Seismic Evaluation & Upgrade Criteria for Existing 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.

Seismic Evaluation and Upgrade Criteria for Existing Welded Moment Resisting Steel Frame Construction

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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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

The purpose of this document is to provide practicing engineers and building officials with a resource document for the evaluation and upgrade of moment-resisting steel frame structures to resist the effects of earthquakes. The recommendations were developed by practicing engineers based on professional judgment and experience and a program of laboratory, field, and analytical research. **No warranty is offered with regard to the recommendations contained herein, either by the Federal Emergency Management Agency, the SAC Joint Venture, the individual joint venture partners, their directors, members or employees. These organizations and their employees do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, products or processes included in this publication. The reader is cautioned to carefully review the material presented herein and exercise independent judgment as to its suitability for application to specific engineering projects.** 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 these *Seismic Evaluation and Upgrade Criteria for Existing Welded Moment-Resisting Frame Construction* is to provide engineers and building officials with guidelines for evaluating the probable earthquake performance of existing buildings and structures of welded moment-resisting steel frame (WMSF) construction and for designing upgrades to such structures to improve their probable performance. It is one of a series publications prepared by the SAC Joint Venture addresses 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.

- *Post-earthquake Guidelines for Moment-Resisting Steel Frame Construction* - These guidelines provide recommendations for: performing post-earthquake inspections to detect damage in steel frame structures, evaluating the damaged structures to determine their safety in the post-earthquake environment and repairing damaged structures.

- *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.

1.2 Intent

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

a) Engineers engaged in the evaluation and upgrade of existing, steel frame structures that may be subject to the effects of future earthquake ground shaking.

b) Regulators and building departments responsible for control of the design and construction of structural upgrades in regions subject to the effects of 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.

This publication is not intended for use in the assessment of earthquake-damaged structures, in the period immediately following an earthquake, nor for determining appropriate repair criteria for such structures. This information is contained in a companion document.

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.

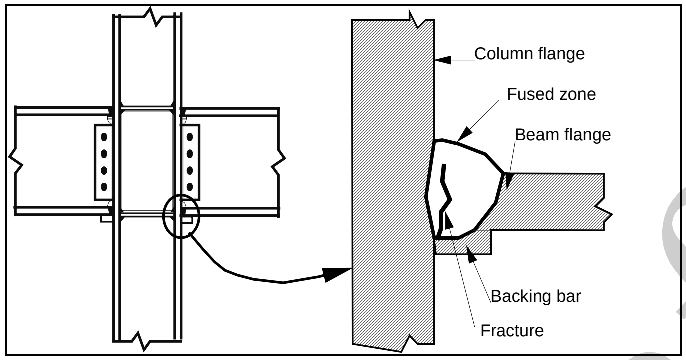


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

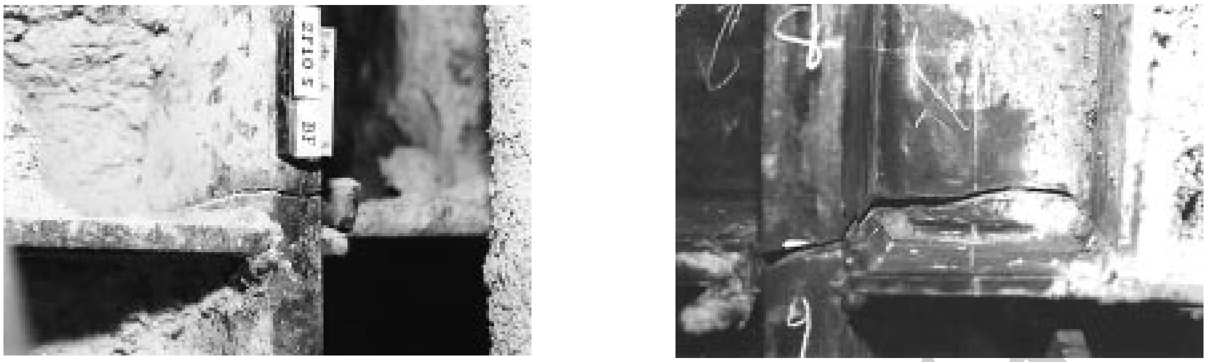
In some cases, the fractures progressed completely through the thickness of the weld, and if fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-2a). Other fracture patterns also developed. In some 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 Application

This publication supplements the seismic evaluation criteria provided in *FEMA-310, NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, and the seismic upgrade design criteria contained in *FEMA-273, NEHRP Guidelines for Seismic Rehabilitation of Buildings* . In addition, it provides guidelines for performing estimates of potential economic losses associated with the earthquake performance of WSMF buildings. It supersedes similar recommendations contained in *FEMA-267, Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Moment Frame Structures* and the various supplements to *FEMA-267*.

Users are cautioned that in many jurisdictions, the design of modifications to existing structures are evaluated based on the provisions contained in the building code for the design of new structures. The criteria presented herein are independent of, and not directly related to, the building code provisions. Prior to performing an upgrade design, in accordance with this criteria, users should ascertain the acceptability of this approach with the local building code authority.

1.5 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).

1.6 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.

1.7 Guidelines Overview

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

- **Chapter 2 - Evaluation Overview** - This chapter provides an historic perspective of the development of moment-resisting steel frame design and construction practice in the United States. It also includes discussion of the performance of welded moment-resisting steel frame construction in recent earthquakes and the causes for much of the damage observed in this construction. Guidelines for collection basic data on the configuration, details and materials of construction of a building, prior to conducting an evaluation are presented as is a brief introduction into the types of evaluation that may be conducted.

- **Chapter 3 - Performance Evaluation** - This chapter presents detailed analytical procedures for determining the probable structural performance of a welded moment-resisting steel frame structure given the seismicity of its site. These procedures allow the calculation of the probability that either of two structural damage states, termed respectively, incipient damage and collapse prevention, will be exceeded in a given period of time, typically taken as 50 years. If the calculated probability is unacceptably high, then the structure can be upgraded and re-evaluated for acceptable performance.

- **Chapter 4 - Loss Estimation** - This chapter provides two alternative methods of estimating probable earthquake repair costs for a welded moment-resisting steel frame structure. One of these is a rapid loss estimation methodology that allows estimation of probable repair costs using basis information on the structure's configuration and age, and the intensity of ground shaking at the site. The second method is a more detailed procedure that relates probable repair costs directly to analytical evaluations of the response of the structure to various levels of ground shaking.

- **Chapter 5 - Seismic Upgrade** - This chapter presents guidelines for two approaches to seismic upgrade of existing welded moment-resisting steel frame structures. The first approach, termed simplified upgrade, consists of modification of individual moment-resisting connections to reduce their susceptibility to ground shaking induced brittle fracture. The second method is a detailed procedure in which the performance of the structure is first evaluated, an upgrade approach is conceived and designed in a preliminary manner, then the performance of the upgraded structure evaluated. This process is repeated until acceptable performance is obtained. Upgrades in this second method may consist of connection upgrades, as in the simplified upgrade approach, but may also include modification of the structural system, such as introduction of braces, or energy dissipation devices.

