the more detailed evaluations described in Chapters 4 and 5, whether or not the preliminary evaluation procedure indicates the potential for significant damage. This is especially the case for large, high occupancy structures, buildings incorporating irregular structural features as defined by current building codes and structures expected to achieve higher performance levels. Similarly, owners may wish to *consider evaluation of the performance of the buildings when subjected to future earthquakes. Readers are referred to the companion publication, FEMA-XXX "Evaluation and Upgrade Criteria for Welded Moment-Resisting Steel Frame Construction" for guidelines on performance evaluation and upgrade options for such structures.*

3.1.2 Evaluator Qualifications

Post-earthquake evaluations require the application of considerable engineering knowledge and judgment in order to determine if different conditions within a building are the result of damage and the likely effect of such damage with regard to the ability of the structure to withstand additional loading. In order to perform these tasks properly, the evaluator should posses 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 guidelines as the structural engineer. References to the structural engineer throughout these guidelines 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.

3.1.3 Scope of Preliminary Evaluation

The result of the Preliminary Evaluation is a Post-earthquake Condition Designation. Depending on the designation, additional, more detailed evaluation may or may not be recommended, and guidelines are provided for continuing, limiting or prohibiting occupancy. Screening criteria include ground shaking severity estimates, proximity to other structures known to be damaged, and significant observable damage to the building itself. This chapter provides guidelines for preliminary screening evaluations. Buildings identified by screening as likely to have been damaged should be subjected to detailed evaluations, in accordance with the guidelines of Chapter 4 o5. Following a completion of the preliminary evaluation, a written report documenting the scope and findings of the evaluation should be prepared and presented to the Owner and other appropriate parties.

3.2 Post-earthquake Condition Assessment

Following the performance of a post-earthquake evaluation of a building it will be necessary to inform the Owner and other interested parties of the building condition. The condition ratings presented in Table 3-1 are recommended for this purpose.

Table 3-1 - Post-earthquake Condition Designations

Condition	Finding	Description

Condition	Finding	Description
Green-1	No significant damage	The building does not appear to have experienced significant damage either to structural or non-structural components and provides approximately the same level of safety for occupants that existed prior to the earthquake. Repairs are not required.
Green-2	Minor non-structural damage	The building does not appear to have experienced significant damage to structural elements, but has experienced some damage to non-structural components. It provides approximately the same level of safety for occupants that existed prior to the earthquake. Repairs of nonstructural damage may be conducted at convenience.
Green-3	Minor damage	The building appears to have sustained minor damage to structural and non-structural elements. It provides approximately the same level of safety for occupants that existed prior to the earthquake. Repairs of structural and nonstructural 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 non-structural components and poses a limited safety hazard as a result. Occupancy of the building in areas subject to this damage should be limited until repairs are instituted. Repairs to structural damage may be made at convenience.
Yellow-2	Damaged - structural	The building appears to have experienced significant damage to structural elements that may have impaired its ability to resist additional loading. Although the building does not appear to be an imminent collapse risk, occupancy should be curtailed to essential uses until repairs or stabilization can be implemented, or a more reliable assessment of the building's condition can be made.
Yellow-3	Damaged - extent unknown	The building appears to have sustained significant damage to structural elements and this may have impaired its ability to resist additional loading or to nonstructural elements that pose a significant hazard to occupants. Although the building does not appear to be in imminent collapse risk, occupancy should be curtailed to essential uses until a more detailed evaluation can be performed and the condition of the building ascertained.

Condition	Finding	Description
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 obtained.
Red-2	Unsafe	The building appears to have sustained significant damage to structural that has substantially impaired its ability to resist additional loading or to nonstructural elements that pose a significant hazard to occupants. It appears to be a potential collapse hazard and should not be occupied for any purpose.

Commentary: It is not uncommon during the post-earthquake evaluation process to discover that although a building has relatively little damage, it has severe structural deficiencies 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 the course of a post-earthquake evaluation, the engineer should notify the Owner and Building Official of these conditions.

3.3 Preliminary Evaluation Procedures

A preliminary evaluation includes a general review of the building's construction characteristics to determine its structural system; a visit to the building site to observe its overall condition and note obvious signs of damage; development of estimates of the intensity of ground shaking experienced by the building; and review of the performance of similar buildings in the same vicinity.

The objective of the preliminary evaluations is to determine on a preliminary basis whether or not a building is likely to have sustained significant damage and to develop recommendations for more detailed evaluation, when appropriate. Typically, it will not be possible to make definitive conclusions with regard to a building's damage state on the basis of preliminary evaluations. Accordingly, recommendations for occupancy actions and/or repair should generally be made conservatively at this stage.

3.3.1 Data Collection

In order 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. It is also necessary to conduct a site visit to observe signs of damage.

3.3.1.1 Documents

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 WSMF structure
- Year of design and construction and code used as a basis - this may provide information on particular vulnerabilities, such as the presence of weak stories, or use of particular weld metals
- Identification of materials and typical details of connections and elements for areas of particular vulnerability
- Identification of the location of moment-resisting frames
- Identification 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
- Identification of architectural elements that either affect the behavior of the structural system or elements, or that may be vulnerable to damage and be a threat to occupants including precast concrete cladding systems, interior shaft walls, etc.

3.3.1.2 Preliminary Site Inspection

Every building situated on sites that have experienced strong ground shaking should be subjected to a rapid post-earthquake inspection to ascertain whether there is apparent damage, and to determine if it appears that structural damage may have occurred. Preliminary site inspections should include the following:

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

   ☑ Obvious indications of permanent inter-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 inter-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

- ☑ 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 non-structural components, such as suspended ceilings, light fixtures, ducting, masonry partitions, etc., that could result in potential hazards

   - ☑ Damage to floor slabs and finishes, partitions, etc. that may suggest damage to adjacent beams

   - ☑ Indications of discrete areas of the building where inter-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

   - ☑ Determine if elevator counterweight and rail systems are intact

3. Evaluate building for permanent inter-story drift.

   Preliminary evaluation of the building for permanent inter-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.

4. Perform preliminary visual inspection of selected moment-resisting framing for indications of damage. Refer to Section 3.3.2 for guidelines on preliminary moment-resisting connection inspections.

   - ☑ If visual observation of building exterior or interior indicates zones of permanent inter-story drift, perform selective removal of architectural finishes to expose framing. Observe for indications of yielding, buckling or other damage to framing, or connections.

   - ☑ If visual observation of building exterior or interior indicates zone of concentrated inter-story drift demand, perform selective removal of architectural finishes to expose framing. Observe for indications of fracture, yielding or buckling of framing, or damage to connections.

- ☑ If visual observation of building exterior or interior indicates neither zones of permanent inter-story drift, or of concentrated inter-story drift demand - perform selective removal of architectural finishes to expose framing throughout structure. Observe for indications of yielding or buckling of framing, or damage to connections. Exposures and observation should be made of at least one beam-column connection per line of framing per story. For highly redundant structures, with many lines of framing per story, exposures and observations may be limited to one beam-column connection on one line of framing in each of four quadrants of the structure.

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

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

3.3.1.3 Instrumented Buildings

The structural engineer should 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.

Commentary: 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 non-structural damage in aftershocks.

3.3.2 Preliminary Inspection

Preliminary inspections are performed as part of the process of preliminary evaluation, conducted in accordance with Chapter 2. The specific connections to be inspected as part of a preliminary evaluation shall be determined in accordance with Section 3.3.1. The level of inspection performed as part of preliminary inspection is dependent on whether or not fireproofing is present. Section 3.3.2.1 presents recommendations for preliminary inspection when fireproofing is present. Section 3.3.2.2 presents recommendations for inspection when fireproofing is not present.

3.3.2.1 Fireproofing 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. Note that if there is reason to believe the fireproofing is an asbestos containing material, removal should be performed by appropriately trained personnel with proper personnel protection. The engineer should not personally attempt to remove fireproofing suspected of being an asbestos containing material unless he has been trained in the appropriate hazardous materials handling procedures and is wearing appropriate protective equipment.

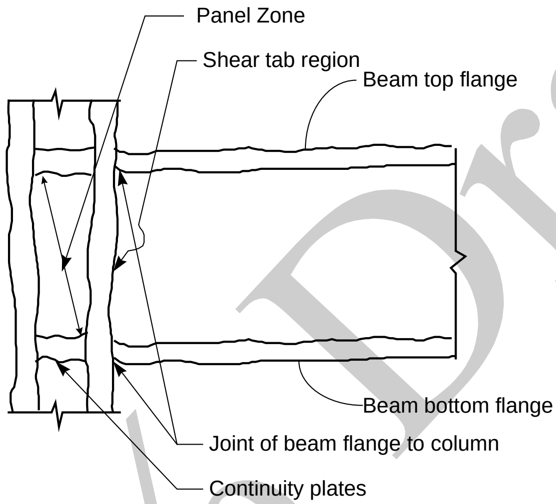


Figure 3-1 Observation Zones for Fire Proofed Beam-Column Connections

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

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

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

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

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

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

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

Commentary: The presence of fireproofing will tend to obscure many types of damage, unless the damage is very severe. However, removal of fireproofing can be a difficult and time consuming process. For the purposes of preliminary inspection in buildings with fireproofing, inspection is limited to that readily observable with the fire proofing in place. Removal of fireproofing and more careful visual inspection in such buildings is limited to inspections performed as part of detailed evaluations, in accordance with Chapters 3 and 4 of these guidelines. An exception is the case when observation indicates that the fireproofing has noticeably cracked, spalled or loosened, indicating that damage has probably occurred to the steel framing beneath. In this case, removal of fireproofing is recommended as part of the preliminary inspection to determine the extent of damage.

In most buildings constructed prior to 1979, the original fireproofing materials commonly contained friable asbestos fibers. Disturbing such material without wearing suitable breathing apparatus can result in a significant health hazard both to the person performing the work and also to others located in the area. For this reason, owners have been gradually addressing these hazards either by encapsulating such fireproofing, to prevent it from being disturbed, or replacing it with non-hazardous materials. In buildings constructed prior to 1979, the engineer should not permit fireproofing to be removed without properly trained personnel using appropriate procedures unless the owner can present suitable evidence that the material does not contain friable asbestos.

3.3.2.2 Bare Steel

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

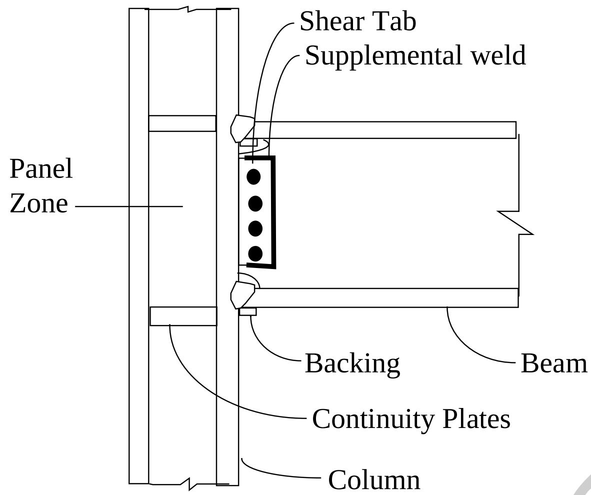


Figure 5-3 Components of Moment Connection

The inspection should be by visible means. Observe all exposed surface for cracks, buckling, yielding, and loosened or broken bolts. The area inspected should include that portion of the beam within a distance d_b of the face of the column, that portion of the column below the connection and within a distance d_c 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 recorded in sketch form.

Note that visual inspection should not be performed casually. After a fracture forms in steel framing, it can close up again under further loading of the building. Such "closed" fractures, though obscure, can typically be detected by careful observation, sometimes aided with touch to detect roughness in the surface in the vicinity of a potential fracture. In some cases, it may be necessary to use methods of NDT, such as Ultrasonic Testing or Liquid Dye Penetrant to confirm the presence of such cracks. Such confirmation can be performed as part of the more detailed inspections undertaken as part of either a level 1 detailed evaluation (Chapter 4) or a level 2 detailed evaluation (Chapter 5).

Certain types of damage (C2, C3, C5, Section 5.3.2; 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 feel gauge 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. NDT will be required to confirm the extent of such damage, and can be performed as part of the more detailed evaluation.

3.3.3 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 the probability that the building has sustained damage, the likely severity of the damage, and the nature of appropriate following actions. The following sections provide minimum recommendations in this regard. The structural engineer, may on the basis of the evaluated data, or his/her own engineering judgment make a more conservative assessment.

3.3.3.1 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 inter-story drift in any level of 0.4% 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 a number of fractures of types: G4, G7, C2, C3, C6, C7, W3, W3, S1, S2, S3, S4, S5, S6, P3, P5, P6, P7 or P9

In the event that any of the above conditions is detected the building should be assessed on a preliminary basis as conforming to damage class 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. Refer to Table 3-1.

3.3.3.2 Finding of Damaged Condition

If none of the conditions indicated in Section 2.3.2.1 are determined to exist, and one or more of the conditions indicated in this Section are present, an assessment should be made that the building has potentially sustained significant damage.

- permanent inter-story drift greater than 0.25% of story height is observed
- significant architectural or structural damage is observed in the building
- one or more columns of the building have noticeably settled relative to adjacent columns
- entry to the building has been limited by the building official because of earthquake damage, regardless of the type or nature of the damage
- the building employs fracture-vulnerable moment-resisting connections and:
  - ⇒ ground motion exceeds the PGA limits indicated in Table 3-2, or;

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

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

⇒ 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 the vertical projection of the rupture area when no surface rupture has occurred.

In the event that any of the conditions indicated above is found to exist the building should be assessed on a preliminary basis as conforming to damage class Yellow-3, of Table 3-1. A detailed evaluation should be recommended and notification should be made advising of potentially significant damage and suggesting caution be exercised with regard to continued occupancy.

Commentary: A building should be considered to have fracture vulnerable moment-resisting connections if detailing of the connections is comprised of direct connection of the beam flanges to the columns with complete joint penetration welds, and the building was constructed prior to 1994. Similarly, beam-column connection details where the welded joints are not capable of developing the full capacity of the connected elements (e.g. welds of beam flanges to columns that employ fillet welds or partial penetration groove welds) should be considered to be fracture-critical.

In the above, the term "significant" has been used without definition or quantification. The intent is to use known damage as an indicator of the severity of ground motion experienced. Damage is dependent not only on the strength of ground motion, but also on the quality and condition of the affected construction. Relatively moderate damage to buildings having regular configuration and adequate lateral-force-resisting systems may be a more significant indicator of strong ground motion than heavy damage to construction in poor condition or having other poor earthquake resisting characteristics. The building official and/or structural engineer should use their own judgment in determining the significance of such damage.

The absence of significant observable damage to WSMF structures on sites believed to have experienced strong ground motion, per Table 3-2, should not be used as an indication that detailed evaluations are not required. Many WSMF buildings that were structurally damaged by the Northridge and Loma Prieta earthquakes had little apparent damage based on casual observation.

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

In many cases in the past, buildings have initially been posted as unsafe without adequate investigation of their condition. Upon reconsideration and technical evaluation, such buildings have subsequently been re-posted to allow occupancy. In such cases, the building need not be considered to have been posted.

Table 3-2 - Ground Motion Indicators of Potential Damage

1997 NEHRP MCE Map* Short Period Contour Area	Estimated Peak Ground Acceleration	Level of Damage to Buildings Within 1 Kilometer
$S_S \geq 0.50$	≥ 0.20g	Prevalent partial collapse of URM buildings. High levels of Non-structural damage. Considerable damage to ordinary buildings.
$0.20 \geq S_S > 0.50$	≥ 0.12g	Considerable damage to URM buildings. Slight damage to well-designed buildings. Prevalent non-structural damage.
$0.12 \leq S_S < 0.20$	≥ 0.08g	Damaged chimneys. Some fallen plaster Limited damage to URM buildings.

* ASCE-7, 1998 and IBC 2000 Maps.

Commentary: A number of techniques are available for estimating the distribution of ground motion in an area following an earthquake. Frequently, the USGS or other government agencies will develop maps of ground motion intensity, shortly after an earthquake occurs. In regions with a large number of strong motion accelerographs present, actual ground motion recordings produce the best method of mapping contours of ground motion. These should be used if located near the building, and are located on sites having similar characteristics. In other regions, empirical techniques, such as the use of standard ground motion attenuation relationships (e.g., Joyner and Boore - 1994; Campbell and Bazorgnia - 1994) may be required. These can be supplemented with analytically derived estimates such as those obtained by direct simulation of the fault rupture and ground wave propagation. It may be desirable to retain a qualified geotechnical engineer or earth science consultant to make these estimates. It should be noted, however, that lacking direct instrumental evidence, site-specific ground motion estimates are, at best, uncertain and subject to wide variations *depending on the assumptions made. Therefore, the best indicator of the severity of ground motion at a site is often the performance of adjacent construction. The criteria of Table 3-2 are provided to help assure that sites which experienced relatively strong ground motion are not overlooked as a result of inaccurate estimates of the ground motion severity.*

3.3.3.3 Finding of Undamaged Condition

If none of the conditions indicated in Sections 3.3.3.1 or 3.3.3.2 are determined to exist, it is recommended that the building be assigned a condition assessment of Green-1, Green-2, or Green-3 of Table 3-1, as appropriate. No further evaluation is recommended.

3.3.4 Reporting and Notification

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

3.3.4.1 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 evaluation has been performed, and indicating the assessed condition designation of the building, recommended occupancy restrictions and follow-up actions. Appendix A to this document includes sample placards.

- If a building has been posted either as "damaged" (condition Yellow-3) or "unsafe" (condition Red-1), additional written notification should be served on the Owner at his/her legal address, indicating the status of the posting, the Owner's rights and any actions required on the Owner's part.

3.3.4.2 Private Consultants

If permitted by the local authority having jurisdiction, 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. Appendix A to this document includes sample placards.

A formal report should be prepared indicating the scope of evaluation that has been performed, the findings of the evaluation, including a description of any damage encountered, the appropriate 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.

4. Level 1 Detailed Post-Earthquake Evaluations

4.1 Introduction

Detailed evaluation is the second step of the post-earthquake evaluation process, for buildings with yellow or red condition designations assigned during the preliminary evaluation. Prior to performing a detailed post-earthquake evaluation, it is recommended that a preliminary evaluation, in accordance with the procedures of Chapter 3, be conducted, to avoid the extensive effort required in a detailed evaluation for those buildings that are unlikely to have been damaged, and also to permit rapid identification of those buildings that may have been so severely damaged that they pose a significant threat to life safety.

Many WSMF 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, then it is necessary to perform an analysis (either by determining a damage index in accordance with the guidelines of this Chapter, or by performing structural analysis in accordance with the guidelines of Chapter 5) to determine the effect of discovered damage on the structure's ability to resist additional loading. This chapter provides simplified guidelines for a detailed evaluation method in which occupancy and repair decisions are made based on the calculation of a damage index, related to the distribution and severity of different types of damage in the structure. An understanding of the distribution of damage in the structure should be obtained by performing visual inspections of critical connections. Although it is preferred that damage indices be calculated based on a determination of the condition of all connections in the building, 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 an alternative series of detailed evaluation guidelines, termed a level 2 evaluation, based on a structural analysis of the damaged structure's ability to resist additional strong ground shaking. In order to perform a level 2 evaluation, is necessary to conduct a complete inspection of all fracture critical connections in the building.

Commentary: The level 1 evaluation approach contained in this chapter is based on the methodology presented in FEMA-267. The level 2 evaluation is a more comprehensive approach that is compatible with the overall approach developed by SAC for performance evaluation of structures.

The level 1 detailed evaluation procedure consists of gathering available information on the structure and a multi-step inspection evaluation, decision and reporting process. Although it is preferable to conduct a complete inspection of all fracture-critical connections, it is permissible to inspect only a 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. This damage index is calculated based on individual connection damage indices, d_i, *assigned to the inspected connections. These connection damage indices vary between 0 and 10 with 0 representing no significant earthquake damage and 10 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, guidance is 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 3.3.1, but extended to include identification of the the primary lateral and gravity load-resisting systems, typical detailing, presence of irregularities, etc. 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 guidelines, and current ground motion estimates, 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 lateral force-resisting frames, and the details of their construction including members sizes, material properties, and connection configurations.

4.3 Evaluation Approach

Analyses of damaged buildings show that although damage occurs at slightly higher frequency in locations predicted to have high stress and deformation demands, damaged connections tend to be widely distributed throughout building frames, often at locations analyses would not predict. This suggests that there is some randomness in the distribution of the damage. To reliably detect all such damage, it is necessary to subject each fracture critical connection to detailed inspections. Fracture critical 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 the end 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 critical connections are recommended as part of a level I evaluation, the 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. Three alternative methods for sample selection are provided. When substantial damage is found within the sample of connections, additional connections are 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 may also be used (level 2 evaluation) as provided for in Chapter 5 of these guidelines.

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 if 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.

Commentary: As used in these guidelines, the term "connection" means that assembly of elements including the beam, clip plates, bolts, welds, etc. that connect a single beam to a single column. Interior columns of frames will typically have two connections (one for each beam framing to the column) at each floor level. Exterior columns of frames will have only one connection at each floor level.

4.4.1 Inspect 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:

Step 1: All moment-resisting connections in the building are subjected to a complete visual inspection in accordance with Section 4.4.1.1, with supplemental nondestructive examination, as suggested by that section.

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

Step 3: Calculate the damage index at each floor 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.

Step 4: Based on the calculated damage indices, determine appropriate occupancy, and structural repair strategies, in accordance with Section 4.4.1.4. 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. Guidelines 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 the fireproofing to allow direct visual observation of the connection area. Detailed inspections may be conducted in stages. An initial stage inspection may be performed by removing only the limited amount of fireproofing indicated in Figure 4-1 and following the inspection checklist of Section 4.4.1.1.1. If such initial inspection indicates the presence or potential presence of damage, than a complete inspection, in accordance with the checklist of Section 4.4.1.1.2 should be performed. To accommodate a complete inspection, removal of fireproofing as indicated in Figure 5-2 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 an initial inspection, per Section 4.4.1.1.1. Refer to Chapter 3 for cautions with regard to removal of fireproofing materials.