- **Chapter 6 - Connection Qualification** - This chapter presents modeling guidelines and performance data for different types of beam column connections. It also includes a procedure for determining the performance capabilities of connection types not specifically included in these Guidelines.

- **Chapter 7- Structural Specifications** - This chapter presents a guideline specification, in CSI format, that may be used as the basis for a structural steel specification for moment-resisting steel frame construction. Note that this guideline specification must be carefully coordinated with other sections of the project specifications when implemented as part of the construction documents for a project.

- **Chapter 8 - Materials and Fracture Resistant Design** - This chapter provides fundamental information on the basic properties of steel materials and the conditions under which structural steel fabrications can be subjected to brittle fractures. A more detailed treatment of this information may be found in the companion publication, **FEMA-XXX State of Art Report on Materials and Fracture**.

2. Evaluation Overview

2.1 Scope

This section provides a discussion of the history of the development of moment frame buildings and the general earthquake damage and vulnerabilities associated with WSMF structures. An overview of the evaluation procedures is presented along with corresponding sections regarding material property and condition assessment approaches.

2.2 Moment-Resisting Steel Frame Building Construction

2.2.1 Introduction

Steel frames have been used in building construction for more than one hundred years. In the early 20th century, typical steel frames were of riveted construction. Beam-column connections were typically of the partially restrained type in which beam flanges were attached to columns through a series of clip and seat angles, such as illustrated in Figure 2-1. Designers often assumed that these assemblies acted as “pinned” connections for gravity loads and “fixed” connections for lateral loads. Although some hot-rolled shapes were available, these were typically limited to beam applications. Columns and girders were often fabricated out of plate and angle sections. Frames were typically designed for lateral wind loading, employing approximate methods of frame analysis, such as the portal method or cantilever method.

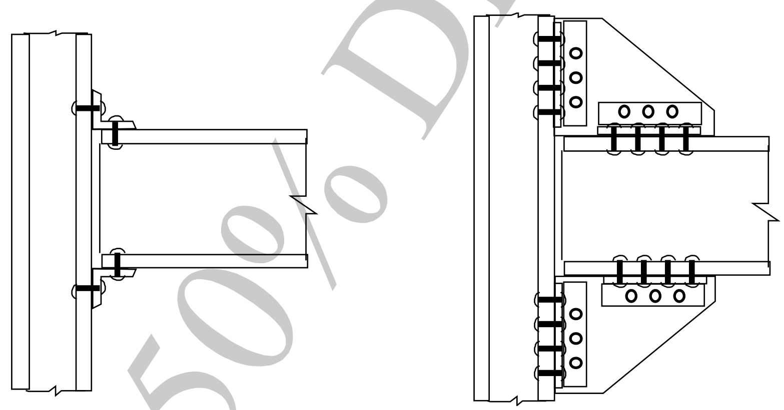


Figure 2-1 - Typical Early Beam-Column Connections

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

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

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

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

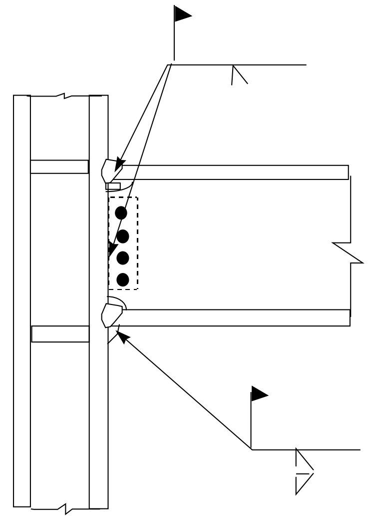


Figure 2-2 - Typical Bolted Web, Welded Flange Connection

2.2.2 WSMF Construction

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

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

2.2.3 Damage to WSMF Construction in the 1994 Northridge, California Earthquake

Following the January 17, 1994 Northridge, California earthquake, more than 100 steel buildings with welded moment-resisting frames were reported to have experienced beam-to-column connection fractures. The damaged structures covered 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 are 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 these buildings was quite severe. In addition to fracturing of the welded joints between beam and column flanges, other elements sustained damage, including girders, columns, columns panel zones (including girder flange continuity plates and column web doubler plates), the welds of the beam to column flanges and the shear tabs which connect the girder webs to column flanges.

Despite the obvious local strength impairment resulting from this damage, many WSMF buildings did not display overt signs of structural damage, such as permanent drifts, or extreme damage to architectural elements. 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. In order to reliably determine if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing and perform nondestructive examination including visual inspection and ultrasonic testing. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. A few WSMF buildings sustained so much connection damage that it was been deemed more practical to demolish the structures rather than to repair them.

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

Review of statistics obtained from a data base of the damage reported under this ordinance program indicates that the damage was less severe than had originally been perceived. Reports for approximately 1/3 of the buildings under the ordinance indicated that no damage was found in the structures. Reports for another 1/3 of the buildings indicated that damage was limited to rejectable defects at the roots of the beam flange to column flange welds. Only 1/3 of the total reports indicated damage other than weld root defects. Of the buildings with reported damage other than weld root defects, two-thirds had less than 10% of their connections fractured. Only eleven percent of all the buildings included in the ordinance had more than 10% of their connection damaged, while relatively few buildings (13% of the total) accounted for 90% of all damage other than defects at the weld roots.

The distribution of damage in these buildings points to some important potential findings. The concentration of damage in a relatively small sample of buildings would seem to indicate that in order to sustain severe damage, a WSMF building must either experience very strong response to the earthquake ground motion, or, as a result of design configuration and/or construction quality conditions, be particularly susceptible to damage. It would seem that most WSMF buildings, are not particularly susceptible to severe damage under relatively strong levels of ground shaking.

Although more than 100 buildings inspected immediately following the Northridge earthquake were reported as having damage, in many cases this reported damage consisted of discontinuities at the roots of the complete joint penetration welds between the beam bottom flange and column flange. There is strong evidence to suggest that most such conditions are not damage at all, but rather, pre-existing construction defects that were not detected during the original construction quality assurance program. Subsequent research in other buildings and cities suggests that the presence of such defects is wide spread and generally present in the population of buildings constructed prior to the Northridge earthquake.

Notwithstanding the above, a number of buildings did experience brittle fracture damage in their beam column connections. The amount of damage sustained by buildings was generally related to the severity of ground shaking experienced at the building site as well as the severity of response of the structure to the ground shaking. However, the presence of construction defects in the welded joints was also a significant factor in the initiation of fracture damage. Joints with severe defects at the weld roots were more susceptible to fracture initiation than joints without such defects. Since the distribution of joints with defects in an existing structure is somewhat random, this tends to minimize the effectiveness of structural analysis to predict the exact locations where damage is likely to occur under ground shaking. However, probabilistic methods based on structural analysis have been successful in indicating the most likely locations for damage. Therefore, the Guidelines contained in this document are based on such approaches.