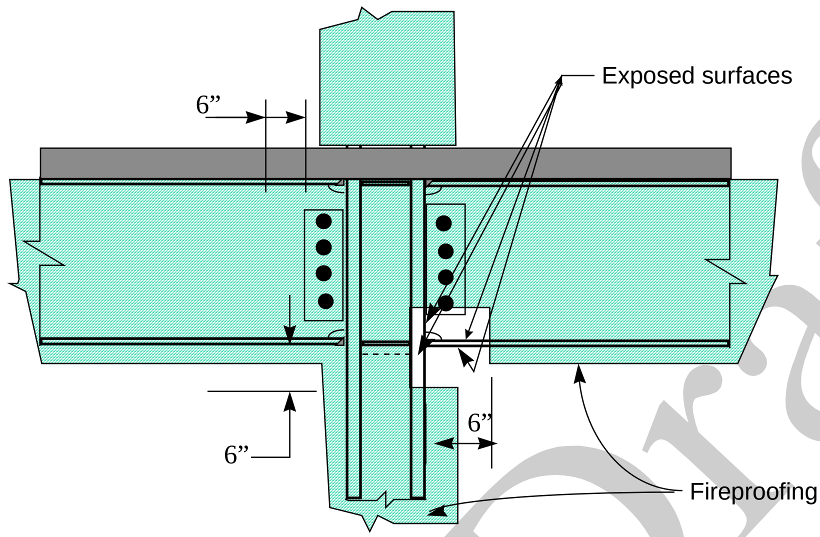


Figure 4-1 Fireproofing Removal for Initial Connection Inspection

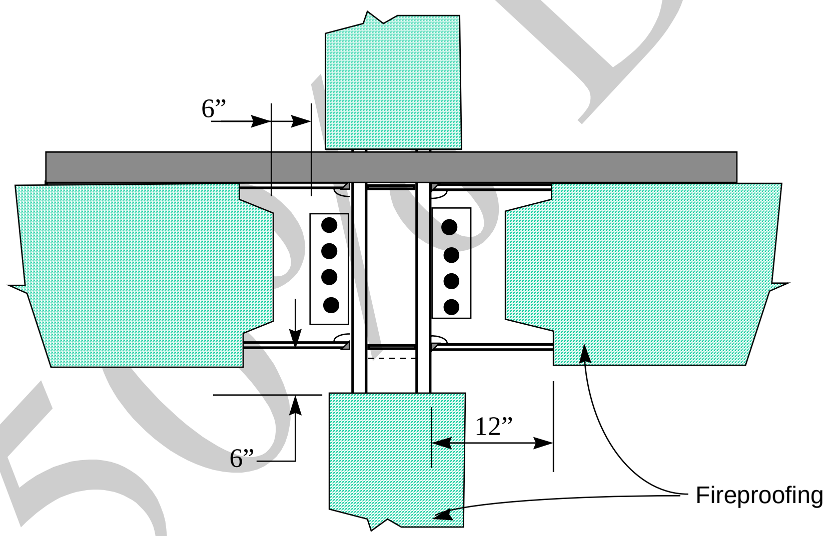


Figure 4-2 Fireproofing Removal for Complete Connection Inspection

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 NDT), the location and type of any observed damage, and if no damage was observed, an indication of this. Appendix A to this Guideline 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 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. This suggests that it may be appropriate to initially inspect only the welded joint of the bottom beam flange to the column, and if damage is found at this location to extend the inspection to the remaining connection components.

4.4.1.1.1 Initial Inspections

Perform the inspections and observations indicated in the checklist below. Prior to performing the inspection, remove fireproofing, as indicated in Figure 5-1 If there are indications of damage, then perform a complete inspection in accordance with the guidelines of Section 4.4.1.1.2.

- ☑ Observe the beam framing into the connection for trueness to line, and potential indications of lateral flexural-torsion buckling (damage type G8, Section 2.2.1).
- ☑ Observe condition of fireproofing along the beam within one beam depth of the column for cracking or spalling of the fireproofing material along the beam surface, indicating potential yielding or buckling of the beam flanges (damage types .G1, G2, Section 2.2.1)
- ☑ Observe the top and bottom surface of the exposed beam bottom flange for fractures (damage types G3, G4, Section 2.2.1).
- ☑ Observe the exposed surfaces of the complete joint penetration weld between the beam bottom flange and column for fractures (damage types W2, W3, W4 Section 2.2.3)
- ☑ Observe the exposed surfaces of the column flange for fractures (damage types C1, C2, C3, Section 2.2.2)
- ☑ Observe the condition at the bottom of weld backing on the bottom flange. If gaps are present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section 2.2.2)
- ☑ Observe the bottom surface of the top flange fireproofing at the locations where the beam flanges join the column flanges (or continuity plates for minor axis connections) for cracks or losses of material that could indicate cracking at the full 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 clip connection from the beam web to the column for loosened, cracked or spalled material indicative of potential damage to shear tabs (damage types S1 through S5, section 2.2.4)

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

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

4.4.1.1.2 Detailed Inspections

Perform the inspections and observations indicated in the checklist below. Prior to performing the inspection, remove fireproofing, as indicated in Figure 5-2. Note that inspection of the top surface of the top flange of the beam and the adjacent column flange will typically be obscured by the diaphragm. If inspections from the exposed bottom surface of the top beam flange indicate a potential for damage to be present, then the diaphragm should be locally removed to allow a more thorough inspection.

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

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

- ☑ Observe the top and bottom surface of the exposed beam bottom flange 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 top and bottom flange. If gaps are present, insert feeler gauge to detect potential damage (damage types C1, C4, C5, Section 2.2.2)

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

- ☑ Observe the column panel zone for cracks, or distortion (damage types P1 through P8, Section 2.2.5)
- ☑ Observe the exposed flanges of the column for distortion (damage type C6, Section 2.2.2)

4.4.1.2 Damage Characterization

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

Table 4-1a - Connection Damage Indices

Type	Location	Description[1]	Index[2] d_j
G1	Girder	Buckled Flange	3
G2	Girder	Yielded Flange	1
G3	Girder	Top or Bottom Flange fracture in HAZ	5
G4	Girder	Top or Bottom Flange fracture outside HAZ	5
G5	Girder	Top and Bottom Flange fracture	10
G6	Girder	Yielding or Buckling of Web	2
G7	Girder	Fracture of Web	10
G8	Girder	Lateral-torsional Buckling	5
C1	Column	Incipient flange crack (detectable by UT)	3
C2	Column	Flange tear-out or divot[5]	8
C3	Column	Full or partial flange crack outside HAZ	8
C4	Column	Full or partial flange crack in HAZ	8
C5	Column	Lamellar flange tearing	5
C6	Column	Buckled Flange	8
C7	Column	Fractured column splice	10
W2	CJP weld	Crack through weld metal exceeding t/4	5
W3	CJP weld	Fracture at girder interface	5
W4	CJP weld	Fracture at column interface	5
S1a	Shear tab	Partial crack at weld to column (beam flanges sound)	3
S1b	Shear tab	Partial crack at weld to column (beam flange cracked)	7
S2a	Shear tab	Crack in Supplemental Weld (beam flanges sound)	1
S2b	Shear tab	Crack in Supplemental Weld (beam flange cracked)	7
S3	Shear tab	Fracture through tab at bolt holes	10
S4	Shear tab	Yielding or buckling of tab	3
S5	Shear tab	Damaged, or missing bolts[4]	6
S6	Shear tab	Full length fracture of weld to column	10

P1	Panel Zone	Fracture, buckle, or yield of continuity plate[3]	3
P2	Panel Zone	Fracture of continuity plate welds[3]	3
P3	Panel Zone	Yielding or ductile deformation of web[3]	1
P4	Panel Zone	Fracture of doubler plate welds[3]	3
P5	Panel Zone	Partial depth fracture in doubler plate[3]	3
P6	Panel Zone	Partial depth fracture in web[3]	7
P7	Panel Zone	Full (or near full) depth fracture in web or doubler plate[3]	10
P8	Panel Zone	Web buckling[3]	5
P9	Panel Zone	Fully severed column	10

Notes To Table 3-2a:

1. See Figures 4-3 through 4-7 for illustrations of these types of damage.
2. Where multiple damage types have occurred in a single connection, then:
   a. Sum the damage indices for all types of damage with d=1 and treat as one type. If multiple types still exist; then:
   b. For two types of damage refer to Table 3-2b. If the combination is not present in Table 3-2b and the damage indices for both types are greater than or equal to 4, use 10 as the damage index for the connection. If one is less than 4, use the greater value as the damage index for the connection.
   c. If three or more types of damage apply and at least one is greater than 4, use an index value of 10, otherwise use the greatest of the applicable individual indices.
3. 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.
4. 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.
5. 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[1]

Girder, Column or Weld Damage	Shear Tab Damage	Damage Index	Girder, Column or Weld Damage	Shear Tab Damage	Damage Index
G3 or G4	S1a	8	C5	S1a	6
	S1b	10		S1b	10
	S2a	7		S2a	6
	S2b	7		S2b	10
	S3	10		S3	10
	S4	6		S4	10
	S5	10		S5	10
	S6	10		S6	10
C2	S1a	8	W2, W3, or W4	S1a	8
	S1b	8		S1b	10
	S2a	8		S2a	8
	S2b	8		S2b	10
	S3	10		S3	10

	S4	8		S4	10
	S5	8		S5	10
	S6	10		S6	10
C3 or C4	S1a	8			
	S1b	10			
	S2a	8			
	S2b	10			
	S3	10			
	S4	10			
	S5	10			
	S6	10			

1. See Table 3-2a, footnote 2 for combinations other than those contained in this table.

More complete descriptions (including sketches) of the various types of defects and damage are provided in Chapter 2. When the engineer can show by rational analysis that other values for the relative severities of damage are appropriate, these may be substituted for the damage indices provided in the tables. A full reporting of the basis for these different values should be provided to the building official, upon request.

Commentary: The connection damage indices provided in Table 4-1 (ranging from 0 to 10) represent judgmental estimates of the relative severities of the various types of damage. Damage severity is judged in two basic respects, the impact of the damage on global stability and lateral resistance of the frame 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 10 indicates very severe damage.

When initially developed, these connection damage indices were conceptualized as estimates of the connection's lost capacity to reliably participate in the building's lateral-force-resisting system in future earthquakes (with 0 indicating no loss of capacity and 10 indicating complete loss of capacity). However, due to the limited data available, no direct correlation between these damage indices and the actual residual strength and stiffness of a damaged connection was ever made. They do provide a convenient measure, however, of the extent of damage that various connections in a building have experienced.

Analyses conducted by SAC 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) consistently throughout a moment-resisting frame results in only a modest increase in the vulnerability of a structure to developing P-delta instability and collapse. However, if a number of connections develop fractures at both flanges of *the beam-column connection, significant increase in vulnerability occurs. As a result of this, damage that results in the loss of effectiveness of a single flange joint to transfer flexural tension stress is assigned a relatively modest damage index of 5, 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 10, 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.*

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

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

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

$$D_i = \frac{1}{n}\sum_{j=1}^{n}\frac{d_j}{10} \tag{4-1}$$

where: D_i is the 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, per Tables 4-1a and 4-1b for the j^{th} connection in the group at that floor

4.4.1.4 Determine Maximum Floor Damage Index

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

4.4.1.5 Determine Recommended Recovery Strategies for the Building

Recommended post-earthquake recovery strategies are as indicated in Table 4-2, based on the maximum damage index, D_{max}, determined in the previous steps.

Table 4-2 - Recommended Repair and Modification Strategies

Observation[6]	Recommended Strategy (Cumulative)	Note
$D_{max}>0$	Repair all connections discovered to have $d_j \geq 5$	1,2
$D_{max} > 0.1$	Repair all connections discovered to have $d_j \geq 2$	1,2
$D_{max}> 0.5$	A potentially unsafe condition should be deemed to exist unless a level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j > 1$.	3

Notes to Table 3-4:

1. Includes damage discovered either as part of Step 2 or Step 3.
2. Although repair is recommended only for the more seriously damaged connections, the repair of all connections that are damaged or otherwise deficient should be considered.
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 1/2, or
   b. level 2 analyses indicate that a dangerous condition does not exist, or
   c. recommended repairs are completed for all connections having $d_j > 3$.

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. 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 result in substantial reduction in the vulnerability of the structure to damage from future earthquakes, while upgrade has the potential to greatly reduce future damage and losses.

A companion document to this publication, FEMA-XXX, Evaluation and Upgrade Criteria for Existing Moment-Resisting Steel Frame Construction provides guidelines for assessing the probable performance *of steel frame buildings and for designing upgrades to improve this performance.*

4.4.2 Inspect a Sample of Connections

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

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

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

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

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

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

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

Step 6: Given the average damage index for connections in the group, determine the probability, P, that the connection damage index for any group, at a floor level, exceeds 1/2, and determine the maximum estimated damage index for any floor, D_{max}.

Step 7: Based on the calculated damage indices and statistics, determine appropriate occupancy, structural repair and modification strategies. If deemed appropriate, the structural engineer may conduct detailed structural analyses of the building in the as-damaged state, to obtain improved understanding of its residual condition and to confirm that the recommended strategies are appropriate or to suggest alternative strategies. Guidelines 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 at a premium. Prudent owners may want to consider having an investigation plan already developed (Steps 1 and 2) before an earthquake occurs, and to have an agreement with appropriate structural engineering and inspection professionals and organizations to give priority to inspecting their buildings rapidly following the occurrence of an earthquake.

4.4.2.1 Evaluation Step 1 - Categorize Connections by Groups

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

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

For buildings of two or more stories, the roof connections may be excluded from the initial inspection process. However, when these guidelines recommend inspection of all connections within a group or building, the roof connections should be inspected.

4.4.2.2 Step 2--Select Samples of Connections for Inspection

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

For each group of connections, select a representative sample for inspection in accordance with any of Methods A, B, or C, below. If the evaluation is being performed to satisfy a requirement imposed by the building official, a letter indicating the composition of the groups, and the specific connections to be inspected should be submitted to the building official prior to the initiation of inspection. The owner or structural engineer may at any time in the investigation process elect to investigate more connections than required by the selected method. However, the additional connections inspected may not be included in the calculation of damage statistics under Step 4

(Section 4.4.2.4) unless they are selected in adherence to the rules laid out for the original sample selection, given below.

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

4.4.2.2.1 Method A - Random Selection

Connections are selected for inspection such that a statistically adequate random sample is obtained. The minimum number of connections to be inspected for each group is determined in accordance with Table 3-3. The following limitations apply to the selection of specific connections:

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

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

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

     - re-entrant corners
     - set-backs
     - soft or weak stories
     - torsional irregularities (connections at perimeter columns)
     - diaphragm discontinuities

   - Connections incorporating the largest size framing elements.

2. The balance of the sample should be selected randomly from the remaining connections in the group.

Up to 10% of the connections in the sample may be replaced by other connections in the group to which access may more conveniently be made.

Table 3-3 - 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	2	200	27
10	3	300	37
15	4	400	45
20	5	500	53
30	7	750	72
40	8	1000	99
50	10	1250	104
75	13	1500	120
100	17	2000	147

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 sample sizes contained in Table 3-3 were developed from MIL-STD-105D, a well established quality control approach that has been widely adopted by industry.

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

It is recognized that in many cases the structural engineer may wish to pre-select those connections believed to be particularly vulnerable. However, unless these pre-selected connections are fairly well geometrically distributed, a number that is more than about 20% 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 20% should be pre-selected on this basis, the alternative approach of Method C should be used.*

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

There are a number of methods available for determining the randomly selected portion of the sample. To do this, each connection in the group (excluding pre-selected connections) should be assigned a consecutive integer identifier. The sample may then be selected with the use of computer spread sheet programs - many of which have a routine for generation of random integers between specified limits, published lists of random numbers, or by drawing of lots.

4.4.2.2.2 Method B - Deterministic Selection

Connections are selected to satisfy the following criteria:

1. At least one connection is selected on every column face of every line of moment-resisting framing in the group;
2. At least one connection is selected on every floor from every frame;
3. No more than 50% of the connections in a sample may be selected from any floor or column face than would be done if the number of inspected connections was equally apportioned among either the column faces or floors; and

Up to 10% of the connections in the sample may be replaced by other connections in the same frame and group to which access may more conveniently be made.

Commentary: It is recognized that in many cases the structural engineer may be reluctant to select connections in a random manner, as provided by Method A. For those cases, Method B is acceptable since it assures that every floor and every column is inspected at least once. The structural engineer may select any combination of connections to be inspected that meets these criteria; notwithstanding, care should be exercised to assure that these allowances are not used to subvert the intent of the inspection process to determine the degree of damage to the building, if any.

4.4.2.2.3 Method C - Analytical Selection

Connections are selected for inspection in accordance with the following criteria:

1. The minimum number of connections within the group to be inspected is as indicated in Table 3-3.

2. Up to 60% of the connections may be selected based on the results of rational analysis indicating those connections most likely to be damaged.

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

Prior to initiation of the inspections, the rational analysis and list of connections to be inspected should be subjected to a qualified independent third party review in accordance with Section 4.6. The peer review should consider the basis for the analysis, consistency of the assumptions employed, and assure that overall, the resulting list of connections to be inspected provides an appropriate sampling of the building's connections.

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

For buildings designed and constructed following the 1994 Northridge earthquake, and conforming to the recommendations contained in Chapter 7 of FEMA-267, or conforming to the design recommendations for Special Moment Resisting Frames contained in the 1997 or later edition of the *NEHRP Provisions*, the scope of inspection may be reduced to 1/2 the number of connections recommended in the following sections. If in the course of this reduced scope of inspection, significant structural damage is found (damage to any connection with a damage index per Table 4-1(a or b) that is greater than 5), then full inspections in accordance with the following sections should be performed.

Commentary: In analyses conducted of damaged buildings, there has been a generally poor correlation of the locations of damage and the locations of highest demand predicted by analysis. This is primarily attributed to the fact that the propensity for a fracture to initiate in a connection is closely related to the workmanship present in the welded joints, which tends to be a randomly distributed quantity. Moreover, typical analysis methods do not capture the complex nonlinear stress state that occurs in actual connections. However, there has been some correlation. Analysis is a powerful tool to assist the structural engineer in understanding the expected behavior of a structure, damaged or undamaged. The specific analysis procedure used should be tailored to the individual characteristics of the building. It should include consideration of all building elements that are expected to participate in the building's seismic response, including, if appropriate, elements not considered to be part of the lateral-force-resisting system. The ground motion characteristics used for the analysis should not be less than that required by the building code for new construction, and to the extent practical, should contain the spectral characteristics of the actual ground motion experienced at the site. Qualified independent review is recommended to assure that there is careful consideration of the basis for the selection of the connections to be inspected and that a representative sample is obtained.

4.4.2.3 Step 3--Inspect the Selected Samples of Connections

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

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

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 5.3.2) can only be detected by removal of the backing, or by nondestructive examination. 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, consideration should be given to the use of ultrasonic testing (UT) to determine if hidden fractures are present.

The practice of inspecting a small sample of the total connections present in a building, in order to infer the probable overall condition of the structure is consistent with that followed by most engineers in the Los Angeles area, following the Northridge Earthquake. However, the typical practice following that event included the extensive use of UT in addition to visual inspection. This UT revealed a number of apparent conditions of damage at the roots of the full penetration welds between beam and column flanges. These conditions, which were quite widespread were typically reported by testing agencies and engineers as damage. This practice was encouraged by the FEMA-267 guidelines which classified 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 that indicated by the 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 was found to be significantly different than 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 Northridge earthquake were not damage at all but rather, 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 guidelines have de-emphasized, relative to the recommendations of FEMA-267, the importance of employing NDT in the post-earthquake inspection process.*

Inspections may be terminated when at least 50% of the connections selected for each sample have been inspected if:

1) the inspections have progressed in a manner that retains an adequately random nature and distributed geometry for those connections that are inspected (a distribution throughout the building that is acceptable to the building official); and

2) no connections with damage indices $d_j > 5$ (Table 3-2a or b) are discovered; and,

3) not more than 10% of the total connections inspected are discovered to have $d_j \geq 2$.

If all of these conditions are not met, then inspections should be completed for all connections contained in all samples.

4.4.2.4 Damage Characterization

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

Commentary: The connection damage indices provided in Table 4-1 (ranging from 0 to 10) represent judgmental estimates of the relative severities of the various types of damage. Damage severity is judged in two basic respects, the impact of the damage on global stability and lateral resistance of the frame 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 10 indicates very severe damage.

When initially developed, these connection damage indices were conceptualized as estimates of the connection's lost capacity to reliably *participate in the building's lateral-force-resisting system in future earthquakes (with 0 indicating no loss of capacity and 10 indicating complete loss of capacity). However, due to the limited data available, no direct correlation between these damage indices and the actual residual strength and stiffness of a damaged connection was ever made. They do provide a convenient measure, however, of the extent of damage that various connections in a building have experienced.*

Analyses conducted by SAC 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) consistently throughout a moment-resisting frame results in only a modest increase in the vulnerability of a structure to developing P-delta instability and collapse. However, if a number of connections develop fractures at both flanges of the beam-column connection, significant increase in vulnerability occurs. As a result of this, damage that results in the loss of effectiveness of a single flange joint to transfer flexural tension stress is assigned a relatively modest damage index of 5, 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 10, as are types of damage that could potentially result in impairment of a column or beam's ability to continue to carry gravity loads, aftershocks or other future events. 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.

4.4.2.5 Step 4--Inspect Connections Adjacent to Damaged Connections

Perform additional inspections of moment-resisting connections near connections with significant damage as follows:

1) when a connection is determined to have a damage index $d_j \geq 5$, inspect all moment-resisting connections immediately adjacent (above and below, to the left and right) to the damaged connection in the same moment frame (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.

2) when a connection is determined to have a damage index dj > 9, inspect the two moment-resisting connections immediately adjacent (above and below, to the left and right) to the damaged connection in the same moment frame (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.

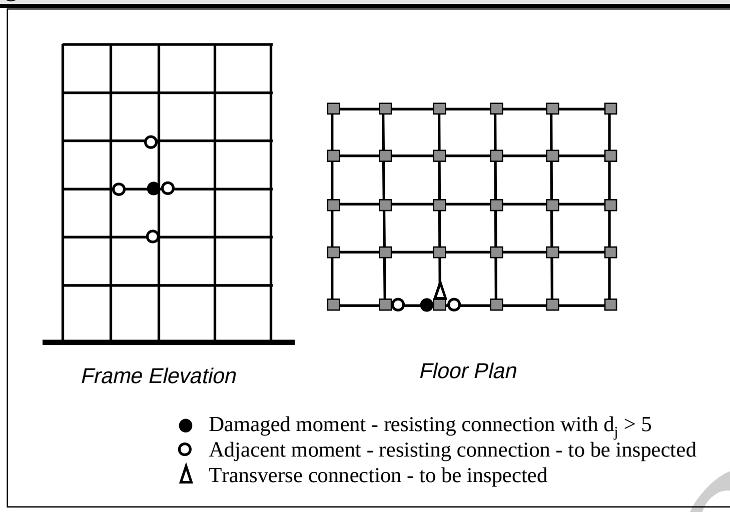


Figure 4-3 - Inspection of Connections Adjacent to Damaged Connection ($d_j \geq 5$)

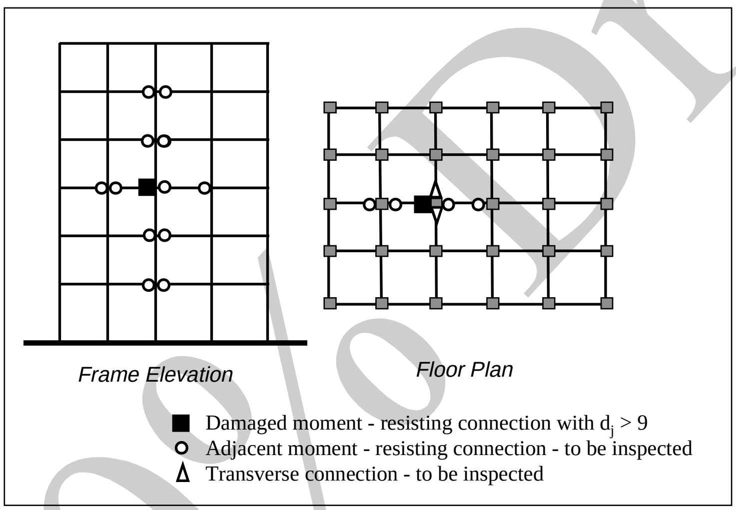


Figure 4-4 - Inspection of Connections Adjacent to Damaged Connection ($d_j \geq 9$)

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 ($dj \geq 5$), then inspect the connections near these additional connections, as indicated in 1) and 2) above. Continue this process, until one of the following conditions occurs:

a) The additional connection inspections do not themselves trigger more inspections, or

b) 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.

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

4.4.2.6 Step 5--Determine Damage Statistics for Each Group

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

$$d_{avg} = \frac{1}{n}\sum_{j=1}^{n}\frac{d_j}{10} \qquad (4\text{-}2)$$

$$\sigma = \sqrt{\frac{1}{n-1}\sum_{j=1}^{n}\left(\frac{d_j}{10} - d_{avg}\right)^2} \qquad (4\text{-}3)$$

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

d_j is the damage index, per Tables 4-1a and 4-1b for the j^{th} inspected connection in the sample

The additional connections selected using the procedure of Section 4.4.2.5 (Step 4) are not included in the above calculation.

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

In this procedure, the maximum damage index at a floor D_{max} is estimated based on the damage indices determined for the connections actually inspected, and the probability P that D_{max} exceeds a value of 1/2 is determined.

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

$$D = d_{avg} \qquad (4\text{-}4)$$

$$S = \frac{\sigma}{\sqrt{k}} \qquad (4\text{-}5)$$

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

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

$$b = \left(\tfrac{1}{2} - D\right)\Big/ S \qquad (4\text{-}6)$$

Using the value of "b" calculated from equation 4-6, determine P_f, from Table 4-4. P_f is the probability that if all connections had been inspected, the cumulative damage index at any floor would have been found to exceed 1/2. If the probability P_f is high, this strongly suggests the possibility that there has been a significant reduction in seismic resisting capacity.

Then determine the probability P that if all connections within the group had been inspected, the connections within the group on at least one floor (out of q total floors in the group) would have been found to have a cumulative damage index of 1/2 or more from the equation:

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

Table 4-4 - P_f as a function of parameter "b"

"b"	P_f - (%)	"b"	P_f - (%)
-1.2816	90	1.2265	11
-0.8416	80	1.2816	10
-0.5244	70	1.3408	9
-0.2533	60	1.4051	8
0.0000	50	1.4395	7.5
0.2533	40	1.4758	7
0.5244	30	1.5548	6
0.8416	20	1.6449	5
0.8779	19	1.7507	4
0.9154	18	1.8808	3
0.9542	17	1.9600	2.5
0.9945	16	2.0537	2
1.0364	15	2.1701	1.5
1.0803	14	2.3263	1
1.1264	13	3.0962	.1
1.1750	12	3.7190	.01

* Note - Intermediate values of P_f may be determined by linear interpolation

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

$$D_i = \frac{(k_i - m_i)d_{avg}}{k_i} + \left(\frac{1}{k_i}\right)\sum_{j=1}^{m_i}\frac{d_j}{10} \qquad (4\text{-}8)$$

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.

Commentary: The criterion for damage evaluation used in this Guideline is to assume that a cumulative damage index of 1/2 marks the threshold at which a structure may become dangerous. Such a damage index could correspond to cases where 1/2 of the connections in a building have been severely damaged; cases where all of the connections 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, by the Central Limit Theorem, 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. Therefore, the probability that a damage index of 1/2 has been exceeded at a floor, in a group with k connections may be approximated by determining how many multiples (b) times the standard deviation (S), when added to the mean damage index (D) equals 1/2. Or, in equation form :

$$D + bS = 1/2 \qquad (4\text{-}9)$$

Solution of this equation for the multiplier "b" results in the required relationship of equation 4-9.

Damage Indices (from Table 4-1) that are largely judgmental are used to characterize the loss of reliable seismic performance capability of individual connections. These indices are added, averaged and otherwise statistically manipulated for use as an indication of the average damage index for groups of connections, entire frames and ultimately of the lateral system itself. It should be clear that use of such an approximate, judgmentally defined characterization of strength cannot rigorously calibrate the loss of lateral resistance, or the residual strength and stiffness of the building.

In spite of the somewhat arbitrary nature of the 1/2 damage index criterion and the judgmental nature of the suggested way of testing whether that criteria has been exceeded, it is believed that the results of *these procedures will lead to reasonable conclusions in most cases. However, it is always the prerogative of the responsible structural engineer to apply other rational techniques, such as direct analyses of the remaining structural strength, stiffness, and deformation capacity as a verification of the conclusions provided by these procedures. Particularly in anomalous or marginal cases, such additional checks based on engineering judgment are strongly encouraged .*

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

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

Table 4-5 - Recommended Condition Designation and Repair Strategies

Observation[6]	Condition Designation	Recommended Strategy (Cumulative)	Note
$P>0$ or $D_{max}>0$	Green - 3	Repair all connections discovered to have $d_j \geq 5$	1,2
$P > 5\%$ or $D_{max} > 0.1$	Green - 3	Repair all connections discovered to have $d_j \geq 2$	1,2
$P > 10\%$ or $D_{max} > 0.2$	Green - 3	Inspect all connections in the group. Repair all connections with $d_j \geq 2$	2
$P > 25\%$ or $D_{max} > 0.5$	Red - 2	A potentially unsafe condition should be deemed to exist unless a level 2 evaluation is performed and indicates that acceptable confidence is provided with regard to the lateral stability of the structure. Notify the building owner of the potentially unsafe condition. Inspect all connections in the building. Repair all connections with $d_j > 1$.	3

Notes to Table 3-4:

1. Includes damage discovered either as part of Step 2 or Step 3.
2. Although repair is recommended only for the more seriously damaged connections, the repair of all connections that are damaged or otherwise deficient should be considered.
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 1/2, or
   b. level 2 analyses indicate that a dangerous condition does not exist, or
   c. recommended repairs are completed for all connections having $d_j > 3$.

Commentary: The value of P (the probability that the connections on at least one floor have a cumulative damage index of 1/2 or more) and D_{max} (the maximum damage index at a floor level within a group) were determined in Method A by using a random selection process, and thus represent a statistically valid basis for the characterization of the damage index for the group of connections, and thus for the building. Method B selects the connections by using a specified distribution throughout the building based on forcing selection of connections in every column line and floor. Method C selects the connections, based on engineering characterization of those most likely to have been damaged, modified to reflect a distribution throughout the structure. While the connections selected by Methods B and C are not truly random, they are widely distributed and have some characteristics of a random distribution. Such selections are judged to be sufficiently random-like to warrant processing as if the connections were selected randomly. Thus regardless of whether *method A, B, or C was used, decisions on disposition of the building, and the need for repair measures can defensibly be based on the values of these two key parameters, as determined for each group of connections.*

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. 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 result in substantial reduction in the vulnerability of the structure to damage from future earthquakes, while upgrade has the potential to greatly reduce future damage and losses. Upgrade should be given especially strong consideration for those structures that have experienced substantive damage, as evidenced by high calculated D_j*'s for relatively moderate levels of ground shaking.*

A companion document to this publication, FEMA-XXX, Evaluation and Upgrade Criteria for Existing Moment-Resisting Steel Frame Construction provides guidelines for assessing the probable performance of steel frame buildings and for designing upgrades to improve this performance.

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 4.4.2.8, 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 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. This report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following material:

- Building Address
- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, 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 (SMAW, FCAW, etc.) 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 documentation of all instructions to the inspectors, and of 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 3-2a, photographs should be included for all connections with damage index $d_i \geq 5$.
- 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 actions (repair and modification measures and 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 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 remediatoin 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.

Commentary: Following completion of the detailed damage assessments, the structural engineer should prepare a written report. The report should include identification of any potentially hazardous conditions which were observed, including 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, visual inspection and NDT records, data sheets, and reports as attachments.

The nature and scope of the evaluations performed should be clearly stated. 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.

4.6 Qualified Independent Engineering Review

Independent third party review, by qualified professionals, is recommended throughout these Guidelines when alternative approaches to evaluation or design are taken, or where approaches requiring high degrees of structural engineering knowledge and judgment are taken. Specifically, it is recommended that qualified engineering review be provided in any of the following cases:

- Where an evaluation is being performed and the engineer elects to select connections for inspection by a method other than those provided in these Guidelines.
- Where the calculated damage index D_{max} exceeds 50% and the engineer has determined that an unsafe condition does not exist.
- Where an engineer has decided not to repair damage otherwise recommended to be repaired by these Guidelines.

- When any story of the building has experienced a permanent lateral drift exceeding 0.7% of the story height and proposed repairs do not correct this condition.

Where independent review is recommended, the analysis and/or design should be subjected to an independent and objective technical review by a knowledgeable reviewer experienced in the design, analysis, and structural performance issues involved. The reviewer should examine the available information on the condition of the building, the basic engineering and reliability concepts, and the recommendations for proposed action.

Commentary: The independent reviewer may be one or more persons whose collective experience spans the technical issues anticipated in the work. When more than one person is collectively performing the independent review, one of these should be designated the review chair and should act on behalf of the team in presenting conclusions or recommendations.

Independent third party review is not a substitute for plan checking. It is intended to provide the structural engineer of record with an independent opinion, by a qualified expert, on the adequacy of structural engineering decisions and approaches. The seismic behavior of WSMF structures is now understood to be an extremely complex issue. Proper understanding of the problem requires knowledge of structural mechanics, metallurgy, welding, fracture mechanics, earthquake engineering, and statistics. Due to our limited current state of knowledge, even professionals who possess such knowledge face considerable uncertainty in making design judgments. Third party review should only be performed by qualified individuals.

4.6.1 Timing of Independent Review

The independent reviewer(s) should be selected prior to the initiation of substantial portions of the design and/or analysis work that is to be reviewed, and coordination of the review should start as soon as sufficient information to define the project is available.

4.6.2 Qualifications and Terms of Employment

The reviewer should have no other involvement in the project before, during, or after the review. The reviewer should be selected and paid by the owner and should have an equal or higher level of technical expertise in the issues involved than the structural engineer-of-record. The reviewer (or in the case of peer review teams, the review chair) should be a structural engineer who is familiar with governing regulations for the work being reviewed. The reviewer should serve through completion of the project and should not be terminated except for failure to perform the duties specified herein. Such termination should be in writing with copies delivered to the building official, owner, and structural engineer-of-record.

4.6.3 Scope of Review

Review activities related to evaluation of the safety condition of a building should include a review of available construction documents for the building, all inspection and testing reports, any analyses prepared by the structural engineer of record, the method of connection sample selection, and visual observation of the condition of the structure, as well as review of any mathematical models and analyses performed as part of the post-earthquake evaluation. Review should include consideration of the proposed design approach, methods, materials, and details.

4.6.4 Reports

The reviewer should prepare a written report to the owner and building official that covers all aspects of the structural engineering review performed, including conclusions reached by the reviewer. Such reports should include statements on the following:

- Scope of engineering review performed with limitations defined.
- The status of the project documents at each review stage.
- Ability of selected materials and framing systems to meet performance criteria with given loads and configuration.
- Degree of structural system redundancy, ductility, and compatibility, particularly in relation to lateral forces.
- Basic constructability of structural members and connections (or repairs and modifications of these elements).
- Other recommendations that would be appropriate to the specific project.
- Presentation of the conclusions of the reviewer identifying any areas which need further review, investigation, and/or clarifications.

4.6.5 Responses and Corrective Actions

The structural engineer-of-record should review the report from the reviewer and develop corrective actions and other responses as appropriate. Changes during the construction/field phases that affect the seismic resistance system should be reported to the reviewer in writing for action and recommendations.

4.6.6 Distribution of Reports

All reports, responses, and corrective actions prepared pursuant to this section should be submitted to the building official and the owner along with other plans, specifications, and calculations required. If the reviewer is terminated by the owner prior to completion of the project, then all reports prepared by the reviewer, prior to such termination, should be submitted to the building official, the owner, and the structural engineer-of-record within ten (10) working days of such termination.

4.6.7 Engineer-of-Record

The structural engineer-of-record should retain the full responsibility for the structural design as outlined in professional practice laws and regulations. The independent review engineer(s) should not be asked to or be expected to assume any responsibility for the structural evaluation or subsequent repair designs.

4.6.8 Resolution of Differences

If the structural engineer-of-record does not agree with the recommendations of the reviewer, then such differences should be resolved by the building official in the manner specified in the applicable Building Code.

5. Level 2 Detailed Post-Earthquake Evaluations

5.1 Introduction

Detailed evaluation is the second step of the post-earthquake evaluation process. Prior to performing a detailed post-earthquake evaluation, it is recommended that a preliminary evaluation, in accordance with the Guidelines of Chapter 3, be conducted, to avoid the extensive effort required in a detailed evaluation for those buildings that are unlikely to have been damaged, and also to permit rapid identification of those buildings that have been so severely damaged that they pose a significant threat to life safety.

Many WSMF buildings damaged in past earthquakes have displayed few outward signs of this 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. It may also be necessary to perform an analysis to determine the effect of discovered damage on the structure's ability to resist additional loading. Chapter 4 provides a series of guidelines 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 critical connections. Although it is preferred that all fracture critical connections be inspected, the procedures of Chapter 4 permit inspections to be limited to a representative sample. This chapter provides guidelines for a detailed evaluation processes in which a detailed analytical evaluation of the damaged structure's ability to resist additional strong ground shaking is conducted. In order to perform such an analysis it is necessary to inspect all fracture critical connections in the building in order to understand their condition.

> *Commentary: The level 1 evaluation approach of Chapter 4 is based on the methodology presented in FEMA-267. The level 2 evaluation approach described in this Chapter is a more comprehensive analytical approach that is compatible methodology developed by SAC for design and performance evaluation of WSMF structures.*

5.2 Data Collection

Prior to performing a detailed evaluation, the original construction drawings should be reviewed (if available) to identify the primary lateral and gravity load-resisting systems, typical detailing, presence of irregularities, etc. Pertinent available engineering and geotechnical reports, including any previous damage survey reports and current ground motion estimates, 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 members sizes, material properties, and connection configurations.

> *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 residual stiffness and strength in WSMF structures and the analytical procedures of this Chapter include direct consideration of such framing. Data collection should obtain sufficient information on this framing, as well as that intended to provide the structure's lateral-force resistance to permit an accurate analytical model of the structure to be developed.*

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

- 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 end 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.

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

> *Commentary: Most welded, moment-resisting beam-column connections constructed prior to 1994 will be of the fracture critical 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 and Alaska. However, fracture critical connections may exist in some post-1994 buildings,*

particularly those constructed in zones of lower seisimicity.

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. An analysis of the response of this model to a repeat of the initial earthquake event that caused the damage is then conducted to determine a level of confidence that the building is capable of resisting this ground motion without collapse. If the analysis indicates sufficient confidence that the building would not collapse in a repeat of the initial earthquake event, building occupancy may continue as the building is repaired. If sufficient confidence of the building's ability to resist the ground motion without collapse is not indicated, then occupancy should be limited or prevented during the repair period.

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

- *comparison of the building's residual lateral-force-resisting capacity with that specified by the building code for design of structures*
- *comparison of the building's residual lateral-force resisting capacity with that which existed prior to the onset of damage*
- *application of the engineer's judgment as to the extent which the building poses an imminent or extreme hazard*

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

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

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

In both of the above analytical approaches, an attempt is generally made to estimate the adequacy of the building to withstand collapse given that ground motion comparable to that used as a basis for design in the building code is experienced. However, building codes are based on hazards that have very long return periods. Such levels of ground shaking are unlikely to occur in the relatively brief period of time, perhaps a year following a damaging earthquake event, during which a building is either repaired or demolished, and therefore may be excessively conservative with regard to determining building safety.

Review of statistics of past earthquakes indicates that within the relatively brief period of a year or so following a major earthquake in a region, the most likely events that the region will experience are of a similar or reduced magnitude to the original shock. Therefore, these guidelines recommend evaluation of damaged structures for their ability to resist collapse (ability to provide Collapse Prevention performance) for such an event. For the purposes of accounting for variability in the likely locations and magnitudes of major after shocks, and also to permit development of confidence levels for ability of the building to provide Collapse Prevention performance, a one year return period is assumed for an arbitrary after shock, comparable in intensity at the building site to the initial shock. Variability in ground motion is somewhat arbitrarily accounted for by assuming a distribution of likely ground shaking at the building site due to such an aftershock that has a mean value equal to that which caused the original damage and having a coefficient of variation of 0.5.

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

5.4 Field Inspection

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

Geologic site hazards such as fault rupture, landslide, rock fall, and liquefaction may influence the damage in a building and also its future performance. A detailed discussion of these hazards is provided in *FEMA 273* and should be considered as part of a post-earthquake evaluation. The existing foundation system should be inspected to try to detect whether or not differential settlement has occurred. Differential movement between columns in a frame element 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

Knowing the specific mechanical and chemical properties of structural steel structures is critical to proper evaluation using level 2 methods. Mechanical properties of component and connection material dictate the structural behavior of the component under load. Mechanical properties of greatest interest include the expected yield (F_{ye}) and tensile (F_{te}) strengths of base and connection materials, modulus of elasticity, ductility, toughness, elongational characteristics, and weldability.

The effort required to determine these properties is related to the availability of original and updated construction documents, original quality of construction, accessibility, and condition of materials and the level of confidence required for the evaluation. For the purpose of level 2 evaluations, material properties should be based on the information presented on the original construction documents as supplemented by

Table 4-1, unless such documents are not available, in which case building specific sampling and testing should be performed.

When construction documents do not adequately define the material specifications for a structure, or the original construction documents are not available, the determination of material properties is best accomplished through removal of samples and laboratory testing. Sampling should take place in regions of reduced low stress, to minimize the effects of the resulting reduced area. It may be required to weld new material onto the component to offset the removal of samples. If a connector such as a bolt is removed for testing, a comparable bolt should be reinstalled at the time of sampling. Destructive removal of a welded connection sample must be accompanied by repair of the connection.

If sampling of in-place material is used to determine physical properties, the statistical values shall be calculated in accordance with the following. The expected, or mean, value shall be taken as given by the equation:

$$\bar{x} = \sum x \Big/ n \qquad (5\text{-}1)$$

The median value, $\hat{x}$, shall be taken as that value that is larger than 50% of the values determined from the sample. The standard deviation shall be calculated from the equation:

$$\sigma_x = \sqrt{\frac{\sum x^2 - \left(\sum x\right)^2}{n(n-1)}} \qquad (5\text{-}2)$$

where n is the size of the sample. The coefficient of variation shall be calculated from the equation:

$$COV = \frac{\sigma_x}{\bar{x}} \qquad (5\text{-}3)$$

Table 5-1 - Expected Material Properties for Structural Steel of Various Grades

Material Specification	Year of Construction	Expected Yield Strength - F_{ye} Ksi	Expected Tensile Strength - F_{ue}, Ksi	CVN Toughness ft-b
Plate and Shape				
A-7	1949 - 1965			
A-36	1960 - 1990			
	1990 -			
A242	1941 -			
A441	1960 -			
Group 1 and 2				
Group 3 and 4				
Group 5				
A572	1966 -			
Grade 42				
Grade 50				
Grade 60				
Grade 65				
A913				
Grade 50				
Grade 65				
Bolts				
A307	1947 -			
A325	1964 -			
A490	1982 -			
Weld Material				
E60XX[1]				
E70XX[1]				

Notes: 1- If the actual welding consumable specification is available refer to XXX for information

When available construction documents do not provide sufficient information on the material specifications to permit estimation of material properties, it is necessary to utilize proven destructive and nondestructive testing methods. To achieve the desired accuracy, mechanical properties should be determined in the laboratory. Particular laboratory test information that may be sought include yield and tensile strength, elongation, and Charpy V-notch toughness. For each test, industry standards published by the ASTM exist and should be followed. Applicable ASTM Standards are indicated in Table 5-2.

Table 5-2 - Standard Test Methods for Material Properties

Property	ASTM Standard Specification	
	Number	Title
Structural shape: Yield Strength, Tensile Strength, Charpy V-Notch Toughness	A370	Standard Test Methods and Definitions for Mechanical Testing of Steel Products
Weld metal: Tensile Strength, Toughness		
Bolts: Tensile Strength		

Of greatest interest to steel building system performance are the expected yield and tensile strength of the installed materials. Notch toughness of structural steel and weld material is also important for connections. Virtually all steel component elastic and inelastic limit states are related to yield and tensile strengths. Past research and accumulation of data by industry groups have resulted in published material mechanical properties for most primary metals and their date of fabrication, as indicated in Table 4-1. This information may be used, together with tests from recovered samples, to rapidly establish expected strength properties for use in component strength and deformation analyses.

Review of other properties derived from laboratory tests such as hardness, impact, fracture, and fatigue is generally not needed for steel component capacity determination, but may be required for connection evaluation. These properties may not be needed in the analysis phase if significant rehabilitative measure are already known to be required.

To quantify material properties and analyze the performance of welded moment connections, more extensive material property data is required including the carbon equivalent of the existing component(s). Appropriate welding procedures are dependent upon the chemistry of base metal (specifically elements in the IIW Carbon Equivalent formula). It is recommended that the carbon equivalent formula contained in American Welding Society, D1.1 Structural Welding Code, be used.

When construction documents do not adequately indicate the materials specifications for building components, the guidelines given below should be followed for determining the expected yield (F_{ye}) and tensile (F_{te}) strengths:

- If no knowledge exists of the structural systems and materials used in these systems, at least two strength tensile coupons should be removed from each element type for every four floors. If it is determined from testing that more than one material grade exists, additional testing should be performed until the extent of use for each grade in component fabrication has been established. If it is determined that all components are made from the same material specification, the requirements immediately preceding this maybe followed.

- In the absence of construction records defining welding filler metals and processes used, at least one weld metal sample for each construction type should be obtained for laboratory testing. The sample shall consist of both local base and weld metal, such that composite strength of the connection can be derived. If ductility is required at or near the weld, the design professional may conservatively assume that no ductility is available in lieu of testing.

- Bolt specifications may typically be determined by reference to markings on the heads of the bolts. Where head markings are obscured, or not present, testing requirements for bolts are the same for other steel components as given above.

For all laboratory test results, the mean yield and tensile strength may be interpreted as the expected strength for component strength calculations.

For other material properties, the design professional shall determine the particular need for this type of testing and establish an adequate protocol consistent with that given above. In general, it is recommended that a minimum of three tests be conducted.

5.6 Structural Performance Confidence Evaluation

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

- Inter-story drift

- Axial forces, moments and shears on individual elements

These structural response parameters are related to the amount of damage experienced by individual structural components as well as the structure as a whole. These guidelines specify acceptance criteria for each of the design parameters indicated above, for a Collapse Prevention performance level. The Collapse Prevention level, is that performance level in which the structure is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force-resisting system, large permanent lateral deformation of the structure, and, to a more limited extent, degradation in the vertical load-carrying capacity. However, all significant components of the gravity load-resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for re-occupancy, further aftershock activity could credibly induce collapse.

Acceptance criteria are limiting values for the various design parameters, at which damage corresponding to the Collapse Prevention performance level, has a significant probability of exceedance. Acceptability of structural performance is evaluated considering both local (element level) performance and global performance. Acceptance criteria have been developed on a reliability basis, incorporating load and resistance factors related to the uncertainty inherent in the evaluation process, such that a confidence level can be established with regard to the ability of the structure to actually provide Collapse Prevention performance for the ground motions anticipated to occur within the time period immediately following a damaging earthquake.

Once an analysis is performed, predicted demands are factored by load factors, λ, to account for the uncertainty inherent in their computation, as well as variability in structural response, and compared against acceptance criteria, which have also been factored, by resistance factors, ϕ, to account for uncertainties and variation inherent in structural capacity. If the factored demands are less than the factored acceptance criteria (capacities), then the structure is indicated to be capable of meeting the desired performance, with at least a mean level of confidence. If the factored demands exceed the factored acceptance criteria, then there is less than a mean level of confidence that the predicted performance will be attained for the specified exceedance probability. Procedures are provided to permit calculation of the level of confidence, based on the ratio of factored capacity to factored demand. If the predicted level of confidence is inadequate, then the occupancy of the structure should be suspended until such time as the structure can be temporarily shored, and/or repaired, and a suitable level of confidence attained. In some cases it may be possible to improve the level of confidence with regard to the ability of a building to resist collapse by performing a more detailed analysis. More detailed and accurate analyses allow better understanding of the structure's behavior to be attained, resulting in modifications to the load and resistance factors.