Commentary: The SAC Joint Venture has established the data base of damage inspection information from the Northridge earthquake for access on-line, through the SAC World Wide Web site.

2.2.4 Damage to WSMF Construction in Other Earthquakes

Following the discovery of unanticipated damage to WSMF construction in the Northridge earthquake, engineers and building officials became concerned that similar, but as yet undetected damage, may have occurred in WSMF buildings that had been affected by other earthquakes, such as the 1989 Loma Prieta earthquake in the San Francisco Bay Area. As part of Phase 2, a concerted effort was undertaken to determine the amount and extent of earthquake damage resulting, mainly, from other recent earthquakes, and also from the 1971 San Fernando Earthquake. Specifically, WSMF damage information was gathered from the 1989 Loma Prieta and 1992 Landers and Big Bear series of events. Unfortunately, since no mandatory inspection programs of WSMF buildings were conducted following these earthquakes, only limited data is available. This data clearly demonstrates that buildings subjected to these events were susceptible to the same types of damage as that experienced by buildings affected by the Northridge earthquake.

One year to the day following the Northridge earthquake, on January 17, 1995, a magnitude 6.9 earthquake occurred near Kobe, Japan. Kobe is a very large city with a population of about 1.5 million and had many WSMF structures in its building stock. These structures ranged from relatively small low-rise buildings constructed in the 1950s and 1960s to modern high-rise structures constructed within the last 10 years. Design and construction practice in Japan is significantly different from that common in the United States. Many of the smaller Japanese WSMF structures employ cold formed, tubular steel columns, with the beams, rather than columns, run continuously through moment-resisting connections. Many of these connections failed resulting in more than 50 collapses of these buildings. Many of the larger buildings experienced connection fractures similar to those observed in buildings affected by the Northridge earthquake, although connection detailing and welding processes employed in these buildings were different than those common in the United States. Just as in the United States, the Japanese believed that this damage was serious enough to warrant investment in a large program of research and development to determine the cause of the poor performance of WSMF buildings in Kobe and to develop new techniques for design and construction of more reliable WSMF buildings.

The combined discovery of extensive damage to WSMF buildings affected by the San Fernando, Loma Prieta, Landers, Big Bear, and Kobe earthquakes provided conclusive evidence that the damage discovered in the Northridge earthquake was not a unique, or freak occurrence, resulting from some peculiar characteristics of the ground motion experienced in the Northridge earthquake or of design and construction practice in Southern California.

2.2.5 Post-Northridge Earthquake Construction Practice

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

- Common use of very large framing members even in relatively small buildings. Initial testing of WSMF connections, conducted in the 1960s and 1970s, utilized assemblies that employed small size elements, typically W18 beams and light W12 and W14 column sections. Typical buildings damaged by the Northridge earthquake employed W30 or larger beams connected to very heavy W14 columns. It appears that scale effects play a significant role in the behavior of WSMF connections and that details that behave well for connections of small sections do not necessarily behave as well for larger sections.

- One of the basic rules of detailing structures for superior seismic performance is to design connections of elements such that the connection is stronger than the elements themselves so that any inelastic behavior occurs within the element and not the connection. There are several reasons for this. The strength and ductility of any connection is highly dependent on the quality of the workmanship employed. Connections, being relatively limited in size, must undergo extensive local ductility if they are to provide significant global ductility. The basic fabrication process for connections, employing cutting, welding, and bolting, tend to induce a complex series of effects on both the residual stress state and metallurgy of the connected parts, that is often difficult to predict. Despite these common axioms of earthquake resistant design, typical detailing practice prior to the Northridge earthquake relied on the development of large inelastic behavior within the beam-column connections.

- Welding procedures commonly employed in the erection of WSMF buildings resulted in deposition of low toughness weld metal in the critical beam flange to column flange joints. This weld metal is subject to the initiation and development of unstable brittle fractures when subjected to high stress demands and used in situations with significant geometric stress risers, or notches.

- Welding practice in many of the damaged structures was found to be sub-standard, despite the fact that quality assurance measures had been specified in the construction documents and that construction inspectors had signed documents indicating that mandatory inspections had been performed. Damaged welds commonly displayed inadequate fusion at the root of the welds as well as substantial slag inclusions and porosity. These defects resulted in ready crack initiators that enabled brittle fractures to initiate in the low-toughness weld metals.

- Detailing practice for welded joints inherently resulted in the presence of fracture-initiators. This includes failure to remove weld backing and runoff tabs from completed joints. These joint accessories often contain or obscure the presence of substandard welds. In addition, they introduce geometric conditions that are notch-like and can serve as fracture initiators.

- The presence of low toughness metal in the fillet region of some structural shapes. The metallurgy of the material in the fillet or "k-line" region of a rolled shape often has lower toughness properties than material in other locations of the section due to a number of shape production factors including a relatively prolonged cooling rate for this area, as well as significant cold working during shape straightening. While not normally a problem, the combined presence of weld access holes through this region at the beam-column connection and large induced stresses from buckling and yielding of the beam flanges under inelastic frame action can result in initiation of fractures in this region. These problems are made more severe by improperly cut weld access holes which can result in sharp notches and crack initiation points. This was not a common problem in the Northridge earthquake because most of connections that experienced damage did so because of other, more significant vulnerabilities. However, much of the damage that occurred to Japanese structures in the Kobe earthquake was apparently the result of this problem.

- In the 1980s, some engineers came to believe that shear yielding of the panel zones in a beam-column connection, as opposed to flexural hinging of the beam, was a more benign and desirable way to accommodate frame inelastic behavior. In response to this, in the mid-1980s the building code was modified to include provisions that allowed the design of frames with weak panel zones. Contrary to this belief that panel zone yielding is beneficial and desirable, excessive yielding actually produces large secondary stresses at the beam flange to column flange joint, which can exacerbate the initiation of fractures.

- The yield strength of structural shape material had become highly variable. In the 1980s and 1990s, the steel production industry in the United States underwent a major realignment with new mills coming on-line and replacing older mills. Although there had always been significant variation in the mechanical properties of structural steel material, the introduction of material produced by these newer mills introduced significant additional variation. The newer mills used scrap-based steel production which tends to produce higher strength material than did the older mills. In fact, much of the A-36 material produced by these newer mills also met the strength requirements for the higher strength A-572 specification. Many designers had traditionally specified A-572 material for columns and A-36 material for beams, in order to economically obtain structures with weak beams and strong columns. The introduction of higher strength A-36 material into the market effectively negated the intent of this specification practice and often resulted in frame assemblies in which the beams were stronger than the columns or panel zones were weaker than intended, relative to the beam strength. These combined effects resulted in greater strength demands on the welded joints and permitted them to occur in an unanticipated manner.