Table 5-3 summarizes the recommended posting condition for a building, as a function of the level of confidence determined based on the calculated confidence attained with regard to the structure's ability to resist collapse for the level of ground shaking likely to be experienced in the immediate post earthquake period. Refer to Table 3-1 for information on the recommended actions related to each posting.

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

Confidence Level of Attaining Collapse Prevention Performance	Recommended Occupancy Posting[1]
90% or greater confidence of non-collapse	Green-1, Green-2, or Green-3, as appropriate

50% or greater confidence of non-collapse but less than 90%	Yellow-2[1]
25% or greater confidence of non-collapse but less than 50%	Red-1[1]
less than 25% confidence of non-collapse	Red-2[1]
1- Refer to table 2-1 for explanation of postings	

Four alternative analytical procedures are permitted in these guidelines, for the prediction of building response parameters. These are the same basic procedures contained in FEMA-273 including the Linear Static Procedure (LSP); the Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP) and Nonlinear Dynamic Procedure (NDP) Procedure. Section 3.5.6 outlines these procedures in some detail. The reader is referred to FEMA-273 for additional information and discussion.

Commentary: These guidelines adopt a load and resistance factor design (LRFD) model for evaluation of structural performance. The purpose of this LRFD approach is to develop estimates of the confidence level inherent in a damaged building with regard to its ability to provide collapse prevention performance given the probable ground shaking which may be experienced in the period immediately following a damaging earthquake, taken as 1 year.

In order to permit this process to occur, it is necessary to presume a hazard relationship for the site, during this immediate post-earthquake period. Most strong earthquakes are followed by a large number of aftershocks, that decrease in frequency over time. Aftershocks typically occur on the same fault on which the main shock occurred, though occasionally, an earthquake on a nearby fault has been triggered by the redistribution in crustal strains produced by the earthquake. Aftershocks typically are of the same magnitude as the main shock, or smaller, though there are some instances when after shocks have actually exceeded the first shock. Generally, aftershock activity decays to insignificant levels within a period of approximately a year following the main event.

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

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

Using the S_a value estimated to have been experienced during the first damaging earthquake, a structural analysis is performed to determine the maximum inter-story drift demand for the damaged structure under a repeat of that event. This is factored by a load factor, λ, to account for the uncertainty and variation inherent in the analytical process related to inaccuracies inherent in the analytical approach, the modeling of the structure, and the estimation of the ground motion itself. The load factor λ, is calculated as:

$$\lambda = \beta e^{\left(\frac{k}{2b}\Sigma \sigma_i^2\right)} \tag{5-4}$$

where β is a bias factor, that accounts for under or over-prediction of inter-story drift inherent in a particular analytical procedure, k is the slope of the hazard curve, evaluated in log-log coordinates, b is a regression coefficient that relates variation in inter-story drift to hazard, and which may typically be taken as unity, and $\Sigma\sigma_i^2$ is the sum of the standard deviations of the logarithmic distribution of interstory drift predictions relative to the various random and uncertain parameters. Tabulated values of these λ factors are provided in these guidelines for various analytical procedures and typical framing conditions.

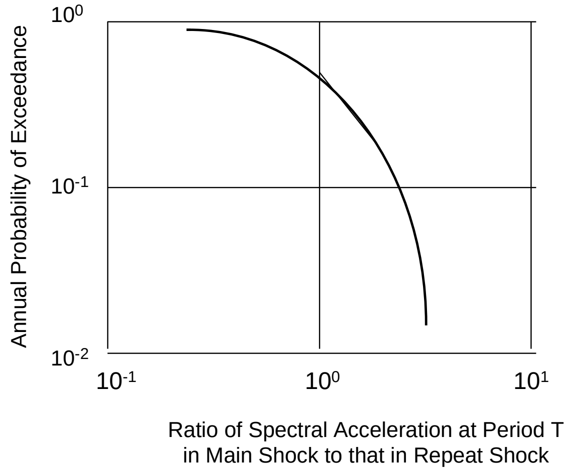


Figure 5-1 Presumed Post-earthquake Hazard Curve

The factored demand, calculated from the analysis represents a mean estimate of the probable maximum inter-story drift demand during the immediate post-earthquake period, given the assumed distribution of ground shaking during this period, represented by the hazard curve.

These guidelines also tabulate median estimates of interstory drift capacity for individual elements and the global structure. These drift capacities are dependent on frame and connection configuration. In addition to drift capacities, capacity reduction factors, ϕ, that adjust the estimated capacity of the structure to a mean value are also provided.

Once the factored demand and capacities are determined, a parameter, γ_{con} is calculated from the equation:

$$\gamma_{con} = \frac{\phi \Delta_C}{\lambda \Delta_D} \qquad (5\text{-}5)$$

The value of γ_{con} is then used directly to determine an associated confidence level for the desired performance, based on tabulated values related to both the slope of the hazard curve and also the uncertainty inherent in the estimation of the building's demand and capacities. Values of γ_{con} exceeding 1.0 indicate greater than mean confidence of achieving the desired performance. Values less than 1.0 indicate less than mean confidence.

5.7 Ground Motion Representation

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

5.7.1 Instrumental Recordings

When an actual recording of the ground shaking that caused the damage, obtained from the building site, or a nearby site with similar conditions is available, this shall be used directly to perform analyses of the damaged structure. The ground acceleration history shall be converted into a smoothed, 5% damped response spectrum, similar in form to the generalized response spectrum described in FEMA-273, and completely enveloping the actual response spectrum obtained for the acceleration record over the period range 0.5T to 2.0T, where T is the computed fundamental period of the damaged structure. If the selected analytical procedure is response history analysis, a suite of accelerograms constructed in accordance with the recommendations of FEMA-273 shall be used, one of which shall be the actual site recording.

Commentary: The best possible estimate of ground shaking experienced at a site consists of actual ground motion recordings obtained from a free-field instrument located at the building site. Free field instruments are preferable to instruments located within the building or another structure as they will not be influenced by structural response effects.

Even in zones of very high seismicity, very few buildings have functioning strong motion instrumentation, so therefore, 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 was at a comparable distance and azimuth to the fault rupture, as was 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 NEHRP Provisions, FEMA-302.

The intent of post-earthquake assessment analyses is not to evaluate the damaged building's response for the actual ground shaking that caused the original damage, but rather to evaluate this response for ground shaking likely to be experienced in the immediate post-earthquake period. As previously discussed, this is likely to be similar, though not identical to that which caused the original damage. For this reason, response spectra obtained from actual ground motion recordings are smoothed, to approximate a standard Newmark and Hall spectrum, as described in FEMA-273.

5.7.2 Estimated Ground Motion

When instrumental recordings of the damaging ground shaking, as described in Section 4.7.1 are not available, an estimated response spectrum for this ground shaking shall be constructed. These spectra shall be constructed as recommended by FEMA-273 except that rather than using mapped values for the parameters S_S and S_1, these parameters shall be calculated using standard attenuation relationships and appropriate estimates of the magnitude of the damage causing event, its distance from the building site, the site soil characteristics, faulting mechanism and other parameters required by the attenuation equation. Alternatively, these parameters may be estimated based on available recordings of ground shaking from the damage causing event.

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

5.8 Analytical Procedures

In order to evaluate the performance of an MRSF structure it is necessary to construct a mathematical model of the structure that represents its strength and deformation characteristics and to conduct an analysis to predict the values of various design parameters when it is subjected to design ground motion. This section provides guidelines for selecting an appropriate analysis procedure and for modeling. General requirements for the mathematical model are presented in Section 5.9.

Four alternative analytical procedures are available for use in systematic performance evaluation of WMSF structures. The basic analytical procedures are based on the analytical procedures contained in *FEMA-273* and shall generally be conducted in accordance with the recommendations of that publication, except as specifically modified herein. 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 or an elastic time history analysis

- Nonlinear static procedure - a simplified nonlinear analysis procedure in which the forces and deformations induced by a monotonically increased pattern of 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 ground motion time history is determined through numerical integration of the equations of motion for the structure.

Structural stiffness is altered during the analysis to conform to nonlinear hysteretic models of the structural components.

Commentary: The purpose of structural analyses performed as part of the post-earthquake assessment process is to predict the values of key response parameters, that are indicative of the damaged structure's performance, if it should be subjected to a repeat of the damaging ground motion. Once the values of these response parameters are predicted, the structure is evaluated for its ability to resist collapse using the basic equation:

$$\gamma_{con} = \frac{\phi\Delta_C}{\lambda\Delta_D} \tag{5-6}$$

where:

λ = *a load factor to account for uncertainty in the prediction of demands (the value of the response parameters)*

D = *the demand predicted for a repeat of the original damaging shaking*

ϕ = *a capacity reduction factor to account for uncertainty in the capacity of the structure*

C= *the capacity of the structure to resist collapse as measured by the specific design parameter (acceptance criteria)*

γ_{con} = *an index parameter by which confidence in performance prediction can be related*

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

Analyses conducted in support of post-earthquake assessment, under these guidelines, take a markedly different approach. Rather than evaluating the forces and deformations induced in the structure under arbitrarily reduced loading levels, these analysis procedures attempt to predict, within probabilistically defined bounds, the actual values of the important response parameters related to collapse for the level of ground shaking likely to be experienced in the immediate post-earthquake period.

The ability of this post-earthquake assessment evaluation to reliably *estimate the probable performance of the structure is dependent on the ability of the analysis procedure to predict the values of these response parameters within acceptable levels of confidence. The linear dynamic procedure is able to provide relatively reliable estimates of the response parameters for structures that exhibit elastic, or near elastic behavior, however, few damaged structures will behave in such a manner. The linear static procedure inherently has more uncertainty associated with its estimates of the response parameters because it less accurately accounts for the dynamic characteristics of the structure. The nonlinear static procedure is more reliable than the linear procedures in predicting response parameters for structures that exhibit significant nonlinear behavior, particularly if they are irregular. However, it does not accurately account for the effects of higher mode response and therefore, when used for structures in which higher mode response is significant, must also be accompanied by a linear dynamic analysis. If appropriate modeling is performed, the nonlinear dynamic approach is most capable of capturing the probable behavior of the real structure in response to ground motion, however, there are considerable uncertainties associated even with the values of the response parameters predicted by this technique. Unique load factors, λ, are specified for each of the analysis methods, depending on the performance levels, to account for these uncertainties.*

5.8.1 Procedure Selection

Table 5-4 indicates the recommended analysis procedures for various performance levels and conditions of structural regularity. Also indicated in the table are the load factors, λ, associated with each.

Table 5-4 - Recommended Analysis Procedures

Analysis Procedure			
Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Permitted for regular structures, as indicated in FEMA-273 $\lambda = 2.0$	Permitted for regular structures, as indicated in FEMA-273 $\lambda = 1.5$	Permitted for regular or irregular structures, with periods less than 1.0 second and as indicated in FEMA-273 $\lambda = 1.2$	Permitted for all structures, as indicated in FEMA-273 $\lambda = 1.0$

5.8.2 Linear Static Procedure (LSP)

5.8.2.1 Basis of the Procedure

Linear static procedure analysis of damaged MRSF structures shall be conducted in accordance with the Guidelines of *FEMA-273,* except as specifically noted herein. In this procedure, a total lateral force is applied to the structure, and deflections and component forces under this applied loading is determined.

Results of the LSP are evaluated using the applicable acceptance criteria of Section 5.10. Calculated internal forces typically will exceed those that the building can develop, because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 5.10.

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

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

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

Although performance of MRSF structures is closely related to inter-story drift demand, there are some failure mechanisms, notably, failure of column splices, that are more closely related to strength demand. However, since inelastic structural behavior affects the strength demand on such elements, linear analysis is not capable of directly predicting these demands, either, except when the structural response is essentially elastic. Therefore, as with inter-story drift demand, correlation coefficients have been developed that allow approximate estimation of the strength demands on such elements by adjusting demands calculated from the linear analysis.

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

It should be noted that most damaged structures will behave in a more non-linear manner than will undamaged structures, even when subjected to relatively low levels of ground shaking. Beam-column connections with fractures at the bottom flange of the beam, for example, will behave much like undamaged, fully restrained joints when loaded such that the fractured flange is in compression and will behave much like pinned joints when loading produces tension at the bottom flange. Such behavior can not be accurately reflected in elastic analysis. In order to minimize the potential for analysis inaccuracies to result in overly optimistic estimates of the actual response of a damaged structure, these guidelines suggest *what are believed to be conservative modeling assumptions for damaged framing elements. However, the uncertainties inherent in the use of linear methods to model highly damaged structures are so large that it is recommended they not be used for this purpose.*

5.8.2.2 Modeling and Analysis Considerations

5.8.2.2.1 Period Determination

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

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

Method 2: Evaluation of the following equation:

$$T = C_t h_n^{3/4} \qquad (5\text{-}7)$$

where

T = Fundamental period (in seconds) in the direction under consideration

C_t =0.035 for moment-resisting frame systems of steel

h_n = Height (in feet) above the base to the roof level

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

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \qquad (5\text{-}8)$$

where Δ_w and Δ_d are in-plane frame and diaphragm displacements in inches, due to a lateral load, in the direction under consideration, equal to the weight tributary to the diaphragm. For multiple-span diaphragms, a lateral load equal to the gravity weight tributary to the diaphragm span under consideration should be applied to each diaphragm span to calculate a separate period for each diaphragm span. The period so calculated that maximizes the pseudo lateral load shall be used for design of all walls and diaphragm spans in the 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 4-9, shall be independently calculated for each of two orthogonal directions of building response, and applied to a mathematical model of the damaged building structure.

$$V = C_1 C_2 C_3 S_a W \tag{5-9}$$

where:

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 may be calculated using the procedure indicated in Section 3.3.3.3 in FEMA 273 with the elastic base shear capacity substituted for V_y. Alternatively, C_1 may be taken from Table 4-5

Linear interpolation shall be used to calculate C_1 for intermediate values of T.

T = Fundamental period of the damaged building in the direction under consideration. If soil-structure interaction is considered, the effective fundamental period T shall be substituted for T.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum, estimated for the original damage causing ground shaking.

C_2 = Modification factor to represent the effect of stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems are listed in Table 5-5. Linear interpolation shall be used to estimate values for C_2 for intermediate values of T.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For values of the stability coefficient θ (see Equation 5-10) less than 0.2, C_3 may be set equal to 1.0 For values of θ greater than 0.1, C_3 shall be calculated as 1 + 5 (θ-0.1)/T. The maximum value θ for all stories in the building shall be used to calculate C_3. Alternatively, the values of C_3 in Table 4-5 may be used.

$$\theta = \frac{P\Delta}{VH} \tag{5-10}$$

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.

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

- In storage and warehouse occupancies, a minimum of 25% of the floor live load
- The actual partition weight or minimum weight of 10 psf of floor area, whichever is greater
- The applicable snow load – see the *NEHRP Recommended Provisions* (BSSC, 1998)
- The total weight of permanent equipment and furnishings

Table 5-5 - Correlation Coefficients for Linear Static Procedure

Performance Level	C1	C2	C3
T< 1.0 Sec T $\geq$ 1.0 Sec	2.0 1.0	1.2	1.4

Commentary: This force, when distributed over the height of the linearly-elastic analysis model of the structure, is intended to produce calculated lateral displacements approximately equal to those that are expected in the real structure during the design event. If it is expected that the actual structure will yield during the design event, the force given by Equation (4-9) may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in Section 4.10 are developed to take this aspect into account.

5.8.2.3.2 Vertical Distribution of Seismic Forces

The lateral load F_x applied at any floor level x shall be determined from the following equations:

$$F_x = C_{vx} V \tag{5-11}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{5-12}$$

where

k = 1.0 for T $\leq$ 0.5 second

= 2.0 for T $\geq$ 2.5 seconds

Linear interpolation shall be used to estimate values of k for the intermediate values of T.

C_{vx} = Vertical distribution factor

V = Pseudo lateral load from Equation (5-9)

w_i = Portion of the total weight W located on or assigned to floor level i

w_x = Portion of the total building weight W located on or assigned to floor level x

h_i = Height (in ft) from the base to floor level i

h_x = Height (in ft) from the base to floor level x

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 Floor Diaphragms

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

5.8.2.3.5 Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral loads in accordance with Equations 5-9, and 5-11 and stiffnesses obtained from Section 5.9. Factored inter-story drift demands, $\lambda\delta_i$, at each story "i", shall be determined by applying the appropriate load factor, λ, obtained from Table 5-4.

5.8.2.3.6 Determination of Column Demands

Columns and column splices shall be evaluated for factored axial demands, P_c', obtained from the equation:

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \qquad (5\text{-}13)$$

where: P is the axial load in the element computed from the analysis
C1, C2, and C3 are the coefficients previously defined, and
λ_c is obtained from Table 4-6

Table 5-6 Value of Load Factors λ_c for Columns - Linear Static Procedure

Column Located In	$\overline{M/M_p}$[1]

	≤ 1	$1 < \overline{M/M_p} \leq 2$	$2 < \overline{M/M_p}$
Top 3 stories of building	1.25	$\dfrac{1.5}{\overline{M/M_p}}$	$\dfrac{1.75}{\overline{M/M_p}}$
10 stories below the top 3 stories	1.25	$\dfrac{1.25}{\overline{M/M_p}}$	$\dfrac{1.35}{\overline{M/M_p}}$
All other	1.25	$\dfrac{1.15}{\overline{M/M_p}}$	$\dfrac{1.25}{\overline{M/M_p}}$

1. $\overline{M/M_p}$ is the average of the ratio of beam moments calculated from the analysis to the plastic moment capacities of the beams, for all beams framing into the column in stories above the level under consideration.

5.8.3 Linear Dynamic Procedure (LDP)

5.8.3.1 Basis of the Procedure

Linear dynamic procedure analysis of damaged MRSF structures should generally be conducted in accordance with the Guidelines of *FEMA-273*, except as specifically noted herein. Coefficients C_1, C_2, and C_3 should be taken as indicated in Table 5-5.

> *Commentary: The linear dynamic procedure is similar in approach to the linear static procedure, described in the previous section. However, because it directly accounts for the stiffness and mass distribution of the structure in calculating the dynamic response characteristics, it is somewhat more accurate. Coefficients C_1, C_2, and C_3, which account in an approximate manner for the differences between elastic predictions of response and inelastic behavior are the same as for the linear static method under the Linear Dynamic Procedure (LDP), design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements are determined using a linearly-elastic, dynamic analysis. Note that although the LDP is more accurate than the LSP for analysis purposes, it can still be quite inaccurate when applied to heavily damaged structures and should be used with caution.*

The basis, modeling approaches, and acceptance criteria of the LDP are similar to those for the LSP. The main exception is that the response calculations are carried out using either Modal Spectral Analysis or Response-History 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.

Estimates of inter-story drift and column axial demands shall be evaluated using the applicable acceptance criteria of Section 5.10. Calculated displacements are factored by the applicable load factor, λ, obtained from Table 5-7 and compared with factored acceptable values, per Section 5.10. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements. These obtained design forces are evaluated through the acceptance criteria of Section 5.10.

5.8.3.2 Modeling and Analysis Considerations

5.8.3.2.1 General

The 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 includes two analysis methods, namely, the Response Spectrum (RSA) and Response-History Analysis (RHA) methods. The RSA uses peak modal responses calculated from elastic dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Modal responses are combined using rational methods to estimate total building response quantities. RSH involves a time-step-by-time-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input. Requirements for the two analysis methods are outlined below.

5.8.3.2.2 Response Spectrum Method

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

The peak member forces, displacements, story forces, story shears, and base reactions for each mode of response should be combined by recognized methods to estimate total response. Modal combination by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule is acceptable.

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

5.8.3.2.3 Response History Analysis

The requirements for the mathematical model for Response-History Analysis are identical to those developed for Response Spectrum Analysis. The damping matrix associated with the mathematical model should reflect the damping inherent in the building at deformation levels less than the yield deformation.

Response-History Analysis should be performed using a minimum of three spectrum compatible ground motions.

Response parameters should be calculated for each ground motion record. If three Response-History Analyses are performed, the maximum response of the parameter of interest should be used for design. If seven or more pairs of horizontal ground motion records are used for Response-History Analysis, the average response of the parameter of interest may be used for design.

Where three dimensional analyses are performed, multidirectional excitation effects should be accounted for by evaluating the response due to concurrent excitation to pairs of time histories. Where two dimensional analyses are performed, multidirectional excitation effects should be accounted for in the same manner as for RSA analysis.

5.8.3.3 Determination of Actions and Deformations

5.8.3.3.1 Factored Inter-story Drift Demand

Factored interstory drift demand shall be obtained by mulitplying the results of the RSA or RSH analysis by the product of the modification factors, C_1, C_2, and C_3 defined in Section 5.8.2.3 and by the applicable λ obtained from Table 5-4.

5.8.3.3.2 Determination of Column Demands

Columns and column splices shall be evaluated for factored axial demands, P_c', obtained from the equation:

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \tag{5-14}$$

where: P is the axial load in the element computed from the analysis
C1, C2, and C3 are the coefficients previously defined, and
λ_c is obtained from Table 4-7

Table 5-7 Value of Load Factors λ_c for Columns - Linear Dynamic Procedure

Column Located In	$\overline{M/M_p}$ [1]		
	≤ 1	$1 < \overline{M/M_p} \leq 2$	$2 < \overline{M/M_p}$
Top 3 stories of building	1.0	$\frac{1.25}{\overline{M/M_p}}$	$\frac{1.5}{\overline{M/M_p}}$
10 stories below the top 3 stories	1.0	$\frac{1.15}{\overline{M/M_p}}$	$\frac{1.25}{\overline{M/M_p}}$
All other	1.0	$\frac{1.10}{\overline{M/M_p}}$	$\frac{1.15}{\overline{M/M_p}}$

1. $\overline{M/M_p}$ is the average of the ratio of beam moments calculated from the analysis to the plastic moment capacities of the beams, for all beams framing into the column in stories above the level under consideration.

5.8.4 Nonlinear Static Procedure (NSP)

5.8.4.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the inelastic material and geometric response of the damaged structure is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the damaged building are modeled directly. The mathematical model of the building is subjected to a pattern of monotonically increasing lateral forces or displacements until either a target displacement is exceeded or mathematical instability occurs. The target displacement is intended to approximate the total maximum displacement likely to be experienced by the actual structure, in response to the ground shaking anticipated during the immediate post-earthquake period. The target displacement may be calculated by any procedure that accounts for the effects of nonlinear response on displacement amplitude; one rational procedure is presented in Section 5.8.4.3.1. Because the mathematical model accounts directly for effects of material and geometric inelastic response, the calculated internal forces will be reasonable approximations of those expected during the anticipated ground shaking, presuming that an appropriate pattern of loading has been applied.

Results of the NSP are to be evaluated using the applicable acceptance criteria of Section 5.10. Calculated inter-story drifts and column and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

5.8.4.2 Modeling and Analysis Considerations

5.8.4.2.1 General

In the context of these *Guidelines*, the 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, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging between zero and 150% of the target displacement, δ_t, given by Equation 5-15. Post-earthquake assessment shall be based on those column forces and inter-story drifts corresponding to minimum horizontal displacement of the control node equal to the target displacement, δ_t.

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

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.

5.8.4.2.2 Control Node

The NSP requires definition of the control node in a building. These *Guidelines* consider the control node to be the center of mass at the roof of a 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 that approximately bound the likely distribution of inertia forces in the anticipated ground shaking. For three-dimensional analysis, the horizontal distribution should simulate the distribution of inertia forces in the plane of each floor diaphragm. For both two- and three-dimensional analysis, at least two vertical distributions of lateral load should be considered. The first pattern, often termed the "uniform pattern," should be based on lateral forces that are proportional to the total mass at each floor level. The second pattern, termed the modal pattern in these *Guidelines*, should be selected from one of the following two options:

- A lateral load pattern represented by values of C_{vx} given in Equation 5-12, which may be used if more than 75% of the total mass participants in the fundamental mode in the direction under consideration; or

- A lateral load pattern proportional to the story inertia forces consistent with the story shear distribution calculated by combination of modal responses using (1) Response Spectrum Analysis of the building including a sufficient number of modes to capture 90% of the total mass, and (2) the appropriate ground motion spectrum.

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. The nonlinear relation between base shear and displacement of the target node should be replaced with a bilinear relation to estimate the effective lateral stiffness, K_e, and the yield strength, V_y, of the building. The effective lateral stiffness should be taken as the secant stiffness calculated at a base shear force equal to 60% of the yield strength. The effective fundamental period T_e should be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{5-15}$$

where:

T_i = Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness of the building in the direction under consideration

K_e = Effective lateral stiffness of the building in the direction under consideration

See Figure 5-2 for further information.

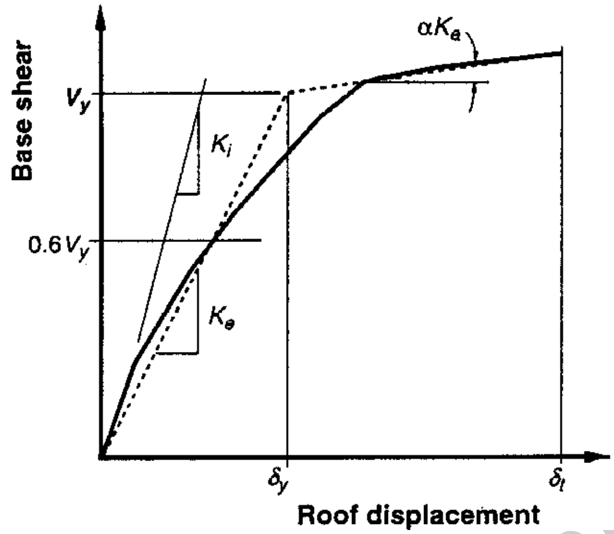


Figure 5-2 - Calculation of Effective Stiffness, K_e

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. The effects of accidental torsion should be considered.

Independent analysis along each principal axis of the three-dimensional mathematical model is permitted unless multidirectional evaluation is recommended.

5.8.4.2.6 Analysis of Two-Dimensional Models

Mathematical models describing the framing along each axis (axis 1 and axis 2) of the building should be developed for two-dimensional analysis. The effects of horizontal torsion should be considered.

If multidirectional excitation effects are to be considered, component deformation demands and actions should be computed for the following cases: 100% of the target displacement along axis 1 and 30% of the target displacement along axis 2; and 30% of the target displacement along axis 1 and 100% of the target displacement along axis 2.

5.8.4.3 Determination of Actions and Deformations

5.8.4.3.1 Target Displacement

The target displacement δ_t for a building with a rigid diaphragm at each floor level should be estimated using an established procedure that accounts for the likely nonlinear response of the building.

One procedure for evaluating the target displacement is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad (5\text{-}16)$$

where:

T_e = Effective fundamental period of the building in the direction under consideration, sec

C_0 = Modification factor to relate spectral displacement and likely building roof displacement.

Estimates for C_0 can be calculated using one of the following:

- the first modal participation factor at the level of the control node
- the modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at the target displacement
- the appropriate value from Table 5-8

C_1 = Modification factor to relate expected maximum inelastic displacement to displacements calculated for linear elastic response

= 1.0 for $T_e \geq T_0$

= $[1.0 + (R - 1)T_0/T_e]/R$ for $T_e < T_0$

Values for C_1 need not exceed those values given in Section 5.8.2.3. In no case may C_1 be taken as less than 1.0.

T_0 = Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R = Ratio of elastic strength demand to calculated yield strength coefficient. See below for additional information.

C_2 = Modification factor to represent the effect of hysteresis shape on the maximum displacement response. Values for C_2 are established in Section 5.8.2.3.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, C_3 should be set equal to 1.0. For buildings with negative post-yield stiffness, values of C_3 should be calculated using as set for in Section 5.8.2.3.

S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, g.

The strength ratio R should be calculated as:

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \quad (5\text{-}17)$$

Table 5-8 **- Values for Modification Factor C_0**

Number of Stories	Modification Factor[1]
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

1. Linear interpolation should be used to calculate intermediate values.

where S_a and C_0 are as defined above, and:

V_y = Yield strength calculated using results of NSP, where the nonlinear force-displacement (i.e., base shear force versus control node displacement) curve of the building is characterized by a bilinear relation (Figure 5-2).

W = Total dead load and anticipated live load, as calculated in Section 5.8.2.3.

Coefficient C_3 should be calculated as follows if the relation between base shear force and control node displacement exhibits negative post-yield stiffness.

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \quad (5\text{-}18)$$

where R and T_e are as defined above, and:

α = Ratio of post-yield stiffness to effective elastic stiffness, where the nonlinear force-displacement relation is characterized by a bilinear relation (Figure 5-2)

For a building with flexible diaphragms at each floor level, a target displacement should be estimated for each line of vertical seismic framing. The target displacements should be estimated using an established procedure that accounts for the likely nonlinear response of the seismic framing. One procedure for evaluating the target displacement for an individual line of vertical seismic framing is given by Equation 4-16. The fundamental period of each vertical line of seismic framing, for calculation of the target displacement, should follow the general procedures described for the NSP; masses should be assigned to each level of the mathematical model on the basis of tributary area.

For a building with neither rigid nor flexible diaphragms at each floor level, the target displacement should be calculated using rational procedures. One acceptable procedure for including the effects of diaphragm flexibility is to multiply the displacement calculated using Equation 5-16 by the ratio of the maximum displacement at any point on the roof and the displacement of the center of mass of the roof, both calculated by modal analysis of a three-dimensional model of the building using the ground shaking response spectrum. The target displacement so calculated should be no less than that displacement given by Equation 4-16, assuming rigid diaphragms at each floor level. No vertical line of seismic framing should be evaluated for displacements smaller than the target displacement. The target displacement should be modified according to Section 5.9 to account for system torsion.

5.8.4.3.2 Floor Diaphragms

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

5.8.4.3.3 Factored Inter-story Drift Demand

Factored inter-story drift demand shall be obtained by multiplying the maximum inter-story drift calculated at the target displacement by the applicable load factor λ obtained from Table 5-4.

5.8.4.3.4 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the the calculated column forces at the target displacement by the applicable load factor λ from Table 5-4.

5.8.4.4 Nonlinear Dynamic Procedure (NDP)

5.8.4.4.1 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, modeling approaches, and 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 histories. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is recommended to carry out the analysis with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during ground shaking.

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

5.8.4.5 Modeling and Analysis Assumptions

5.8.4.5.1 General

The NDP should conform to the criteria of this section. The analysis should be based on characterization of the seismic hazard in the form of ground motion records. The modeling and analysis considerations set forth in Section 5.9 should apply to the NDP unless the alternative considerations presented below are applied.

The NDP requires Response-History Analysis of a nonlinear mathematical model of the building, involving a time-step-by-step evaluation of building response, using discretized recorded or synthetic earthquake records as base motion input.

5.8.4.5.2 Ground Motion Characterization

The earthquake shaking should be characterized by ground motion time histories, prepared in accordance with the guidelines of Section 5.9. A minimum of three pairs of ground motion records shall be used..

5.8.4.5.3 Response-History Method

Response-History Analysis should be performed using horizontal ground motion time histories.

Multidirectional excitation effects should be accounted for by meeting the requirements of Section 5.9. The requirements of Section 5.9 may be satisfied by analysis of a three-dimensional mathematical model using simultaneously imposed pairs of earthquake ground motion records along each of the horizontal axes of the building.

5.8.4.6 Determination of Actions and Deformations

5.8.4.6.1 Modification of Demands

The effects of torsion should be considered according to Section 5.9.

5.8.4.6.2 Factored Inter-story Drift Demand

Factored inter-story drift demand shall be obtained by multiplying the maximum inter-story drift calculated at the target displacement by the applicable load factor λ obtained from Table 3-8.

5.8.4.6.3 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the the calculated column forces at the target displacement by the applicable load factor λ from Table 5-4.

5.9 Mathematical Modeling

5.9.1 Modeling Approach

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

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.

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 Frame 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 5.9.10 for guidelines on modeling damaged connections.

Commentary: Gravity framing, in which beams are connected to columns with either clip angels or single clip plates can provide significant secondary stiffness to a structure and should in general be modeled when performing 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.

5.9.3 Horizontal Torsion

The effects of horizontal torsion must be considered. The total torsional moment at a given floor level should be set equal to the sum of the following two torsional moments:

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

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

In buildings with rigid diaphragms the effect of actual torsion should be considered if the maximum lateral displacement, $\delta_{max,}$ from this effect at any point on any floor diaphragm exceeds the average displacement, δ_{avg}, by more than 10%. The effect of accidental torsion should be considered if the maximum lateral displacement due to this effect at any point on any floor diaphragm exceeds the average displacement δ_{avg}, by more than 10%. This effect should be calculated independently of the effect of actual torsion.

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

If the ratio, η, of (1) the maximum displacement at any point on any floor diaphragm (including torsional amplification), to (2) the average displacement, calculated by rational analysis methods, exceeds 1.50, three-dimensional models that account for the spatial distribution of mass and stiffness should be used for analysis and design. Subject to this limitation, the effects of torsion may be indirectly captured for analysis of two-dimensional models as follows:

a. For the LSP and the LDP, the design forces and displacements should be increased by multiplying by the maximum value of η calculated for the building.

b. For the NSP, the target displacement should be increased by multiplying by the maximum value of η calculated for the building.

c. For the NDP, the amplitude of the ground acceleration record should be increased by multiplying by the maximum value of η calculated for the building.

5.9.4 Foundation modeling

Foundations should, in general, be modeled as non-compliant supports (fixed base condition). Soil-structure interaction may be modeled as permitted by the building code. 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.

5.9.5 Diaphragms

Floor diaphragms should be classified as either flexible, stiff, or rigid. Diaphragms should be considered *flexible* when the maximum lateral deformation of the diaphragm along its length is more than twice the average interstory drift of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm may be used in lieu of the basement story. Diaphragms should be considered *rigid* when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the associated story. Diaphragms that are neither flexible nor rigid should be classified as *stiff*. The interstory drift and diaphragm deformations should be estimated using the seismic lateral forces prescribed in the building code. The in-plane deflection of the floor diaphragm should be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing at that floor.

Mathematical models of buildings with stiff or flexible diaphragms should be developed considering the effects of diaphragm flexibility. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be designed independently, with seismic masses assigned on the basis of tributary area.

Most floor slabs with concrete fill over metal deck may be considered to be rigid diaphragms. Floors or roofs with plywood diaphragms should be considered flexible. The flexibility of unfilled metal deck, and concrete slab diaphragms with large openings should be considered in the analytical model.

5.9.6 P-Delta effects

Two types of P-Δ (second-order) effects are addressed in the *Guidelines*: (1) static P-Δ and (2) dynamic P-Δ.

Commentary: Structure P-delta effect, caused by gravity loads acting on the displaced configuration of the structure, may be critical in the seismic performance of SMRF structures, which are usually rather flexible and may be subjected to relatively large lateral displacements.

Structure P-delta effect has consequences from the perspectives of statics and dynamics. In a static sense this effect can be visualized as an additional lateral loading that causes an increase in member forces and lateral deflections, reduces the lateral resistance of the structure, and may cause a negative slope of the lateral load - displacement relationship at large displacements. This response is obtained from an accurate distributed plasticity analysis of the frame. From a static perspective the maximum lateral load that can be applied to the structure is a critical quantity since this load cannot be maintained as displacements increase, and a sidesway collapse is imminent. From a dynamic perspective this maximum load is not a critical quantity since seismic "loading" implies energy input, and stability is maintained as long as energy can be dissipated within the structural system. In concept, collapse will not occur unless the lateral forces due to P-delta effects exceed the available *restoring forces. These restoring forces include the internal forces generated in the structure, as a result of its displaced shape, as well as inertial forces induced by continued shaking and response of the structure to this shaking.*

An accurate determination of the inelastic response that includes all aspects of member and structure P-delta effects is possible only through a distributed plasticity finite element analysis. To be reliable, this analysis should also incorporate local and flexural-torsional buckling effects. The response determination under cyclic loading is even more complex, particularly if strength and/or stiffness deterioration have to be considered. If local and flexural-torsional buckling problems are avoided, if member P-delta effects and out-of-plane buckling are not important issues, and if strength and stiffness deterioration are prevented, then a second order concentrated plasticity (plastic hinge) analysis should be adequate for an assessment of P-delta effects. The following discussion is based on these assumptions.

For structures of more than one story (MDOF systems), P-delta becomes a problem that depends on the properties of individual stories. P-delta effects reduce the effective resistance of each story by an amount approximately equal to $P_i\delta_i/h_i$*, where* P_i*,* δ_i*, and* h_i *are the sum of vertical forces, interstory deflection, and height, respectively, of story i. Thus, large P-delta effects, which may lead to an effective negative story stiffness at large displacements, are caused by either large vertical story forces (lower stories) or large story drifts.*

Work by Krawinkler (ref) examined the base shear versus roof drift angle (roof displacement over structure height) response of a three story structure, using a basic centerline model (Model M1, discussed later). Responses with and without P-delta effects were examined. When P-delta is ignored, the response maintains a hardening stiffness even at very large drifts (3% strain hardening is assumed in the element models). When P-delta is included, the structural response changes radically, exhibiting only a short strength plateau followed by a rapid decrease in resistance (negative stiffness) and a complete loss of lateral resistance at the relatively small global drift of 4%. This global force- displacement behavior is alarming, but it does not provide much insight into P-delta since this phenomenon is controlled by story properties.

The negative post-mechanism stiffness of the bottom five stories of a 9 story building examined by Krawinkler (ref) is about the same and is approximately equal to -6% of the elastic story stiffness. This negative stiffness arises because the $P\delta/h$ *"shear" counteracts the 3% strain hardening that would exist without P-delta. This research implies that the* *structure would collapse in an earthquake because of complete loss in lateral load resistance if in any of the five bottom stories the drift approaches 16%. A similar conclusion cannot be drawn for the upper stories which show a very small drift at zero lateral resistance. These stories recover effective stiffness as the structure is being pushed to larger displacements because of their smaller P-delta effect. Thus, as the displacements are being increased in the negative stiffness range, the lower stories drift at a much higher rate and contribute more and more to the total structure drift. Deflected shapes of the structure as it is pushed under the given load pattern to the maximum global drift of 0.04 radians constitutes an instability condition at which the structure is at incipient collapse under gravity loads alone because of P-delta effects.*

The amplification of drift in the lower stories and the de-amplification in the upper stories, as the structure is being pushed to larger displacements, shows ratios of story drift angle to roof drift angle, plotted against roof drift angle, for all 20 stories. These curves show that in the elastic range all story drifts are about equal, but that great differences in drifts exist in the inelastic range. The rapid increase in drift in stories 1 to 5 is evident. At very large drifts the contributions of the upper stories to the deflection become negligible.

It needs to be noted that the contributions of the individual stories to drift depend on the load pattern selected in the pushover analysis. In this study the NEHRP'94 (FEMA-222A, 1994) design load pattern with k = 2.0 is selected. Drastic changes in the presented results are not expected if different load patterns would have been chosen. From a design perspective it is critical to understand the behavior characteristics from the pushover analysis in order to evaluate the importance of P-delta.

For steel moment frame structures in which member buckling is prevented, incremental sidesway collapse due to structure P-delta is the predominant global collapse mode. The P-delta problem is not adequately addressed in present codes. The utilization of an elastic stability coefficient θ*, such as the one used in the NEHRP'94 provisions [*$\theta = P\delta/(Vh)$*], provides little protection against the occurrence of a negative post-mechanism stiffness and against excessive drifting of the seismic response.*

Because of the potential importance of P-delta effects on the seismic response of flexible SMRF structures it is imperative in all cases to consider these effects in a nonlinear time history analysis. If two-dimensional analytical models are used it is customary to represent only moment resisting frames and ignore the presence of frames with simple (shear) connections. However, what cannot be ignored is the fact that the *moment resisting frames have to resist the P-delta effects caused by vertical loads tributary to the frames with simple connections. One simple way of including these effects is to add an elastic "P-delta column" to the 2-D model, which is loaded with all the vertical loads tributary to the simple frames. This column should have negligible bending stiffness so it can take on the deflected shape of the moment frames without attracting bending moments.*

5.9.6.1 Static P-Δ Effects

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

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \tag{5-19}$$

where:

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

V_i = Total calculated lateral shear force in the direction under consideration at story *i* due to earthquake response, assuming that the structure remains elastic.

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.

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

In any story in which θ_i is less than or equal to 0.1, the structure need not be investigated further for stability concerns. When the quantity θ_i in a story exceeds 0.1, the analysis of the structure should consider P-Δ effects. When the value of θ_i exceeds 0.33, the structure should be considered potentially unstable.

This process is iterative. For linear procedures, δ_i should be increased by 1/(1-θ) for evaluation of the stability coefficient.

Commentary: For a bilinear SDOF system with mass m and height h the effect of P-delta can be represented using a dimensionless parameter θ =

mg/(Kh) that can be used to describe the decrease in stiffness and strength. The elastic stiffness K is reduced to (1- θ)K, and the post-elastic stiffness α'K is reduced to (α' - θ)K. In this formulation α' is the strain hardening ratio of the system without P-delta effect, and α' - θ is the strain "hardening" ratio with P-delta effects, which is denoted here as the effective strain "hardening" ratio α. If θ > α', then α becomes negative.

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.

5.9.6.2 Dynamic P-Δ Effects

Dynamic P-Δ effects may increase component actions and deformations, and story drifts. Second-order effects should be considered directly for nonlinear procedures; the geometric stiffness of all elements and components subjected to axial forces should be included in the mathematical model.

Commentary: From a dynamic perspective the structure P-delta effect may lead to a significant amplification in displacement response if α is negative and the displacement demands are high enough to enter the range of negative lateral stiffness. The dynamic response of an SDOF system whose hysteretic behavior is bilinear but includes P-delta effects can lead to a negative post-elastic stiffness αK = -0.03K. The presence of the negative stiffness leads to drifting (ratcheting) of the displacement response, which brings the SDOF system close to collapse. Research using a suite of time histories (Ref) mean values of the displacement amplification factor (displacement for α = -0.03 over displacement for α = 0.0) for different strength reduction factors R (R = elastic strength demand over yield strength) and a period range from 0 to 5.0 sec. were developed. It is evident that the displacement amplification depends strongly on the yield strength (R-factor) and the period of the SDOF system. Particularly for short period systems with low yield strength the amplification can be substantial. The diagrams are terminated at the last period of stability, i.e., for shorter periods at least one record did lead to a complete loss of lateral resistance.

5.9.7 Elastic Framing Properties

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

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

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

5.9.8 Nonlinear Framing Properties

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

5.9.9 Verification of Analysis Assumptions

Each component should be evaluated to determine that assumed locations of inelastic deformations are consistent with strength and equilibrium requirements at all locations along the component length. Further, each component should be evaluated by rational analysis for adequate 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, and the post-earthquake gravity load capacity should be investigated. Formation of flexural plastic hinges away from component ends should not be permitted unless it is explicitly accounted for in modeling and analysis.

5.9.10 Undamaged Connection Modeling

Undamaged connections shall be modeled in accordance with the following guidelines.

5.9.10.1 Fully Restrained Connections

Framing connected with typical welded fully restrained moment-resisting connections, such as shown in Figure 5-3, shall be modeled as indicated herein.

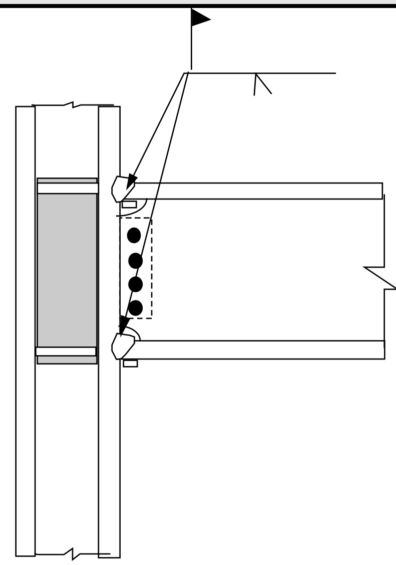


Figure 5-3 Welded Unreinforced Fully Restrained Connection (pre-1994)

5.9.10.1.1 Linear Modeling

Undamaged type FR connections should be modeled using the gross cross section properties and assuming rigid attachment between the beams and columns. Modeling may use either center line - to center line dimensions for beams and columns, or alternatively, rigid or flexible column panel zones may be modeled to offset the ends of the beams and columns from the intersection of the center lines of these members. Rigid offsets, used to represent the panel zone should not exceed 80% of the dimension of the actual panel zone. Panel zone flexibility may be directly considered by adding a panel zone element to the model.

5.9.10.1.2 Nonlinear Modeling

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

5.9.10.2 Simple Shear Tab Connections - with slabs

This section presents modeling guidelines for the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design, and when concrete slabs are present. Figure 5-4 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. A concrete floor slab, or slab on metal deck is present at the top flange of the beam.

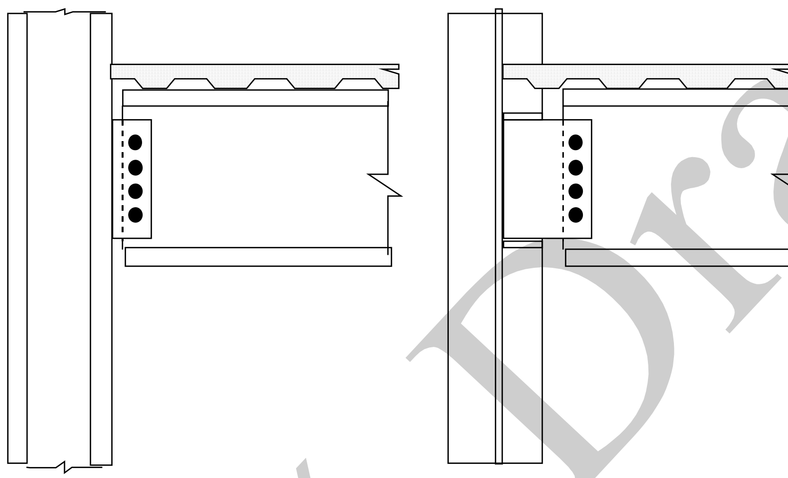


Figure 5-4 Typical Simple Shear Tab Connection with Slab

5.9.10.2.1 Modeling Guidelines - Linear Analysis

One of the following two approaches shall be used to model framing with shear tab connections when slabs are present:

a) Beams and columns connected with shear tabs shall be modeled using their full gross cross section properties. Connections of beams to columns shall be assumed to be pins.

b) Beams connected to columns shall be modeled using 5% of their gross moment of inertia, while columns shall be modeled using the full cross section properties. Framing shall be modeled center line to center line. Beam column connections shall be assumed to be fully rigid.