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

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

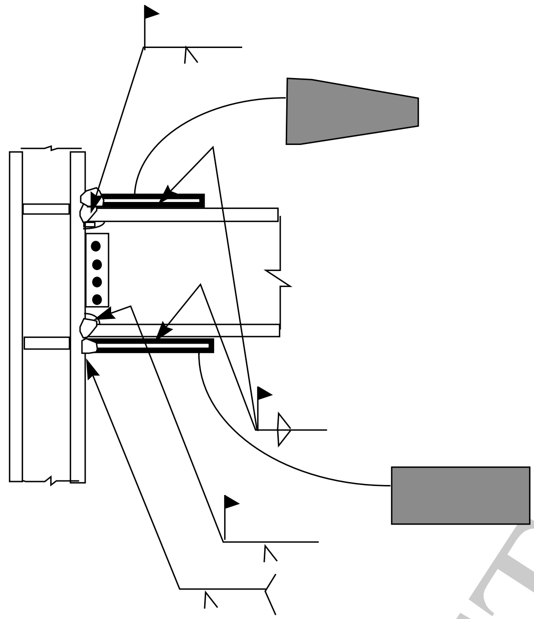


Figure 2-3 - Typical Cover Plated Connection

In the earliest connections of this type, welding was performed with electrodes that deposited material of improved toughness relative to that commonly used prior to the Northridge earthquake, but less toughness than later recommended by the *FEMA-267* Interim Guidelines. In August, 1995, *FEMA-267* was published and the design and fabrication of these connections became more consistent. *FEMA-267* also included information on other types of connections, that were believed capable of providing acceptable performance including haunched connections, reduced beam section connections, vertical rib plate connections, side plate connections and slotted web connections. The recommendations contained in *FEMA-267* were based on preliminary research and were of an interim nature. While it is expected that frames constructed with connections designed using the *FEMA-267* guidelines are more resistant to connection fractures than earlier frames, it should not be assumed that they are completely free of potential for such damage.

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

2.3 Typical Pre-Northridge Connection Damage

Following the January 17, 1994 Northridge, California Earthquake, damage to elements of welded steel moment-resisting frames (WSMF) was categorized as belonging to the weld (W), girder (G), column (C), panel zone (P), or shear tab (S) categories. Damage at a joint may be confined to one category or may include multiple types. The damaged WSMF may also exhibit global effects, such as permanent inter-story drifts. The components of a typical pre-Northridge connection are shown in Figure 2-4. Discovery of these extensive connection fractures, 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 damage to WSMF connections in buildings affected by the 1992 Landers Big Bear and 1989 Loma Prieta earthquakes.

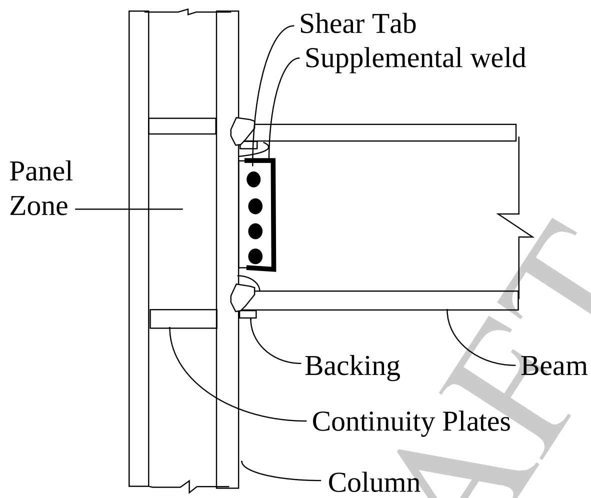


Figure 2-4 Components of Moment Connection

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

In some cases, the fractures progressed completely through the thickness of the weld, and 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 through-thickness failure 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.

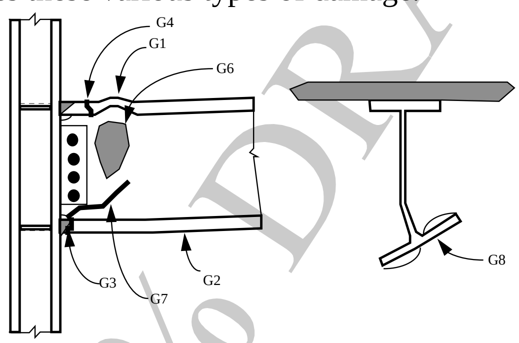
A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-2a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-2b). Investigators have reported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

Once such fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and strength. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-3).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts, or extreme damage to architectural elements. The following sections detail typical damage types, using the system for categorizing damage recommended by *FEMA-267* for post-earthquake damage assessment.

2.3.1 Girder Damage

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



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

Figure 2-5 - Types of Girder Damage

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 torsional buckling of section

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 (Luder 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). Once this type of damage occurs, the girder flange may rapidly loose all tensile capacity under continued or reversed loading, however, it may retain some capacity in compression.

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 laboratory tests and in a number of buildings in Japan, this damage was found to extend from the weld access hole in the web of the girder, a metallurgically and geometrically complex area, into the flange. Shape and smoothness of the weld access hole can affect the propensity of a connection to experience such damage. As shown in Figure 2-5, G-4 damage often occurs at a location of local flange buckling, which is where it has been observed in some testing of large-scale assemblies, after many cycles of load. Also, in some reason tests of post-Northridge unreinforced connections, utilizing notch-tough welding consumables, G4 fractures commonly initiated at the toes of the weld access holes, fracturing across the flange in a brittle manner. Similar damage was found in Japanese buildings following the Kobe earthquake.

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. One of the most important factors is the basic difficulty of making the bottom flange weld. 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. In addition, at the beam bottom flange, the root of the welded beam flange to column flange joint, which often has a number of defects, due to the basic difficulty in making the root pass, is located at the extreme flexural fiber, and therefore is subjected to the greatest strains, while at the top flange, the root of the weld is located away from the extreme flexural fibre and consequently is subjected to a less severe strain condition. This condition is made still more severe by the presence of the backing bar, which tends to further increase the "notch" effect due to defects at the weld root and also effectively obscures the detection of such defects during non-destructive examination. When the column has a relatively weak panel zone, the stress condition at the bottom flange is made still more severe as a result of secondary stresses induced in the welded joint by kinking of the column flanges, which must conform to the shear deformed shape of the panel zone.

2.3.2 Column Flange Damage

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

Table 2-2 - Types of Column Damage

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

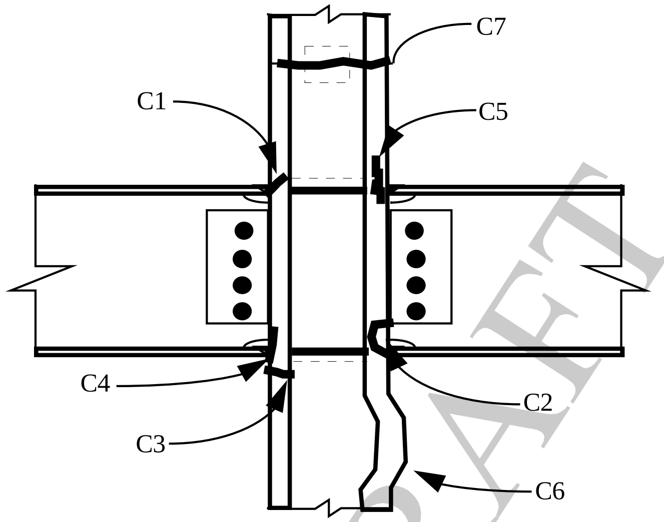


Figure 2-6 - Types of Column Damage

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 the 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 along 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, particularly in older steels, rolled prior to the adoption of electric arc furnace and continuous casting production methods. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands has been known for a number of years and has sometimes been identified as a contributing mechanism for type C2 column flange through-thickness failures.