Commentary: The presence of gravity framing, utilizing shear tab connectors, can provide substantial sitffening to WSMF system provided as the basic lateral force resisting system. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to *bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams, which act as struts, and the diaphragms. The modeling approach suggested in "a" is adequate to determine the influence of this effect on overall structural behavior. As a secondary effect, the relatively small rigidity provided by the shear tab connections provides some additional overall frame stiffness. The modeling approach suggested in "b" is an approximate approach to including this additional stiffening in the model.*

5.9.10.2.2 Modeling Guidelines - Nonlinear Analysis

Framing connected with shear tabs, in structures with slabs present, shall be modeled using center line to center line dimensions. Figure 5-5 presents a general hysteretic model that may be used for analysis of framing with these connections.

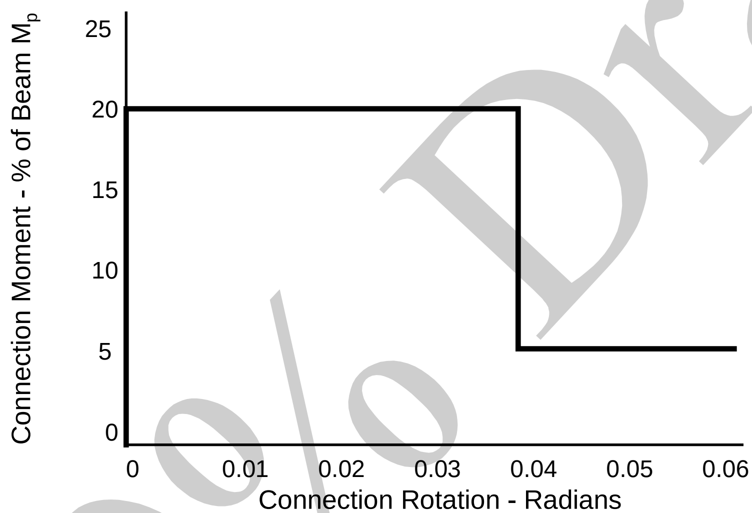
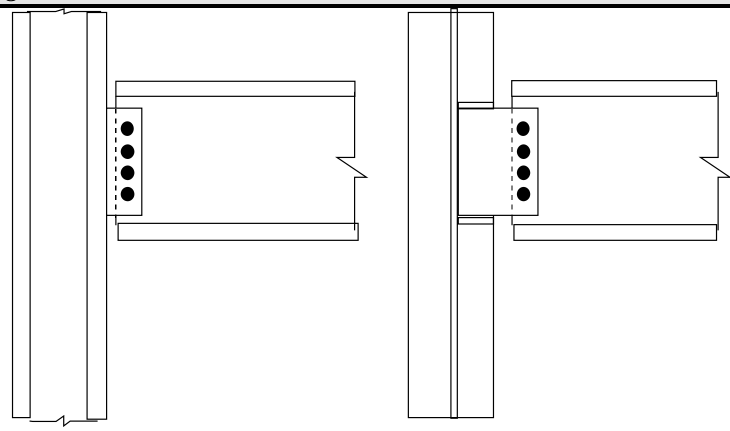


Figure 5-5 General Hysteretic Model for Shear Tab Connections with Slabs

5.9.10.3 Simple Shear Tab Connections - without slabs

This section presents modeling guidelines for the typical single plate shear tab connection commonly used to connect beams to columns for gravity loads, when moment-resistance is not required by the design and slabs are not present. Figure 5-6 presents a detail for this connection. It is characterized by rolled wide flange beams connected to either the major or minor axis of wide flange column sections. Beam webs are connected to the column with a single plate shear tab, welded to the column and bolted to the beam web. Diaphragms consist may not be present, and if present consist of wood sheathing, unfilled metal deck, or horizontal steel bracing.



Major Axis of Column Minor Axis of Column

Figure 5-6 Typical Simple Shear Tab Connection without Slab

Commentary: Shear tabs of the type shown in Figure 5-6, though not as effective in resisting frame lateral drifts as are shear tab connections when slabs are present, as discussed in the previous section, do still have the effect of coupling the deflected shapes of gravity columns to those of columns intended to participate in the lateral-force-resisting system. The connections themselves, have negligible stiffness.

5.9.10.3.1 Modeling Guidelines - Linear Analysis

Beams and columns connected with shear tabs shall be modeled using their full gross cross section properties. Connections of beams to columns shall be assumed to be pins.

Commentary: The presence of gravity framing, utilizing shear tab connectors, can provide substantial sitffening to WSMF system provided as the basic lateral force resisting system. The primary contributor to this added stiffness is the fact that the gravity load columns are constrained to bend to the same deflected shape as the columns of the moment-resisting frame, through their interconnection by the gravity beams, which act as struts. The modeling approach suggested in this section is adequate to determine the influence of this effect on overall structural behavior.

5.9.10.3.2 Modeling Guidelines - Nonlinear Analysis

Framing connected with shear tabs, in structures without slabs present, shall be including in the analytical model. Framing should be modeled using center line to center line dimensions. Framing may be assumed to be pin connected, or alternatively, beams connected to columns with shear tab connections may be assigned 5% of their actual moment of inertia.

5.9.11 Damage Modeling

This section presents guidelines for modeling various conditions of damage. In general, damage results in anisotropic frame behavior with affected framing exhibiting different hysteretic properties for loading in a positive direction, than it does for loading in the reverse direction. Except for nonlinear dynamic analyses, it is generally necessary to utilize multiple models to represent these different behaviors, with loading applied in an appropriate direction for each model.

5.9.11.1 Type FR Connection Damage

Damaged type FR connections should be modeled in accordance with the guidelines of this section. Refer to Chapter 5 for detailed descriptions of the various damage conditions.

a. Connections with any one of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6 damage at the bottom 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 shall be modeled as an undamaged simple shear tab connection with slab, per section 5.9.10.2.

b. Connections with any one of type G3, G4, G7, C2, C4, C5, W2, W3, W4, P5, or P6 damage at 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 shall be modeled as an undamaged simple shear tab connection without slab, per section 5.9.10.3.

c. 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 shall be modeled as an undamaged simple shear tab connection with slab, per Section 5.9.10.2 for loading conditions in which the fracture at the beam bottom flange is placed into tension and shall be modeled as an undamaged simple shear tab connection without slab, per Section 5.9.10.3 for loading conditions in which the fracture at the top flange is placed into tension.

d. If any of the conditions in a, b, or c above is present in combination with shear tab damage, types S1, S2, S3, S4, S5, or S6; the connection shall be modeled as an undamaged simple shear tab connection without slab, per Section 5.9.10.3 for both directions of loading.

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

f. Connections with type P9 damage and oriented as indicated in Figure 5-7 shall be modeled with the beams and columns below the fracture surface assumed to be rigidly connected. The column above the fracture surface shall be assumed to be unconnected for loading that places the column into tension and shall 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 5-7, then the instructions for "top" and "bottom" columns above should be reversed.

5.9.11.2 Column Damage

a. If a column has type C1 or C3 damage in any flange, the column shall 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.

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

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

5.9.11.3 Beam Damage

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

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

5.9.11.4 Other Damage

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

5.10 Acceptance Criteria and Confidence Evaluation

The confidence provided with regard to the damaged building's ability to resist collapse under the levels of ground shaking likely in the immediate post-earthquake period shall be determined through evaluation of the relationship:

$$\gamma_{con} = \frac{\phi C}{\lambda D} \tag{5-20}$$

where:
- ϕ = capacity reduction factor
- C = capacity
- λ = load factor
- D = computed demand

for each of the performance parameters indicated in Table 5-9. The value of γ_{con} determined for each of these performance parameters shall be used to determine a level of confidence associated with achieving the desired performance, either by reference to Table 5-10, or through direct calculation of confidence level through the procedures of Section 5.11. The lowest of the confidence levels obtained for the structure for each of the design parameters shall establish the overall confidence with regard to the structure's ability to achieve the desired performance.

Table 5-9 Performance Parameters Requiring Evaluation of Confidence

Parameter	Discussion
Inter-story Drift	The maximum inter-story drift computed for any story of the structure shall be evaluated. Refer to Section 5.10.1
Column Axial Load	The adequacy of each column to withstand the calculated maximum compressive load for that column shall be evaluated. Refer to Section 5.10.2
Column Splice Tension	The adequacy of column splices to withstand calculated maximum tensile demands for the column shall be evaluated. Refer to Section 5.10.3
Beam-column Connections	The adequacy of individual beam-column connections to withstand induced inter-story drift demands shall be evaluated. Refer to Section 5.10.4

Table 5-10 Confidence Level as a Function of the Parameter γ_{con}

Confidence Level - %	50	65	84	90	95
Linear Static	.2	.5	1	1.2	1.5
Linear Dynamic	.3	.6	1.1	1.3	1.5
Nonlinear Static	.4	.7	1.1	1.3	1.5
Nonlinear Dynamic	.5	.8	1.2	1.3	1.5

Commentary: The process of predicting performance for a structure inherently incorporates a significant degree of uncertainty. This uncertainty may be ascribed to a number of factors including inaccuracies in our modeling and analysis approaches, our lack of knowledge with regard to the construction quality, strength and damping inherent in the building; inability to precisely predict the amount of dead and live load present and other similar factors. In addition, the precise character of the ground motion that will affect the structure and the capacity of the structure to resist the resulting response can not be precisely predicted, nor do we completely understand the factors that affect the apparent variation in these parameters.

Even though it is not possible to precisely predict all of these parameters, it is possible to estimate bounds for each of these, to develop an understanding of the effect of these uncertain and apparently random parameters on the behavior and performance of the structure, and to estimate probabilistic distributions of the likely performance of the structure, considering these bounds, using methods of structural reliability.

The load factors, λ, and capacity reduction factors, ϕ, have been calculated by assuming that the effects of these random and uncertain parameters result in a log normal distribution of response (inter-story drift, member forces) and capacity. The standard deviations for these distributions have been estimated based on statistical distributions of data obtained from laboratory testing of typical beam-column assemblies, analytical evaluations of building structures, and by judgment.

The load factors, λ, include a component that accounts for the statistical distribution of response, given the distribution of random and uncertain response, as well as the bias inherent in the analytical technique used to predict the response parameters. The resistance factors, ϕ, account for the variation and uncertainty inherent in the prediction of *capacity. When the factored demand, λD is exactly equal to the factored capacity, ϕC, then this indicates that given the level of knowledge available with regard to the behavior of the building, there is mean level of confidence that the building will meet the performance being analyzed, in the case of post-earthquake assessments, a state of incipient collapse.*

If greater knowledge can be obtained with regard to the probable behavior of the building, for example through performing more rigorous quality assurance during construction or by performing more rigorous and accurate analytical evaluations of the building, then the uncertainty associated with both the prediction of the building's response and the ability of the building to withstand this response without exceeding the specific performance goal, is reduced. This reduction in uncertainty can be expressed as a reduction in the standard deviations of the distribution of possible response and capacity states of the building. As the uncertainty in response prediction is reduced, for example through the use of more accurate modeling and analytical methods, the load factors associated with the prediction of mean values of response parameters at the desired probability of exceedance may be reduced. Thus, as reflected in Table 5-4, the load factors associated with nonlinear analysis approaches are generally lower than those associated with the linear approaches.

As used in these Guidelines, confidence reflects the extent to which the uncertain parameters that affect performance prediction are understood. A high level of confidence is attained when there is a high level of certainty that the desired performance will be attained, while a low level of confidence reflects a significant degree of uncertainty with regard to the ability of the structure to provide the desired performance. The extent of certainty inherent in the performance prediction, and consequently the level of confidence associated with a building's ability to provide specified performance is indexed to the γ_{con} parameter.

A calculated value of γ_{con} of 1.0 indicates a mean level of confidence of achieving the desired performance at the target annual probability of exceedance. Since it is assumed that performance is log normally distributed with regard to the uncertain parameters, a mean level of confidence is actually somewhat higher than a 50% certainty of being able to achieve the desired performance, approximately on the order of 70% confidence. Values of γ_{con} that exceed 1.0 indicate more certain performance and values less than 1.0, less certain performance.

γ_{con} is calculated as a function of the standard deviation of the log of the uncertain parameters and as a function of the hazard curve for the site

itself. The tabulated values of inter-story drift capacity, resistance factors and confidence parameters contained in this section are based on the study of typical buildings, and the use of a presumed hazard function in the immediate post-earthquake environment as described in Section 5.6. Section 5.11 presents a detailed procedure for calculating the capacity for inter-story drift for various performance levels, the resistance factor associated with that capacity and the confidence parameter, γ_{con}. The more detailed procedures of Section 3.7 may be used, when warranted, to reduce the uncertainty inherent in performance prediction and potentially obtain more optimistic estimates of probable performance.

5.10.1 Interstory Drift Capacity - Global Stability

Factored inter-story drift capacity, ϕC, to maintain global stability shall be taken as the product of the resistance factor ϕ and capacity C, obtained from Table 5-10. In lieu of the values contained in Table 5-10, the more detailed procedures of Section 5.11 may be used to determine inter-story drift capacity as limited by global building response.

Table 5-10 Inter-story Drift Capacity and as Limited By Global Response, and Associated Resistance Factors

Structure Type	Inter-story Drift Capacity	Resistance Factor ϕ
Low Rise -(3 above grade stories or less)	.10	.6
Mid Rise - (4 or more above grade stories, but not more than 12 above grade stories)	.08	.6
High Rise - More than 12 above grade stories	.05	.6

5.10.2 Column Compressive Capacity

The capacity of each individual column to resist compressive axial loads shall be determined as the product of the resistance factor, ϕ, and the compressive strength of the column as determined in accordance with the AISC Load and Resistance Factor Design Specification. For the purposes of this evaluation, ϕ shall be assigned a value of 0.7.

5.10.3 Column Splice Capacity

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

Specification. For the purposes of this evaluation, ϕ shall be assigned a value of 0.7. The tensile strength of partial penetration welded splices shall be determined from the equation:

$$x=a+b \tag{5-21}$$

5.10.4 Beam-Column Connection Capacity

The capacity of individual beam-column connections to resist interstory drift demands without loss of ability to resist gravity loads shall be taken as the product of the resistance factor, ϕ, and capacity C obtained from Table 5-11.

Table 5-11 Drift Capacity of Various Connections

Connection Type	ϕ	C - radians
Type FR, welded unreinforced moment-resisting connection, typical of pre-Northridge practice	0.6	0.02
Single plate shear tab connection with slab	0.9	0.15
Single plate shear tab connection without slab	0.9	015

5.11 Detailed Procedure for Determination Confidence

This section provides detailed procedures for determination of the global inter-story drift capacity of a structure, δ, associated resistance factor ϕ and confidence index, γ_{con}. These more detailed procedures may be used as an alternative to the acceptance criteria of Section 5.10, when more certain estimates of structural performance are desired. Steps involved in the procedures include the following:

- Determination of hazard parameters, in accordance with Section 5.11.1
- Development of a suite of ground motion accelerograms in accordance with Section 5.11.2
- Performance of a suite of dynamic pushover analyses in accordance with Section 5.11.3
- Calculation of factored drift capacity in accordance with Section 5.11.4
- Calculation of confidence index, γ_{con}, and inherent confidence in building performance, in accordance with Section 5.11.5

5.11.1 Hazard Parameters

A 5% damped, elastic response spectrum for the ground shaking that caused the initial damage, at the fundamental period of the damaged structure shall be estimated in accordance with the guidelines of Section 5.7.2.

5.11.2 Ground Motion Accelerograms

A suite of at least 10 ground motion accelerograms shall be developed that are compatible with the 5% damped response spectrum for the site, determined in accordance with Section 5.11.1. The accelerograms shall be scaled to achieve spectral compatibility in accordance with the guidelines of FEMA-273.

5.11.3 Dynamic Pushover Analysis

A nonlinear mathematical model of the damaged building shall be constructed, in accordeance with the modeling rules of Section 5.9. The model shall realistically model the material and geometric nonlinearities that may occur in the structure under large lateral response, including P-Δ effects, panel zone flexibility, if significant, and hysteretic behavior of beam-column connections. The stiffness of beam-column frames, not intended to participate in lateral force resistance shall also be included in the model. Equivalent viscous damping shall be taken as 3%.

For each ground motion, developed in accordance with Section 5.11.3, a dynamic pushover analysis shall be conducted, using the following procedure:

1. The ground motion shall be scaled to an index, spectral response acceleration at the fundamental period of the structure, that produces elastic response.

2. A response history analysis of the structure, for response to this ground motion shall be performed. The maximum inter-story drift obtained from the analysis shall be recorded.

3. The amplitude of the ground motion used in the analysis of step 2 shall be scaled to 110% of the amplitude used in that analysis.

4. Steps 2 and 3 shall be repeated, with the maximum inter-story drift predicted by each successive analysis recorded, until either the structure is predicted to collapse by the analysis or maximum inter-story drift predicted by the analysis exceeds 10%.

5. A plot of the index spectral response acceleration at the structure's fundamental period for each of the analyses and the maximum interstory drift obtained from the analysis shall be created. Such a plot, illustrated in Figure 5-7, is termed a dynamic pushover plot.

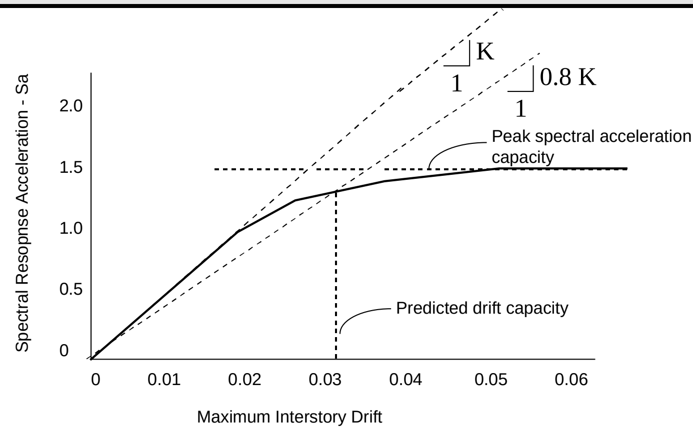


Figure 5-7 Dynamic Pushover Curve

6. The slope of the initial portion of the dynamic pushover plot shall be noted. A line shall be constructed from the origin of the dynamic pushover plot and having a slope of 80% of the slope of the initial portion of the dynamic pushover plot. The inter-story drift at the intersection of this line, having 80% of the slope of the initial portion of the curve, and the pushover curve itself, shall be taken as the inter-story drift capacity of the structure, for this ground motion. Refer to Figure 5-7. The inter-story drift capacity shall not be taken as greater than 0.1.

5.11.4 Determination of Factored Interstory Drift Capacity

The inter-story drift capacities δ_i, determined from each of the dynamic pushover analyses shall be tabulated, together with the natural logarithm of these inter-story drift capacities, $ln(\delta_i)$. The median value of the δ_i statistics shall be determined, as shall the standard deviation, $\sigma_{ln\delta}$ of the natural logarithms of the inter-story drift capacities. A resistance factor, ϕ, shall be determined from the equation:

$$\phi = e^{-k\sigma_{\ln\delta}^2 / 2b} \tag{5-22}$$

where: k = the slope of the hazard curve, plotted in ln-ln coordinates, during the immediate post-earthquake period, which may be taken as having a value of 4.0

b = a hazard parameter that may be taken as having a value of 1.0

$\sigma_{\ln\delta}$ = the standard deviation of the natural logarithms of the predicted inter-story drifts obtained from the pushover anlayses

Factored inter-story drift demand for global response shall be taken as the product of ϕ determined in accordance with equation 5-22 and the median inter-story drift capacity determined from the dynamic pushover analyses.

5.11.5 Determination of Confidence Level

A performance confidence index, γ_{con}, shall be determined in accordance with Section 5.10, for each of the controlling performance parameters indicated in Table 5-9. The confidence parameter K_x, shall be determined from the equation, using the smallest of the values γ_{con}:

$$K_x = \frac{\ln(\gamma_{con})}{b\sigma_{UT}} + \frac{k\sigma_{UT}}{2} \qquad (5\text{-}23)$$

where: k, and b are the hazard parameters defined in Section 5.11.4.
σ_{UT} = is a measure of the uncertainty related to prediction of drift demand, taken from Table 5-12.

Table 5-12 Uncertainty Measures for Different Analytical Procedures

Analytical Procedure	σ_{UT}
Linear Static Procedure	0.6
Linear Dynamic Procedure	0.7
Nonlinear Static Procedure	0.8
Nonlinear Dynamic Procedure	0.9

The level of confidence with regard to the target performance shall be determined by interpolation from, Table 5-13.

Table 5-13 - Values of K_x for Various Levels of Confidence

Confidence Level	K_X
65%	0
84%	1
90%	1.3
95%	1.6

5.12 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 WSMF building, this report should also be submitted to the building official. This report should directly, or by attached references, document the inspection program that was performed, and provide an interpretation of the results of the inspection program and a general recommendation as to appropriate repair and occupancy strategies. The report should include but not be limited to the following material:

- Building Address
- A narrative description of the building, indicating plan dimensions, number of stories, total square feet, occupancy, 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 (SMAW, FCAW, etc.) 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 documentation of all instructions to the inspectors, and of the signed inspection forms for each individual inspected connection.
- A description, including engineering sketches, of the observed damage to the structure as a whole (e.g., permanent drift) as well as at each connection, keyed to the damage types in Chapter 5, photographs should be included for all connections with significant visible damage.
- Calculations demonstrating the determination of a confidence level with regard to the building's ability to resist collapse in the immediate post-earthquake period.
- A summary of the recommended actions (repair and modification measures and 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 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, as an attachment, and should bear the seal of the structural engineer in charge of the evaluation.

Commentary: Following completion of the detailed damage assessments, the structural engineer should prepare a written report. The report should include identification of any potentially hazardous conditions which were observed, including 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, visual inspection and NDT records, data sheets, and reports as attachments.

The nature and scope of the evaluations performed should be clearly stated. 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.

5.13 Qualified Independent Engineering Review

Independent third party review, by qualified professionals, is recommended throughout these Guidelines when alternative approaches to evaluation or design are taken, or where approaches requiring high degrees of structural engineering knowledge and judgment are taken. Specifically, it is recommended that qualified engineering review be provided where:

- the level of confidence that the building can resist collapse is less than 50% and the engineer has determined that an unsafe condition does not exist.
- Where an engineer has decided not to repair damage otherwise recommended to be repaired by these Guidelines.
- When any story of the building has experienced a permanent lateral drift exceeding 1% of the story height and proposed repairs do not correct this condition.
- When nonlinear dynamic analyses are performed as part of the evaluation.

Where independent review is recommended, the analysis and/or design should be subjected to an independent and objective technical review by a knowledgeable reviewer experienced in the design, analysis, and structural performance issues involved. The reviewer should examine the available information on the condition of the building, the basic engineering and reliability concepts, and the recommendations for proposed action.

Commentary: The independent reviewer may be one or more persons whose collective experience spans the technical issues anticipated in the work. When more than one person is collectively performing the independent review, one of these should be designated the review chair and should act on behalf of the team in presenting conclusions or recommendations.

Independent third party review is not a substitute for plan checking. It is intended to provide the structural engineer of record with an independent opinion, by a qualified expert, on the adequacy of structural engineering decisions and approaches. The seismic behavior of WSMF structures is now understood to be an extremely complex issue. Proper understanding of the problem requires knowledge of structural mechanics, metallurgy, welding, fracture mechanics, earthquake engineering, and statistics. Due to our limited current state of knowledge, even professionals who possess such knowledge face considerable uncertainty in making design judgments. Third party review should only be performed by qualified individuals.

5.13.1 Timing of Independent Review

The independent reviewer(s) should be selected prior to the initiation of substantial portions of the design and/or analysis work that is to be reviewed, and coordination of the review should start as soon as sufficient information to define the project is available.