2.3.3 Weld Damage, Defects, and Discontinuities

Six types of weld discontinuities, defects and damage are defined in Table 2-3 and illustrated in Figure 2-7.

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

Type	Description
W1*	Weld root indications
W1a*	Incipient indications – depth <3/16” or $t_f/4$; width < $b_f/4$
W1b*	Root indications larger than that for W1a
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5*	UT detectable indication – non-rejectable

*Considered defects, not damage.

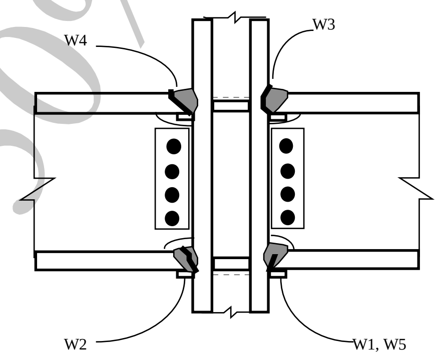


Figure 2-7 - Types of Weld Damage

Commentary: W1 conditions were the single most common type reported in response to a mandatory inspection ordinance enacted by the City of Los Angeles, following the Northridge earthquake. Paret and others developed a data base of the damage reported for 209 buildings under this ordinance and found that for 1/3 of the buildings, no damage at all was reported, while most of the "damage" reported for the remaining buildings consisted of W1 conditions. Only 1/3 of the buildings had reported conditions other than type W1.

Type W1 discontinuities and defects and type W5 discontinuities are detectable only by NDT, unless the backing bar is removed, allowing direct detection by visual inspection or magnetic particle testing. Type W5 consists of small discontinuities. AWS D1.1 permits small discontinuities in welds. Larger discontinuities are termed defects, and are rejectable per criteria given in the Welding Code. It is very likely therefore that some of the weld indications detected by NDT in a post-earthquake inspection are discontinuities which pre-existed the earthquake and do not constitute a rejectable condition, per the AWS standards. Some type W1 indications are small planar defects, which are rejectable per the AWS D1.1 criteria, but are not large enough to be classified as one of the types W2 through W4. Type W1 is the single most commonly reported non-conforming condition reported in the post-Northridge statistical data survey, and in some structures, represents more than 80 per cent of the total damage reported. The W1 classification is split into two types, W1a and W1b, based on their severity. Type W1a incipient root indications are defined as being nominal in extent, less than 3/16" deep or 1/4 of the flange thickness, whichever is less, and having a length less than 1/4 of the flange width. Studies have suggested strongly that almost all type W1a indications are not earthquake damage at all, but rather, previously undetected defects from the original construction process. Nonetheless, rejectable defects would be expected to make a connection more prone to initiation of brittle fracture in future ground shaking.

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.

2.3.4 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 2-8. 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

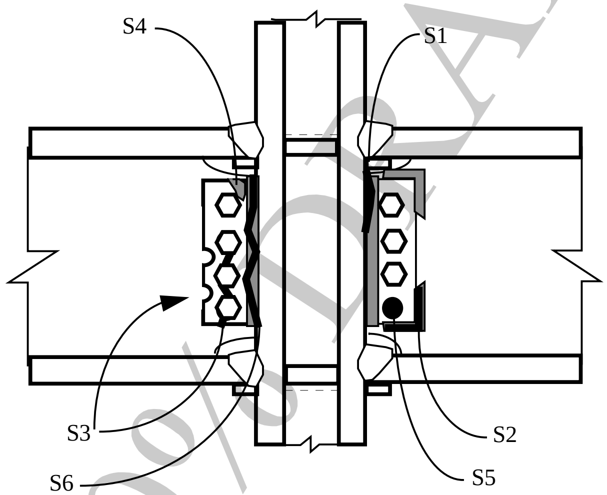


Figure 2-8 - Types of Shear Tab Damage

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.*

2.3.5 Panel Zone Damage

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

Table 2-5 - Types of Panel Zone Damage

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

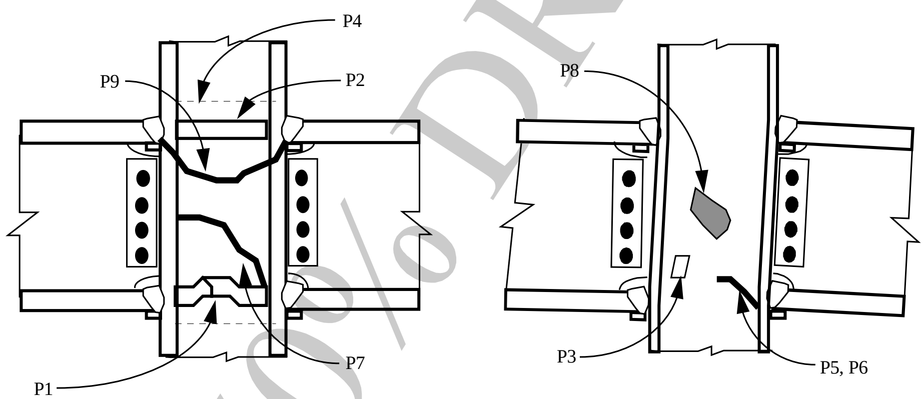


Figure 2-9 - Types of Panel Zone Damage

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

Fractures extending into the column web panel zone (types P5, P6 and P7) have the potential under additional loading to grow and become type P9 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.

2.3.6 Other Damage

In addition to the types of damage discussed in the previous sections, other types of structural damage may also be found in 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.

2.4 Evaluation Procedures

This document provides guidelines for performing several types of evaluation of an existing WSMF structure's probable performance, as outlined below:

- **Performance Evaluation** - The purpose of a performance evaluation is to predict the probability that a structure will experience damage in excess of one of more defined limit states, given the seismicity and characteristics of the building site. In this criteria document, building damage is characterized in terms of two performance levels. Section 3.2.2 provides definition of these performance levels. Once a performance objective for a building has been selected, a performance evaluation can be performed in accordance with Section 3.3 to determine the probability that this performance will be exceeded. The level of confidence that can be attained with regard to the ability of a building to meet a desired performance objective is dependent on the amount of information that is available with regard to the building's configuration and construction, and the rigor of the analytical methods used in the evaluation. The performance evaluation procedures contained in Section 3.3 include procedures for the quantification of uncertainty and confidence with regard to performance prediction. Procedures and information regarding material properties and condition assessments to be utilized in support of the performance evaluation are presented in Section 2.5.

Commentary: In recent years, a series of rapid building performance methodologies including ATC-14, ATC-22, FEMA-178 and most recently FEMA-310 have been developed. These methodologies were developed to provide the engineering community with a consistent yet economical method of determining the probable performance of different types of buildings when subjected to specific earthquake ground shaking levels. These evaluations performed in accordance with these methodologies generally consist of responding to a series of evaluation statements, intended to identify the presence of certain common vulnerabilities, such as soft stories, weak stories, discontinuous lateral force resisting systems, etc., that have been observed to frequently result in poor building performance. The methodologies also commonly employ a series of rapid analytical evaluations that include approximate evaluations of building strength and stiffness.

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

It is recommended that FEMA-310 be performed as a first step in the analytical evaluation of a building's probable seismic performance. Such an evaluation will provide the engineer with a basic understanding of potential critical flaws in the building configuration and provide a basis *for a more detailed analytical evaluation of the building's performance, under the guidelines of this criteria document.*

- **Loss Evaluation** - The purpose of a loss evaluation is to determine the probable repair costs for a structure (or class of structures), if it is subjected to earthquake hazards of defined intensity. In most loss estimation methodologies, repair costs are expressed as a percentage of the building replacement cost. Loss estimation evaluations sometimes include estimates of potential interruption of building occupancy as well as repair cost. Two alternative approaches to loss estimation are provided in this criteria: a rapid loss estimation methodology and a detailed loss estimation method. Rapid loss estimation, described in Section 4.3, can be quickly performed using data on the basic age, size, and construction characteristics of the building, and specification of the intensity of ground shaking for which the loss evaluation is being performed. Detailed loss estimation requires a detailed analytical evaluation of the building and estimation of the ground shaking response accelerations at which different damage states are likely to be exceeded. Section 4.4 provides information on detailed loss estimation methods.

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

The detailed loss estimation methodology provides for the direct consideration of structural characteristics, important to building performance, in the loss evaluation process. In this methodology, structural analyses of the building structure are performed to characterize the probable response of the building to ground motion. Statistical data are then used to relate building response, within defined levels of *uncertainty, to losses that have historically occurred in buildings. The detailed loss estimation methodology is recommended for applications in which it is desired to estimate the probable losses for a single building.*

2.5 Material Properties and Condition Assessments

In order to perform an evaluation of any type for a building, it is necessary to define certain basic structural parameters including the configuration and condition of the structural system and quantification of the material properties. The extent of definition and quantification necessary depends to a large extent on the type of evaluation to be performed and the level of certainty desired with regard to the conclusions of the evaluation. This section identifies the basic parameters requiring consideration and provides guidelines for their acquisition, for the various levels of evaluation. Condition assessment is a very important aspect of planning and executing seismic performance assessment, loss estimation or upgrade design for an existing building. One of the most important steps in condition assessment is a visit to the building for visual inspection.

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

2.5.1 Material Properties

Mechanical properties of component and connection material dictate the structural behavior of the component under load. Mechanical properties of greatest interest include the yield (F_y) and tensile (F_u) strengths of base and connection materials, modulus of elasticity, ductility, CVN toughness, elongational characteristics, and weldability (equivalent carbon content). Although structural steel is an engineered material, there can be significant variability in the properties of materials incorporated in a building. Therefore, it is only possible to characterize the properties of material in a structure on the basis of distributions of properties, with characteristic mean values and coefficients of variation. In general, evaluations are performed using “expected” or mean values of the material properties, based on the understanding obtained of the actual distribution of these properties in the structure. Expected values are denoted in these guidelines with the subscript “e”. Thus, the expected yield and tensile strength of steel material are denoted, respectively, F_{ye} and F_{ue}.

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 desired for the evaluation. If construction documents are not available, the properties of materials indicated in Table 2-6 may be assumed, however, this will affect load and resistance factors used in the performance evaluation, reflecting the level of uncertainty introduced by the use of this approximate data. Determination of material properties is not required for rapid loss estimation. For performance evaluations, material properties should be determined, based on the grades of materials indicated on the construction documents and the information presented in Table 2-7. For more certain performance evaluations and loss estimates, the material properties should be based on the information presented on the original construction documents as supplemented by Table 2-7, 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 may take place in regions of reduced stress such as flange tips at beam ends and external plate edges to minimize the effects of reduced area. 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:

$$\bar{x} = \sum x \Big/ n \qquad (2\text{-}1)$$

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:

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

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

$$COV = \frac{\sigma_x}{\bar{x}} \qquad (2\text{-}3)$$

Table 2-6 - Default Material Properties for Structural Steel

Year of Construction	Component	Material Specification
	Plate and Shape	
1949 - 1963	Shape and Plate	A-7
1963 - 1970	Shape and Plate	A-36
1970 - 1990	Beams and Plate	A-36
	Columns	A-572 Gr. 50
1990 -	Shape	A-572 Gr. 50
	Plate	A-36
	Bolts	
1947 - 1964	All	A-307
1964 - 1990	Not in combination w/ welds	A-307
	In combination w/welds	A-325
	Weld Material	
-1970	E6016	
1970-1994	E70T4	
1994 -	E70TGK2	

Table 2-7 - 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 2-8.

Table 2-8 - 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 2-7. 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 measures 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 of the structural systems and materials used exists, 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.

If a higher degree of confidence in results is desired, the sample size shall be determined using ASTM Standard E22 guidelines. Alternatively, the prior knowledge of material grades from Section 1.2.2.5 may be used in conjunction with Bayesian statistics to gain greater confidence with the reduced sample sizes noted above.

2.5.2 Component Properties

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

- Original cross-sectional shape and physical dimensions.
- size and thickness of additional connected materials, including cover plates, bracing, and stiffeners.
- Existing cross-sectional area, section moduli, moments of inertia, and torsional properties at critical sections.
- As-built configuration of intermediate, splice, end, and base plate

connections.

- Current physical condition of base metal and connector materials, including presence of deformation.

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

2.5.3 Condition Assessment

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

- To examine the physical condition of primary and secondary components and the presence of any degradation.

- To verify or determine the presence and configuration of components and their connections, and the continuity of load paths between components, elements, and systems.

- To review other conditions such as neighboring party walls and buildings and the presence of non-structural components that may significantly influence building performance.

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

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

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

2.5.3.1 Scope and Procedures

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

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

- If detailed design drawings exist, exposure of at least one different primary connection shall occur for each connection type. If no deviations from the drawings exist, the sample may be considered representative. If deviations are noted, then removal of additional coverings from primary connections of that type must be done until the design professional has adequate knowledge to continue with the evaluation and rehabilitation.

In the absence of construction drawings, the design professional should establish inspection protocol that will provide adequate knowledge of the building needed for reliable evaluation and rehabilitation.

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

2.5.3.2 Quantifying Results

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

- Component section properties and dimensions.

- Connection configuration and presence of any eccentricities.