5.13.2 Qualifications and Terms of Employment

The reviewer should have no other involvement in the project before, during, or after the review. The reviewer should be selected and paid by the owner and should have an equal or higher level of technical expertise in the issues involved than the structural engineer-of-record. The reviewer (or in the case of peer review teams, the review chair) should be a structural engineer who is familiar with governing regulations for the work being reviewed. The reviewer should serve through completion of the project and should not be terminated except for failure to perform the duties specified herein. Such termination should be in writing with copies delivered to the building official, owner, and structural engineer-of-record.

5.13.3 Scope of Review

Review activities related to evaluation of the safety condition of a building should include a review of available construction documents for the building, all inspection and testing reports, any analyses prepared by the structural engineer of record, the method of connection sample selection, and visual observation of the condition of the structure, as well as review of any mathematical models and analyses performed as part of the post-earthquake evaluation. Review should include consideration of the proposed design approach, methods, materials, and details.

5.13.4 Reports

The reviewer should prepare a written report to the owner and building official that covers all aspects of the structural engineering review performed, including conclusions reached by the reviewer. Such reports should include statements on the following:

- Scope of engineering review performed with limitations defined.
- The status of the project documents at each review stage.
- Ability of selected materials and framing systems to meet performance criteria with given loads and configuration.
- Degree of structural system redundancy, ductility, and compatibility, particularly in relation to lateral forces.
- Basic constructability of structural members and connections (or repairs and modifications of these elements).
- Other recommendations that would be appropriate to the specific project.
- Presentation of the conclusions of the reviewer identifying any areas which need further review, investigation, and/or clarifications.

5.13.5 Responses and Corrective Actions

The structural engineer-of-record should review the report from the reviewer and develop corrective actions and other responses as appropriate. Changes during the construction/field phases that affect the seismic resistance system should be reported to the reviewer in writing for action and recommendations.

5.13.6 Distribution of Reports

All reports, responses, and corrective actions prepared pursuant to this section should be submitted to the building official and the owner along with other plans, specifications, and calculations required. If the reviewer is terminated by the owner prior to completion of the project, then all reports prepared by the reviewer, prior to such termination, should be submitted to the building official, the owner, and the structural engineer-of-record within ten (10) working days of such termination.

5.13.7 Engineer-of-Record

The structural engineer-of-record should retain the full responsibility for the structural design as outlined in professional practice laws and regulations. The independent review engineer(s) should not be asked to or be expected to assume any responsibility for the structural evaluation or subsequent repair designs.

5.13.8 Resolution of Differences

If the structural engineer-of-record does not agree with the recommendations of the reviewer, then such differences should be resolved by the building official in the manner specified in the applicable Building Code.

6. POST-EARTHQUAKE REPAIR

6.1 Scope

This section provides criteria for structural repair of earthquake damage. *Repair* constitutes any measure(s) taken to restore earthquake damaged elements of the building, including individual members or their connections, or the building as whole, to their original configuration, strength, stiffness and deformation capacity. It does not include routine correction of non-conforming conditions resulting from the original fabrication or upgrades intended to result in improvement in future seismic performance of the building.

Guidelines for acceptable methods of repair are provided in Sections 6.2 through 6.3 below. These Guidelines 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. Guidelines for assessment of the seismic performance capability of existing buildings and upgrade of building to improve performance capability may be found in a companion publication, *FEMA-XXX Evaluation and Upgrade Criteria for Welded Steel Moment-Resisting Structures*.

Commentary: Based on the observed behavior of actual buildings in the Northridge Earthquake, as well as recent test data, WSMF structures constructed with the typical detailing and construction practice prevalent prior to1994 do not have the same 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 excessive construction defects, this risk is higher still.

Based on (limited) testing, it is believed that the repair recommendations contained in these Guidelines can be effective in restoring a building's pre-earthquake condition, and to the extent that workmanship and materials of repair work is improved relative 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. Retrofit can consist of local reinforcement of individual moment connections as well as alteration of the basic lateral-force-resisting characteristics of the structure through addition of braced frames, shear walls, base isolation, energy dissipation devices, etc. Performance evaluation and structural retrofit are beyond the scope of these Guidelines. Criteria for performance evaluation and structural retrofit may be found in a companion *document, Upgrade and Evaluation Criteria for Existing Welded Steel Moment Frame Structures, FEMA-XXX.*

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 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 4 is one method of confirming the building's global structural stability. 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 which 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 to the structural engineer for review.

6.3 Repair Details

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

Commentary: Examples of repair details are provided for some classes of damage, based on approaches successfully performed in the field following the 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 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, these guidelines suggest much greater specification of means and methods than is common in new construction. Although it is recommended that the engineer provide such specification as part of the construction documents, the engineers 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:

a) replacement of damaged portions of base metal (i.e. column and beam section),

b) replacement of damaged connection elements,

c) replacement of connection welds, or

d) repairs to portions of any of the aforementioned components.

Any or all of these techniques may be appropriate. The approach(s) 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.

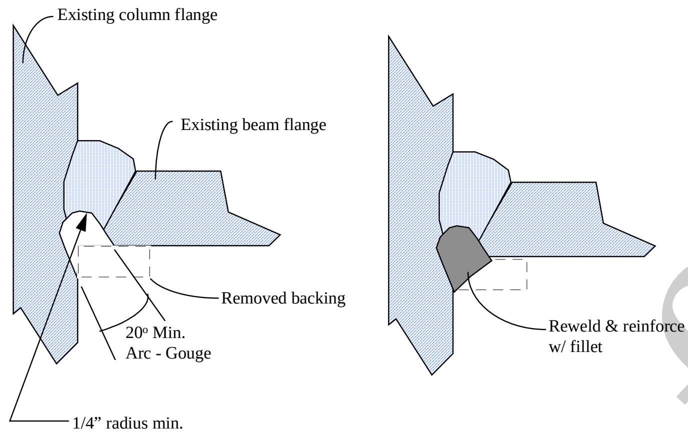
6.3.2 Weld Fractures - Type W Damage

All fractures and rejectable defects found in weld material, either between girder and column or between connection element and structural member, should have sufficient material removed to completely eliminate any discontinuity or defect. NDT should be used to determine the extent of fracture or defect 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 backgouging to eliminate unsound weld material and replacing 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 the Welding Procedure Specification (WPS) and 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 backgouged to sound material, rewelded and a reinforcing fillet added.

The structural engineer is cautioned to observe the provisions of AISC regarding intermixing of weld metals deposited by different weld processes (see AISC LRFD Manual of Steel Construction, second edition, page 6-77, and AISC ASD Steel Construction Manual, ninth edition, page 5-69). As an example, E7018 shielded metal arc welding (SMAW) stick electrodes should not be used to weld over self-shielded flux cored arc welding (FCAW-S) deposits. 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

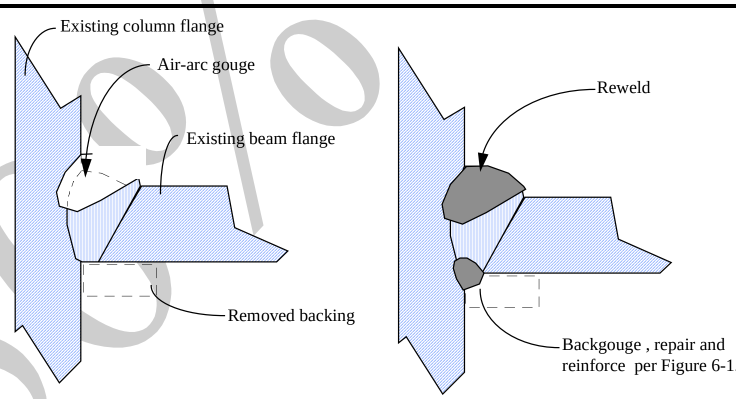
Damage or Defect Class	Figure
Rejectable defects at weld root	Figure 6-1, Figure 6-2
W2	Figure 6-3
W3	Figure 6-3
W4	Figure 6-3
W5	Figure 6-3



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 1/4.î
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.

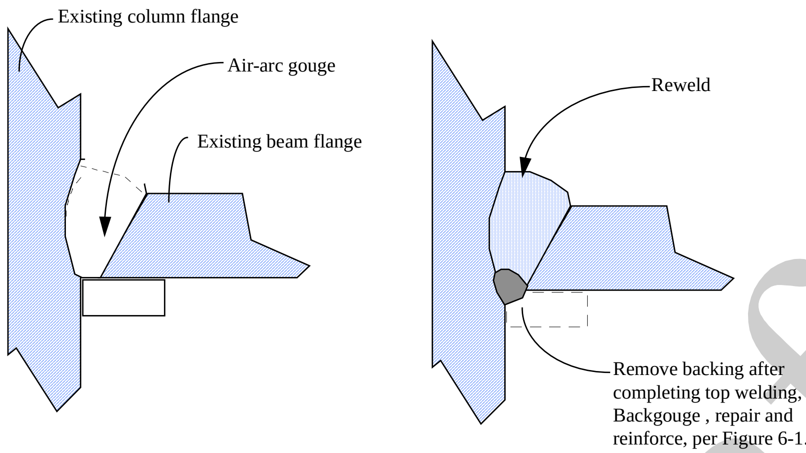
Figure 6-1 - Gouge & Re-weld of Root Defect or Damage



Notes:

1. Remove the entire fracture plus 1/8” of sound metal beyond each end.
2. For additional notes, refer to Figure 5-1

Figure 6-2 - Gouge & Re-weld of Fractured Weld



For notes see Figure 6-1 and 6-2.

Figure 6-3 - Backgouge and Reweld repair

Commentary: 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 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, 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. Alternately, intermixing of FCAW-S and other processes could be permitted provided the subsequent composition is demonstrated to meet material specification requirements.

The guidelines of 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 low toughness material.*

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, as this affects who may be responsible for costs related to the repair of such conditions. Proper observation by knowledgable 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 upset. It is almost impossible to distinguish such cracks from those caused by an earthquake, except through detailed examination of the fracture surface for evidence of oxidation or other signs of age. 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. 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 - Type C1 - C5 and P1 - 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, MT and 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. Tables 6-2 and 6-3 indicate 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 for repair of girders, or if access if restricted, as an alternative beam repair method.

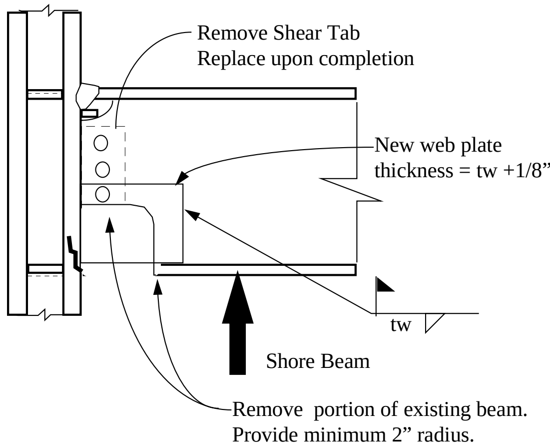
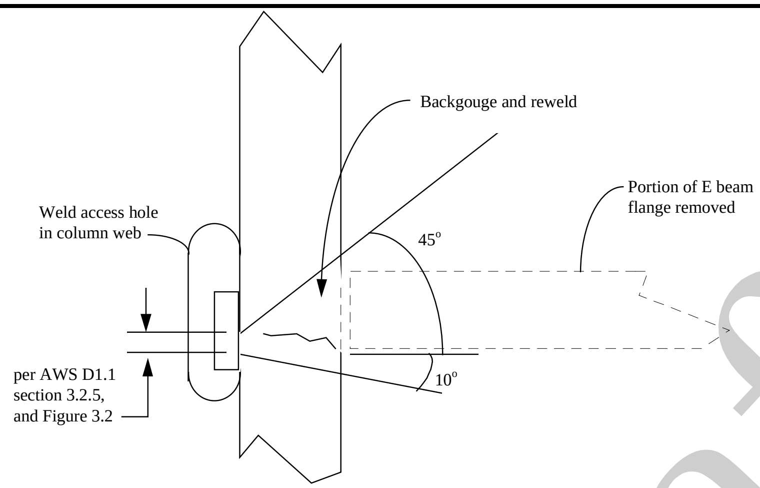


Figure 6-4 - Temporary Removal of Beam Section for Access

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

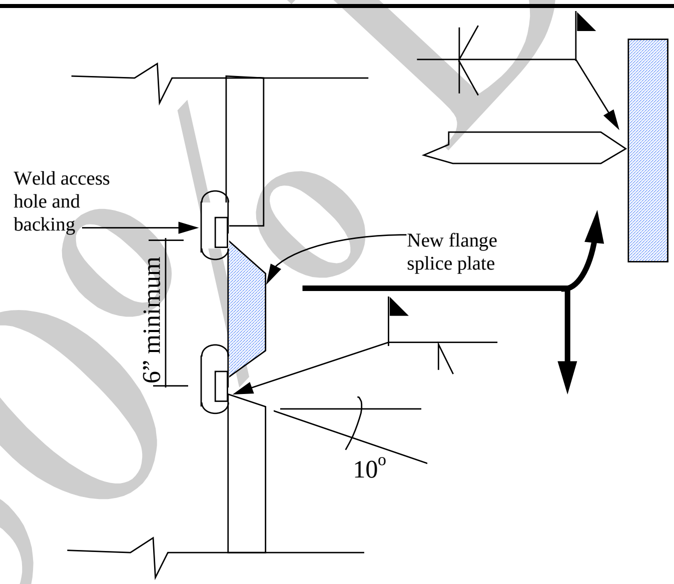
Table 6-2 - Reference Details for Type C and P Damage

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



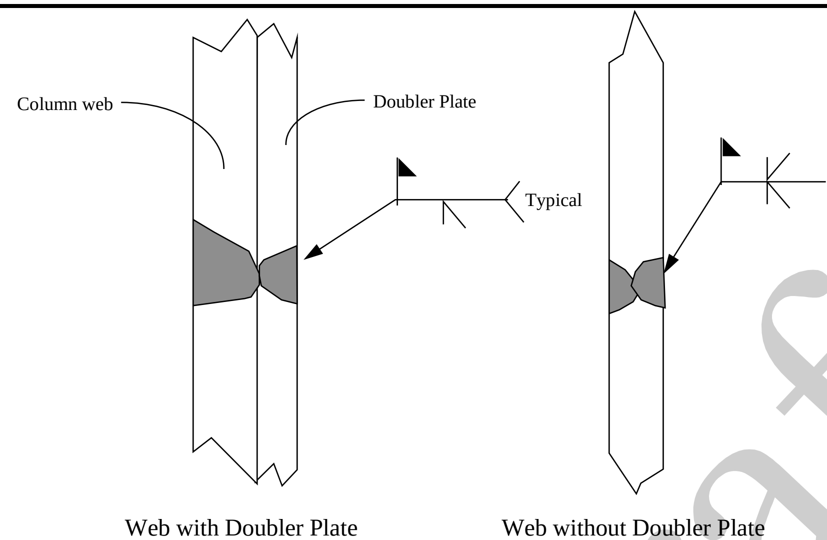
1 Investigate extent of fracture by UT to confirm that fractures are contained with the 45 degree angle zone of a standard pre-qualified CJP groove weld as defined by AWS D1.1, Figure 2.4, Joint Designation B-U4a-G
2 Provide 10^{o} bevel on lower flange plate, to channel slag out of joint.
3 Grind all 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-/+1/4. The welding should be sequenced to connect the column flange to new flange plate welds prior to welding the column web to new flange plate. Bevel the lower edge of the column flange, and upper edge of the splice plate down 10^{o}, to channel slag out of joint.

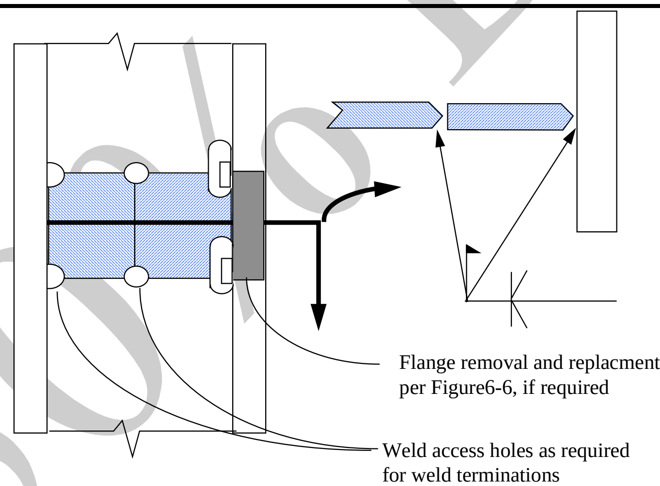
Figure 6-6 - Replacement of Column Flange Repair



Notes:

1. Prepare fractured section of doubler by air-arc gouging, grind and reweld, using web as backing
2. Prepare fractured section of web by air-arc gouging, grind and reweld, 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,-/+1/8î).

Figure 6-8 - Alternate Column Web Repair - Columns without Doubler Plates

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.*

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.

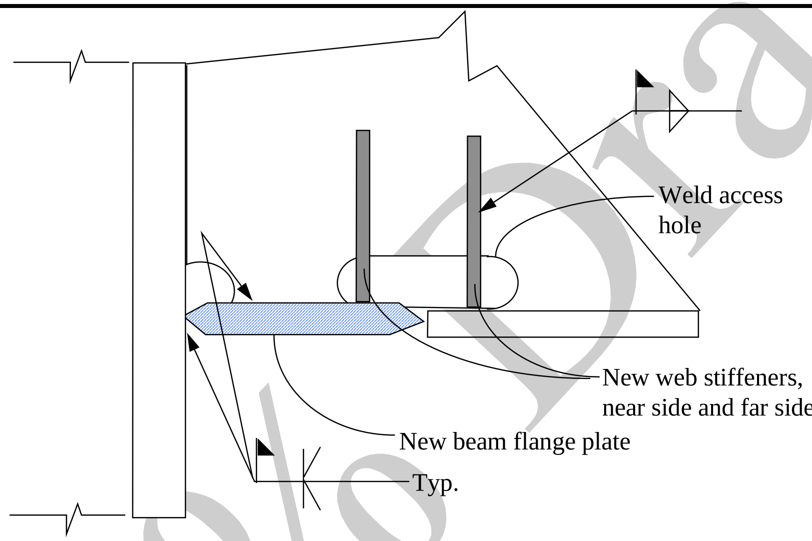
6.3.5 Girder Flange Fractures - Type G3-G5

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 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 backup bars, consequently also eliminating the removal of these backup bars. A suggested joint detail is a B-U3/TC-U5, per AWS D1.1, with 1/3 t_{flange}-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 butt weld. The weld between the splice plate and the column flange should be completed first. If a double bevel weld is selected, the welder may choose to weld the first few passes from one face, then backgouge and weld from the second side. This may help to keep the interpass temperature below the maximum without down time often encountered in waiting for the weld to cool.



Notes:

1. New plate thickness to match beam flange thickness + height of removed web fillet.
2. Weld sequence - a) weld of new flange plate to column; b)weld of flange plate stiffeners to web and flange plate; c) weld of new flange plate to beam flange. d)weld of stiffener plate to beam flange and web

Figure 6-9 - Beam Flange Plate Replacement

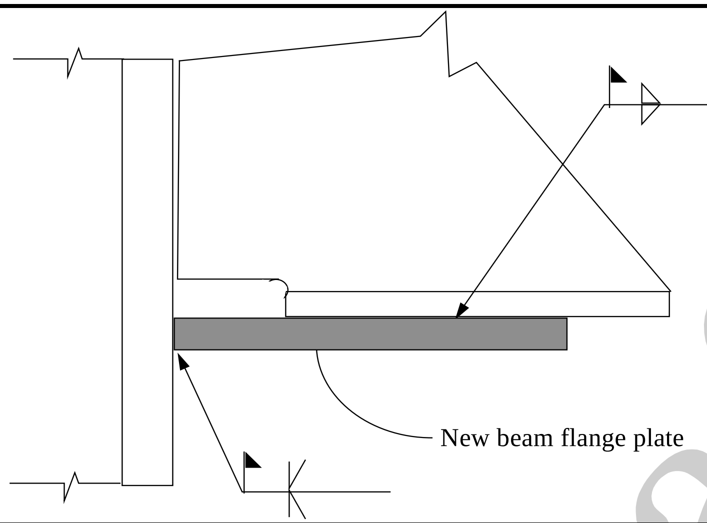
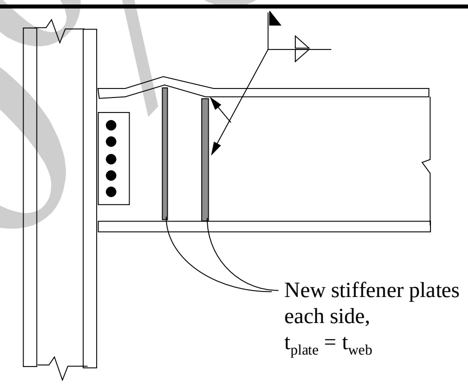


Figure 6-10 - Alternative Beam Flange Plate Replacement

6.3.6 Buckled Girder Flanges - Type G1

Where the top or bottom flange of a girder has buckled, and the rotation between the flange and web is less than or equal to the mill rolling tolerance given in the AISC Manual of Steel Construction (AISC-1994 or AISC-1989) 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 flange and replacing it with material with yield properties similar to the actual yield properties of the damaged material similar to Figure 5-6. 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), (Avent - 1995), (Shonafelt and Horn - 1984)

6.3.8 Gravity connections

Connections not part of the lateral load-resisting system may also be found to require repair due to excessive rotation or demand caused by distress of the lateral load-resisting system in the zone of influence. 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 load resisting system should be repaired in a similar manner, with special consideration given to the nature and significance of the overall structural damage. In buildings that are repaired, but not modified, future earthquakes may cause moment connection failures with resulting large building deflections and high rotation demands at gravity connections. When repairing gravity connections, consideration should be given to providing connections with the ability to rotate with little or no reduction in vertical load carrying capacity, possibly by dissipating energy (through the use of slip critical bolts with horizontal short slotted holes).

Commentary: In many cases, shear connections which were not a part of the lateral-force-resisting system provided an unanticipated redundancy after damage occurred to the primary WSMF lateral system. While repair details *could provide for rotation to minimize damage, such details should not eliminate the beneficial effect of the extra strength and stiffness these shear connections provide. This is especially important in framing systems with low moment frame redundancy.*

The suggestion of providing gravity connections with slotted holes and slip critical bolts may be a reasonable compromise. Such a connection would be capable of providing some additional, unintended, strength and stiffness for the building but would also be able to withstand relatively large rotations without jeopardizing the gravity support the connection is actually intended to provide.

6.3.9 Reuse of Bolts

Bolts in a connection displaying bolt damage or plate slippage should not be re-used. As indicated in the AISC Specification for Structural Joints using ASTM A325 or A490 Bolts (American Institute of Steel Construction - 1985), A490 bolts and galvanized A325 bolts should not be retightened and re-used under any circumstances. Other A325 bolts may be reused if determined to be in good condition. Touching up or retightening 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. Bolts in connections displaying bolt or plate slippage should not be reused.

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 A-490 bolts and galvanized ASTM A-325 bolts become slightly damaged when tightened, and consequently, should not be reused. To determine if an ungalvanized ASTM A-325 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 Specification

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

6.4 Preparation

6.4.1 Welding Procedure Specifications

A separate Welding Procedure Specification (WPS) should be established for every different weld configuration, welding position, and material specification. Two categories of qualified welding procedures are given in AWS D1.1-96. The WPS should be reviewed by the structural engineer responsible for the repairs. The WPS is a set of focused instructions to the welders and inspectors stating how the welding is to be accomplished. Each type of weld should have its own WPS solely for the purpose of that weld. The WPS should include instructions for joint preparation based on material property and thickness, as well as welding parameters. Weld process, electrode type, diameter, stick-out, voltage, current, and interpass temperature should be clearly defined. In addition, joint preheat and postheat requirements should be specified as appropriate, including insulation guidelines if applicable. The WPS should also list appropriate interim specification requirements that are mandated by the project specification.