- Type and location of column splices.
- Interaction of nonstructural components and their involvement in lateral load resistance.

The acceptance criteria for existing components depends on the design professional's knowledge of the condition of the structural system and material properties (as previously noted). All deviations noted between available construction records and as-built conditions should be accounted for and considered in the structural analysis.

3. Performance Evaluation

3.1 Scope

This Chapter provides criteria for evaluating the performance of existing WSMF structures. It includes definition of performance objectives, discussions of expected performance of buildings conforming to current code for different design levels, and procedures for estimating the probability that certain performance levels will be exceeded during a defined period of time.

3.2 Performance Definition

In these guidelines, performance is defined in terms of performance objectives. Each performance objective consists of the specification of an earthquake performance level and a probability that this performance level will be exceeded in a specific period of time. The evaluation procedures contained herein permit estimation of a level of confidence with regard to the ability of a structure to achieve a desired performance objective. The basis for a seismic upgrade design is the selection of one or more performance objectives, and an associated level of confidence with which it is desired to attain this performance. For example, an upgrade design may be intended to provide for a 95% confidence level that a structure is able to provide Collapse Prevention performance with a 2% probability of exceedance in 50 years, or a 50% confidence level that a structure will be able to provide Incipient Damage performance, with a 20% probability of exceedance in 50 years. The user may specify any level of confidence for achieving any desired performance goal.

Commentary: These guidelines adopt, with modification, the performance definition and evaluation approaches contained in the FEMA-273 NEHRP Rehabilitation Guidelines. In the FEMA-273 Guidelines, three discrete structural performance levels are defined. These are termed: collapse prevention, life safety and immediate occupancy.

The collapse prevention performance level represents a damage state of near complete damage, though the building has experienced neither partial or total collapse. Damage sustained has substantially degraded both the stiffness and strength of the structure to resist additional lateral loading and the structure is unsafe for occupancy until shored or repaired, which may be impractical to accomplish.

The life safety level is a performance state in which significant damage has been sustained, however, margin remains against either partial or total collapse. A building meeting this level of performance has not endangered the safety of occupants during response to the earthquake and may or may not be safe for re-occupancy prior to repair or temporary *bracing of the structure. In the FEMA-273 Guidelines, the life safety performance level is conceptually envisaged to occur at ¾ of the building response to ground motion that would produce collapse prevention performance. In the FEMA-302/303 NEHRP Recommended Provisions and Commentary, life safe performance is deemed to occur at structural response levels that are 2/3 those at which collapse prevention performance occurs. Due to the somewhat arbitrary definition of this performance level, and the fact that different guidelines and codes have historically selected alternative definitions for this performance, these SAC guidelines do not utilize this performance level. Instead only the Collapse Prevention and Incipient Damage levels are addressed by these guidelines. User's desiring to evaluate building designs for alternative performance may do so by interpolating between the criteria provided for these two levels.*

The Immediate Occupancy performance level in FEMA-273 represents a performance state in which relatively little damage has occurred and in which the structure retains nearly all of its initial strength and stiffness. Buildings meeting this performance level represent a negligible risk to life safety, both during and after the earthquake event. In these guidelines, this performance level is known as Incipient Damage.

The specification of performance in the FEMA-273 guidelines requires the specification of both a performance level, as discussed above, and also a ground motion at which that performance level is to be obtained. Thus, a performance objective in the FEMA-273 document may be expressed as - the building shall obtain collapse prevention performance for ground shaking demands with a 2% probability of exceedance in 50 years." This implies a rather deterministic approach to performance achievement - "if ground motion with a severity that has a 2% probability of exceedance in 50 years is experienced - then the building will not experience performance in excess of the collapse prevention level." These guidelines take a somewhat different approach, that recognizes the uncertainties inherent both in prediction of the ground shaking, and also the structure's performance.

In the approach taken in these guidelines, rather than specifying that a performance level not be exceeded for ground shaking with a given probability of exceedance; performance objectives are defined as the probability that the performance level itself not be exceeded with a given number of years, taking into account the hazards at the site. Thus performance objectives are expressed in the form:

- *Collapse Prevention performance with a 2% probability of exceedance in 50 years*

- *Incipient Damage performance with a 50% probability of exceedance in 50 years*

Although these performance definitions appear quite similar to those contained in FEMA-273, they are actually quite different. The primary difference is that these definitions recognize that there is a distribution of probabilities that the desired performance level will be exceeded, as a function of ground motion severity. Thus, the fact that there is some probability that a given performance level would be exceeded at ground motions less than those having a specific probability of exceedance can be directly recognized by integrating the distribution of probable building performance with the distribution of probable ground shaking demands at various exceedance probabilities. This process is transparent to the user of these guidelines, except through the assignment of load and resistance factors, λ *and* ϕ*, which are products of the integration of the distributions of structural performance and hazard. The user has the option, either of using the default load and resistance factors contained in these guidelines, or alternatively, by computing their own factors using procedures described herein. The calculation of project specific load and resistance factors may be beneficial for some buildings, in that it will result in attainment of a higher confidence of meeting a desired performance objective, through the application of reduced load factors and increased resistance factors.*

One of the benefits of the performance definition approach taken by these guidelines is that it permits a level of confidence to be established with regard to attainment of the desired performance. Neither the FEMA-273 Guidelines or the FEMA-302 NEHRP Provisions are able to establish a confidence level for the attainment of specified performance. The design recommendations contained in FEMA-XXX, Seismic Design Criteria for New Moment-Resisting Frame Construction, a companion to this publication, are intended to provide a 95% confidence level with regard to attainment of specified performance. That is, it is expected that fewer than 5 out of 100 structures designed in accordance with the guidelines FEMA-XXX, would experience damage exceeding the desired level more often than specified (e.g. Collapse Prevention at a 10% probability of exceedance in 50 years).

The evaluation guidelines of this chapter present a detailed procedure for estimating a confidence level with regard to the probability that a structure will be able to provide a specific performance; taking into account the uncertainties inherent in the knowledge of the structure's construction, the analytical procedures and models used to predict its response, and the inherent variability in ground motion.

Note that these guidelines do not address the performance of nonstructural building elements. For guidelines on evaluation of the performance of these components, refer to FEMA-273.

3.2.1 Hazard Specification

3.2.1.1 General

Earthquake hazards include direct ground fault rupture, ground shaking, liquefaction, lateral spreading, and land sliding. Of these various potential hazards, the one that effects the largest number of structures and causes the most widespread damage is ground shaking. Ground shaking is the only earthquake hazard that the building code design provisions directly address. However, for structures located on sites where any of the other hazards can result in significant ground deformation, these hazards should also be considered in structural performance evaluation.