Commentary: Preparation of the WPS is normally the responsibility of the fabricator/erector. Sample formats for WPS preparation and submission are included in AWS D1.1. Some contractors fill out the WPS by inserting references to the various AWS D1.1 tables rather than the actual data. This does not meet the intent of the WPS, which is to provide specific instructions to the welder and inspector on how the weld is to be performed. The actual values of the parameters to be used should be included in the WPS submittal.

6.4.2 Welder Training

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

6.4.3 Welder Qualifications

Welders must be qualified and capable of successfully making the repair welds required. All welders should be qualified to the AWS D1.1 requirements for the particular welding process and position in which the welding is to be performed. Successful qualification to these requirements, however, does not automatically demonstrate a welder's ability to make repair welds for all the configurations that may be encountered. Specific additional training and/or experience may be required for repair situations. Welders performing repairs should have a minimum of two years of verifiable field experience for the welding process that is employed, as well as experience in arc-gouging and thermal cutting of material. Inexperienced welders should demonstrate their ability to make proper repair welds. This may be done by welding on a mock-up assembly (see Section 5.4.4) 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, start to finish, during the construction of at least the first weld repair. This observation should be made by a qualified welding inspector or Welding Engineer.

6.4.4 Joint Mock-ups

A joint mock-up should be considered as a training and qualification tool for each type of repair the welder is to perform that is more challenging than work in which he/she has previously demonstrated competence, or at the discretion of the structural engineer. This will allow the welder to become familiar with atypical welds, and will give the inspector the opportunity to clearly observe 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 as all welding positions and types of weld would be experienced, thus showing the welder capable of successfully completing the weld in all required positions, and applying all heating requirements.

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, fractured columns, etc. should be indicated on the drawings 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 posses significant ability to participate in the structure's lateral load resisting system. Consideration should be given to limiting the total number of connections being repaired at any one time, as the overall lateral load 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 pro-active role in determining this.*

6.5 Execution

6.5.1 Introduction

Recommended general requirements should include the following:

1. Strict enforcement of the welding requirements in AWS D1.1 as modified in 1994 *UBC* Chapter 22, Division VIII or IX.

Commentary: Following the 1994 Northridge earthquake, the AWS established a presidential task group to determine if deficiencies in the D1.1 code contributed to the unexpected damage, and to determine if modifications to the code should be made. That task group noted some areas of practice, related to steel moment frames in seismic zone, that could be improved relative to D1.1. These included the following recommendations:

a) *the root pass of the complete joint penetration welds of beam to column flanges should not exceed 1/4 inch in size, for prequalified procedures.*

b) *where notch tough weld metal is desired, such as at the critical complete joint penetration welds of beam flanges to columns, the maximum interpass temperature should not exceed 550^o.*

c) *when a FCAW process is used, the welding procedure specification should conform to the electrode manufacturer's recommendations.*

d) *the criteria for joints loaded in tension should apply to both top and bottom flange connections in moment frames.*

Future editions of the AWS D1.1 code may adopt some or all of these recommendations. In the interim period, the structural engineer should consider including these recommendations in the project welding specifications, to supplement the standard AWS D1.1 requirements.

2. Implementation of the special inspection requirements in *1994 UBC* section 1701 {*NEHRP-91* Section 1.6.2.6} and AWS D1.1. Visual inspection means that the inspector inspects the welding periodically for adherence to the approved Welding Procedure Specification (WPS) and AWS D1.1 starting with preliminary tack welding and fit-up and proceeding through the welding process. Reliance on the use of nondestructive testing (NDT) at the end of the welding process alone should be avoided. Use visual inspection in conjunction with NDT to improve the chances of achieving a sound weld.

3. Require the fabricator to prepare and submit a WPS with at least the information required by AWS D1.1 as discussed in Section 4.

4. Welding electrodes should be capable of depositing weld metal with a minimum notch toughness as described in Chapter 8.

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

6. Weld dams do not meet the intent of weld tabs, are not permitted by AWS D1.1, and should not be permitted in the work. Dams are not necessary when proper bead size limitations are observed.

7. 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.

Prior to removing weld backing, 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, a WPS should be prepared for each welding process to be used on the project and should include minimum preheat, maximum interpass temperatures, and the as-gouged cross section which must simulate a prequalified joint design of D1.1. If for any reason the WPS does not meet the prequalified limits of AWS it should be qualified by test, in accordance with Section 5.2 of AWS D1.1 In addition the contractor should propose the method(s) which will be used to remove the weld backing, back gouge to sound metal and when during this process he will apply preheat.

Although project conditions may vary, the following general guidelines may be considered:

> The steel backing may be removed by either 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 MT to determine if any linear indications remain. If the area is free of indications the area may then be re-welded. The preheat should be maintained and monitored throughout the process. If no further modification is to be made or if the modification will not be affected by a reinforcing

fillet weld, the reinforcing fillet may be welded while the connection remains at or above the minimum preheat temperature and below the maximum interpass temperature.

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. If cover plates are to be added, the removal of the weld tabs may occur before or after the plate is added depending on the width and configuration of the plate. This sequence should be submitted to the structural engineer for his/her approval prior to the beginning of the work.

The weld tabs may be removed by air arc or oxy-fuel gouging followed by grinding or by grinding alone. The resulting contour should blend smoothly with the face of the column flange and the edge of the beam flange and should have a radius of 1/4-3/8 inch.

The finished surface should be visually inspected for contour and any visually apparent indications. This should be followed by 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.

If the defect removal has extended into the theoretical weld section, then 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.

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

Any 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 UT results which describe the location depth and extent of the defect(s).

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 which conforms to the WPS. Prior to welding MT should be performed to determine if any additional defects have been exposed.

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

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

Commentary: Removal of the weld backing from the top flange may be difficult, particularly along perimeter frames where access to the outer side is restricted. Nevertheless, there may be benefits to providing a weld with a more favorable contour (i.e. that produced by the reinforcing fillet). Tests conducted to date have not been conclusive with regard to the benefit of top flange weld backing removal. At this time, there is no direct evidence that removal of weld backing from continuity plates in the column panel zone is required.

The decision to remove end dams should be based upon the results of UT. Since numerous stop - starts have occurred in this section of the theoretical weld, rejectable edge indications may reduce the integrity of the weld, especially during dynamic or seismic loading. If, however the area is found acceptable by UT removal is not necessary.

Excessive weaving of the weld bead, which can lead to unacceptable stresses at the toe of each weave, should not be allowed. However, some oscillation of the electrode may be required to obtain good fusion.

6.5.2 Girder Repair

If at bottom flange repairs back gouging removes sufficient material such that a weld backing is required for the repair, after welding the backing should be removed from the girder. Alternatively, a double-beveled joint may be used The weld root should be inspected and tested for imperfections, which if found, should be removed by back-gouging to sound material. A reinforcing fillet weld should be placed at T joints 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 1î beyond the opposite side of the girder web. This is to allow adequate access for clearing and inspection of the initiation point of the weld before the second half-length of the root pass is applied. It is not desirable to initiate the arc in the exact center of the girder width since this will limit access to the start of the weld during post-weld operations. After the arc is initiated, travel should progress towards the end of the joint (outboard beam flange edge), and the weld should be terminated on a weld tab.

b) The weld may be initiated on the weld tab, with travel progressing toward the center of the girder flange width. When this approach is used, the welder should stop the weld approximately 1î 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 near the center of the beam flange should be visually inspected to ensure fusion, soundness, freedom from slag inclusions and excessive porosity. The resulting bead profile should be suitable for obtaining fusion by the subsequent pass to be initiated on the opposite side of the girder web. If the profile is not conducive to good fusion, the start of the first root pass should be ground, gouged, chipped or otherwise prepared to ensure adequate fusion.

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

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

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

6.5.3 Weld Repair (Types W1, W2, or W3)

When W1, 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. As stated in Section 5.3.2, W1 cracks may not be earthquake damage. Regardless, it may be prudent to repair this classification group while the other more serious damage is being repaired.

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

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

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

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

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

6.5.4 Column Flange Repairs - Type C2

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

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

PROJECT ADDRESS: ______________________________

INSPECTION DATE: ______________________

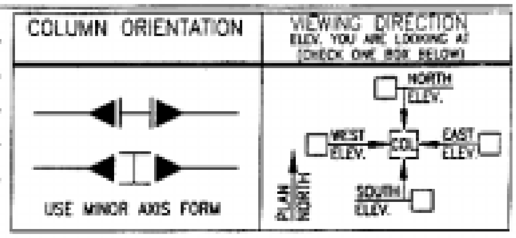
INSPECTOR NAME: ______________________

INSPECTION COMPANY: ______________________

FLOOR FRAMING LEVEL INSPECTED: __________

GRID LOCATION: ______________________

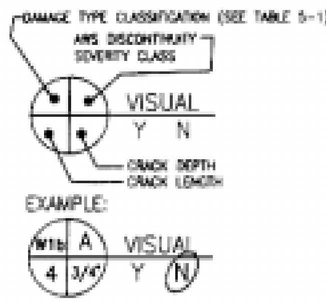
PROVIDE VISUAL INSPECTION OF ALL JOINT WELDS AND BOLTS, AND U.T. INSPECTION OF ALL CJP WELDS.



A.W.S. DISCONTINUITY SEVERITY CLASS

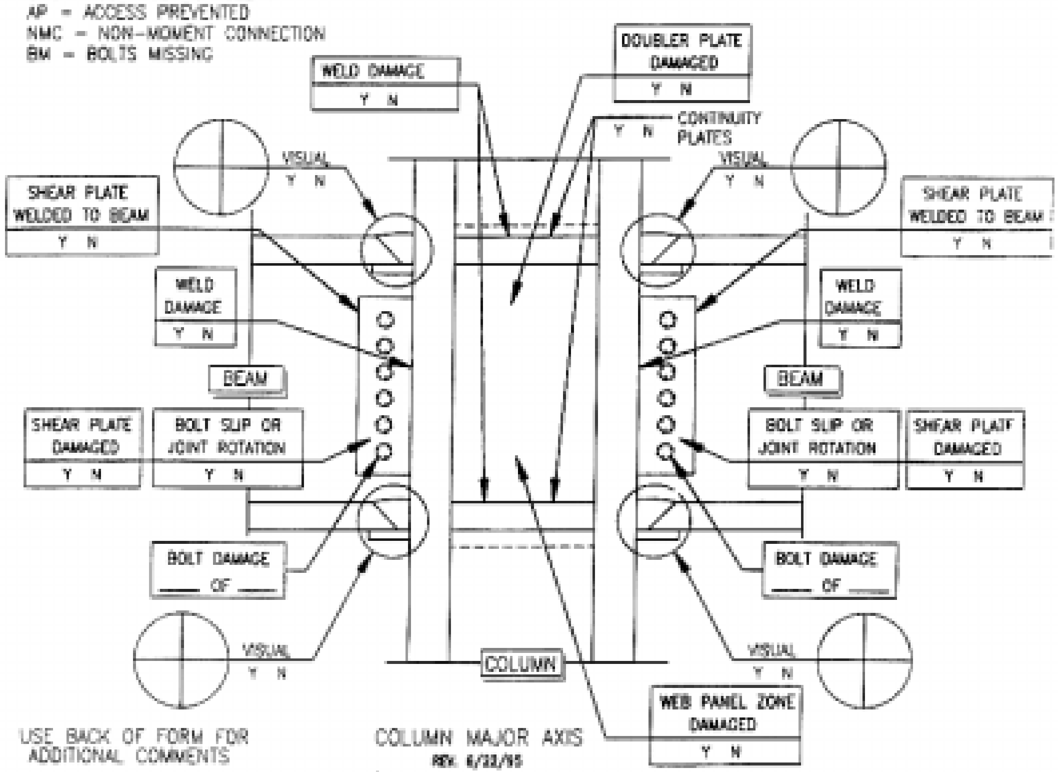
A – LARGE DISCONTINUITIES
B – MEDIUM DISCONTINUITIES
C – SMALL DISCONTINUITIES
D – MINOR DISCONTINUITIES

FOR 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



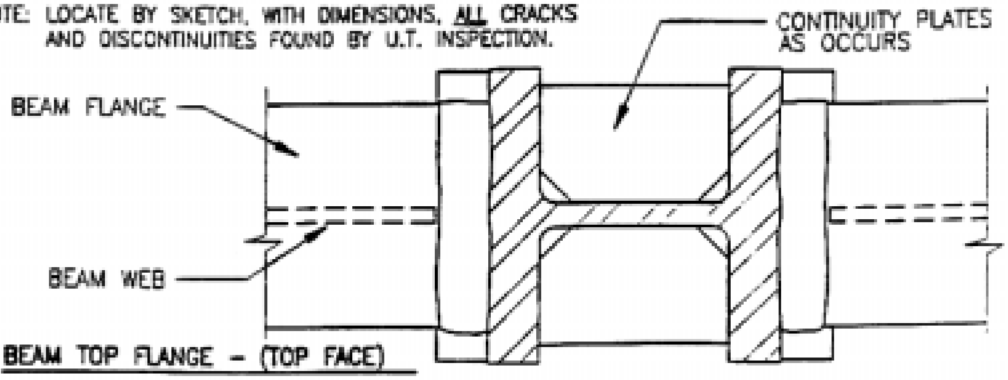
USE BACK OF FORM FOR ADDITIONAL COMMENTS

COLUMN MAJOR AXIS

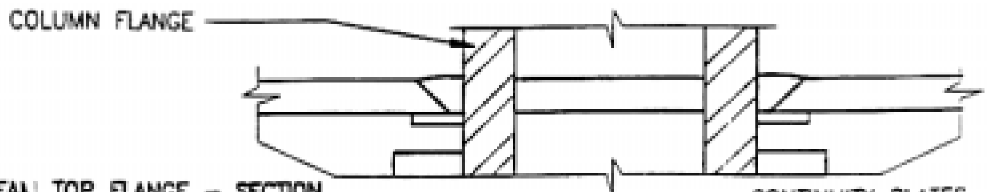
REV. 6/22/95

SUPPLEMENTAL SKETCH FOR LARGE DISCONTINUITIES

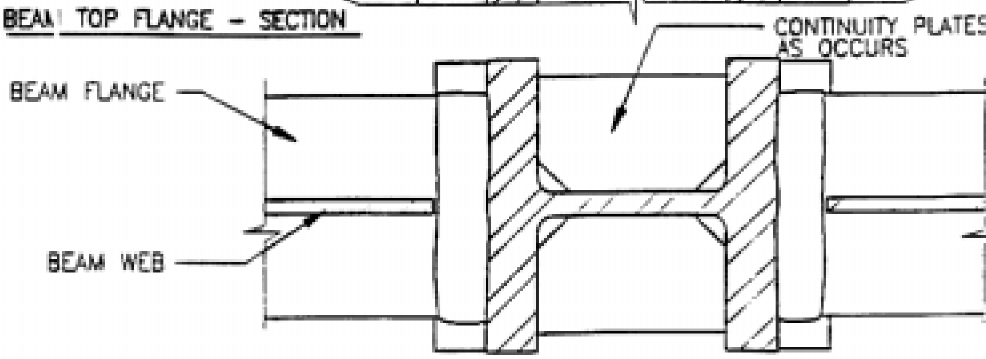
NOTE: LOCATE BY SKETCH, WITH DIMENSIONS, **ALL** CRACKS AND DISCONTINUITIES FOUND BY U.T. INSPECTION.



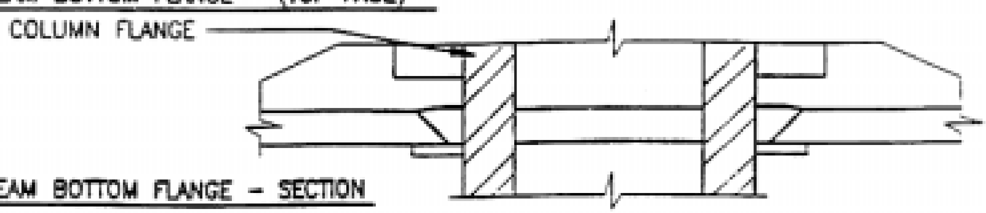
BEAM TOP FLANGE - (TOP FACE)



BEAM TOP FLANGE - SECTION



BEAM BOTTOM FLANGE - (TOP FACE)



BEAM BOTTOM FLANGE - SECTION

ADDITIONAL COMMENTS: ____

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

PROJECT ADDRESS: ____________________

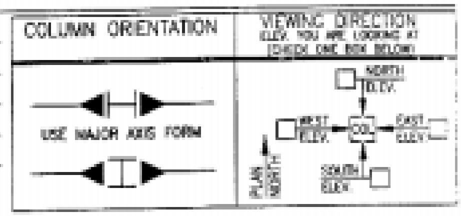
INSPECTION DATE: ____________________

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

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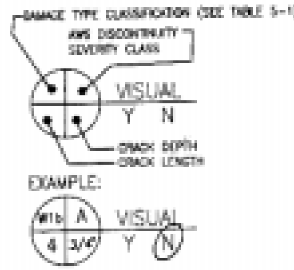
PER AWS ULTRASONIC ACCEPTANCE-REJECTION CRITERIA TABLE 8.2 – AWS D1.1-94



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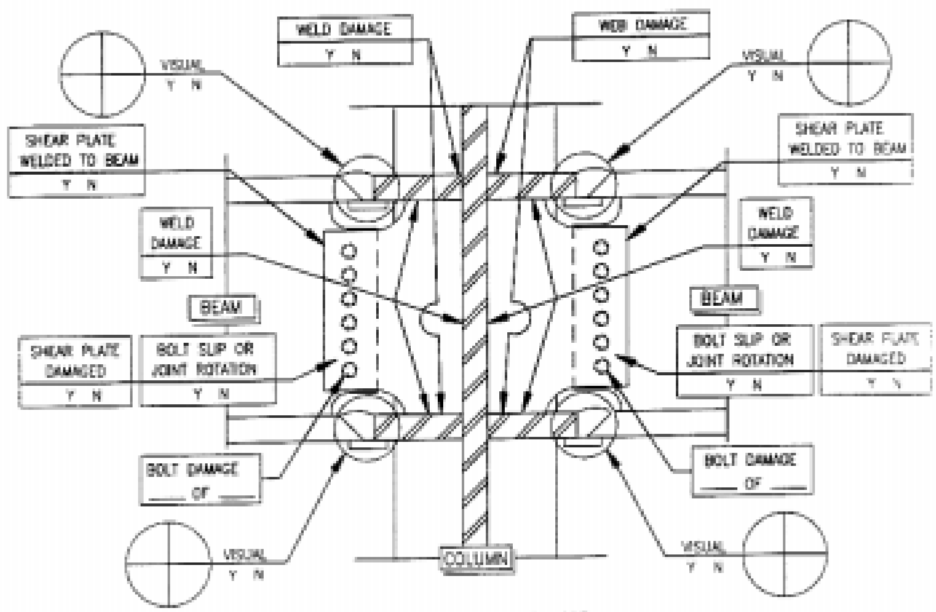
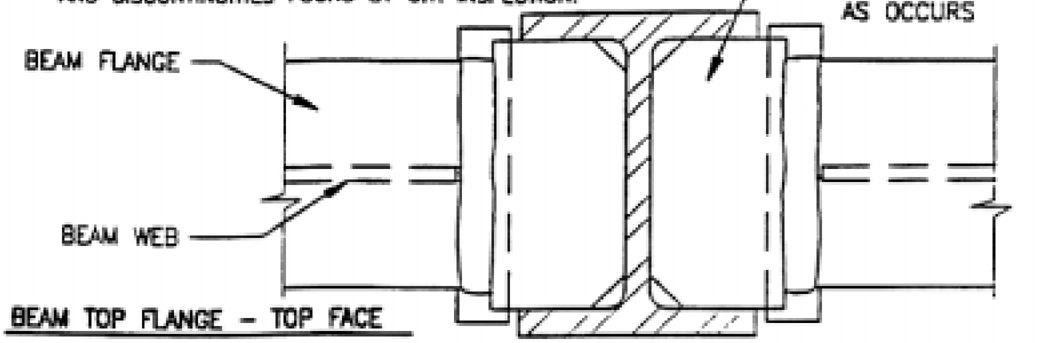


USE BACK OF FORM FOR ADDITIONAL COMMENTS

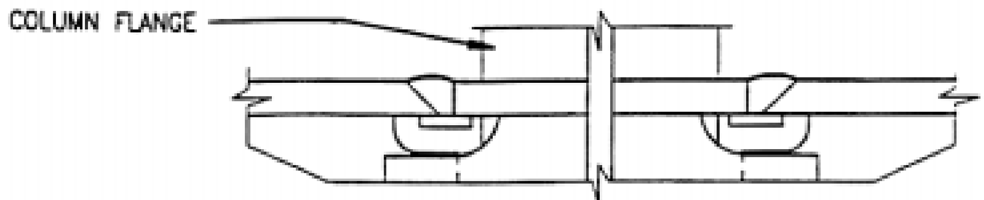
COLUMN MINOR AXIS

SUPPLEMENTAL SKETCH FOR LARGE DISCONTINUITIES

NOTE: LOCATE BY SKETCH, WITH DIMENSIONS, ALL CRACKS AND DISCONTINUITIES FOUND BY U.T. INSPECTION.



BEAM TOP FLANGE – TOP FACE



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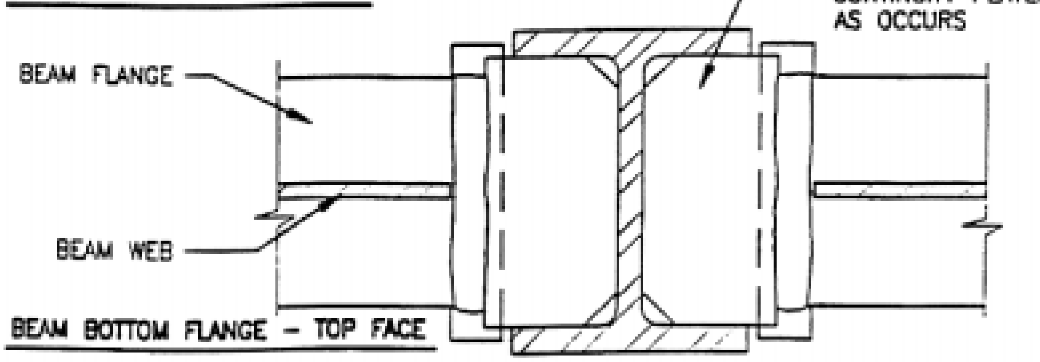
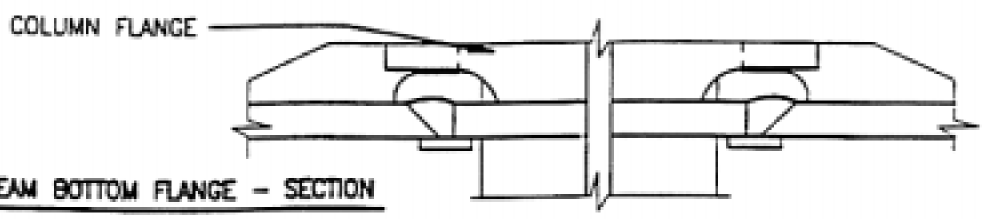


BEAM BOTTOM FLANGE – TOP FACE



BEAM BOTTOM FLANGE – SECTION

ADDITIONAL COMMENTS: ____

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