3.2.1.2 Ground Shaking

Ground shaking hazards are typically characterized by a hazard curve, which indicates the probability that a given value of a ground motion parameter, for example peak ground acceleration, will be exceeded in a period of time, and by acceleration response spectra or ground motion time histories that are compatible with the values of the ground motion parameters obtained from the hazard curve and the local site geology. The ground shaking hazard maps contained in the building code and provided with *FEMA-273* have been prepared based on hazard curves that have been developed by the United States Geologic Survey for a grid-work of sites encompassing the United States and its territories. The building code provisions define two specific levels of hazard for consideration in design and specify methods of developing response spectra for each of these levels. The two levels are:

1. Maximum Considered Earthquake (MCE) ground shaking. This is the most severe level of ground shaking that is deemed appropriate for consideration in the design process for building structures, though not necessarily the most severe level of ground shaking that could ever be experienced at a site. In most regions, this ground shaking has a 2% probability of exceedance in 50 years, or roughly a 2,500 year mean recurrence interval. In regions of very high seismicity, near major active faults, the MCE ground shaking level is limited by a conservative, deterministic estimate of the ground shaking resulting from a maximum magnitude earthquake on the known active faults in the region. Although the probability that such deterministic ground shaking will be experienced at a site can vary considerably, depending on the activity rate of the individual fault, in most near-fault sites MCE ground shaking has approximately a 10% probability of exceedance in 50 years.

2. Design Earthquake (DE) ground shaking. This is the ground shaking level upon which design lateral forces, used as the basis for analysis and design provisions of the code, are based. It is defined as a spectrum that is 2/3 of the shaking intensity calculated for the MCE spectrum. The probability that DE ground shaking will be experienced varies, depending on the regional seismicity.

Commentary: The mean recurrence interval for Design Earthquake ground shaking will vary depending on regional seismicity. In areas of low seismicity the hazard return period will generally range between 750-1250 years, whereas in areas of high seismicity the recurrence interval may range between 300-600 years.

Performance evaluation, conducted in accordance with these guidelines, may be conducted for any level of ground shaking. The ground shaking may be determined probabilistically, i.e., based on the probability that shaking of the specified intensity will be experienced at a site; or it may be defined in a deterministic manner, based on a specified magnitude event occurring along a specific fault or source. Regardless of the method used to define the design ground shaking levels, the ground shaking must be characterized by an acceleration response spectrum or suite of ground motion time histories compatible with that spectrum, and also a hazard curve that expresses the probability that shaking of given intensity is felt at a site within a period of time. *FEMA-273* provides guidelines for development of ground motion response spectra for hazards of different probabilities of exceedance. FEMA-273 also provides approximate hazard parameters, for different regions, that may be used in place of a site specific hazard curve. These hazard parameters are repeated in these guidelines..

Commentary: Detailed guidelines on ground motion estimation and characterization are beyond the scope of this publication. Those interested in such information are referred to the Commentary to the NEHRP Provisions, the FEMA-273 Rehabilitation Guidelines, and references noted therein.

3.2.1.3 Other Hazards

In order to reliably predict the probable performance of a structure, it is necessary to determine if earthquake hazards other than ground shaking, including direct ground fault rupture, liquefaction, lateral spreading, and land sliding are likely to occur at a site and the severity of these effects. The severity of ground fault rupture, lateral spreading and land sliding is characterized by an estimate of permanent ground deformation. The severity of liquefaction is characterized by an estimate of the potential loss in bearing strength of subsoil layers and permanent ground settlement. In order to determine the performance of a structure which is subject to these hazards, the effects of the projected ground displacements should be evaluated using a mathematical model of the structure. The severity of these hazards used in performance evaluation should be compatible with that used in specification of ground shaking hazards.

Commentary: Most sites are not at significant risk form earthquake hazards other than ground shaking. However, these hazards can be very destructive to structures located on sites where they will occur. Accurate determination of the propensity of a site to experience these hazards requires site-specific study by a competent earth scientist or geotechnical engineer. Guidelines on such assessments are beyond the scope of this publication.

3.2.2 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 3-1 describes the overall levels of structural and nonstructural damage that may be expected of buildings when subjected to different levels of ground shaking. These performance descriptions are estimates rather than precise predictions, and variation among buildings of the same Performance Level must be expected. The structural performance levels are presented in Section 3.2.2.1.

Table 3-1 - Building Performance Levels

	Building Performance Levels	
	Collapse Prevention Level	Incipient Damage
Overall Damage	Severe	Light
General	Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, ceilings, and structural elements. Elevators can be restarted. Fire protection operable.
Nonstructural components	Extensive damage.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.
Comparison with performance intended for SUG-1 buildings when subjected to the Design Earthquake	Significantly more damage and greater risk.	Much less damage and lower risk.

Commentary: Building performance is expressed in terms of building performance levels. These building performance levels are discrete damage states selected from among the infinite spectrum of possible damage states that MRSF buildings could experience as a result of earthquake response. The particular damage states identified as building performance levels have been selected because these performance levels have readily identifiable consequences associated with the post-earthquake disposition of the building that are meaningful to the building user community and also because they are quantifiable in technical terms. These include the ability to resume normal functions within the building, the advisability of post-earthquake occupancy, and the risk to life safety.

Although a building's performance is a function of the performance of both structural systems and nonstructural components and contents, only the structural performance levels are defined in the Guidelines. The reference to nonstructural components above is to remind the reader of the probable performance of these elements at the various performance levels.

These guidelines only address methods of evaluating structural performance of MRSF structures. Although the performance of nonstructural components of buildings are critically important to the way in which buildings are used following an earthquake, treatment of this topic is beyond the scope of this document. FEMA-273 provides a more complete set of recommendations with regard to evaluating the performance of nonstructural components.

3.2.2.1 Structural Performance Levels

Two discrete structural performance levels are defined in the guidelines. Acceptance criteria, which relate to the permissible earthquake-induced forces and deformations for the various elements of MRSF structures, are tied directly to these structural performance levels. The performance levels are discrete damage states for which specific acceptance criteria are defined.

Commentary: In addition to performance levels, FEMA-273 incorporates the concept of performance ranges. These performance ranges, rather than representing discrete damage states, span the entire spectrum of potential damage states between no damage and total damage. No acceptance criteria are provided for the performance ranges in FEMA-273. These must be determined on a project specific basis, by interpolation or extrapolation from the performance levels. Performance ranges, as such, are not defined in these guidelines. However, compatible with the FEMA-273 approach, the user has the ability to create their own, custom performance levels, and to develop acceptance criteria for these levels, based on interpolation between the two performance levels, as suits a specific project.

Structural Performance Levels are the Incipient Damage Level and the Collapse Prevention Level. Table 3-2 relates these structural performance levels to the limiting damage states for common vertical elements of MRSF structures. Later sections of these Guidelines specify design parameters (such as inter-story drift limits, inelastic deformation demands, component capacities, and inelastic deformation demands) recommended as limiting values for calculated structural deformations and stresses for different structural components, in order to attain these structural performance levels for a known earthquake demand.

Table 3-2 - Structural Performance Levels

		Structural Performance Levels	
Elements	Type	Collapse Prevention	Incipient Collapse
Girder		Extensive distortion. A few girders may experience fracture	Minor local yielding at a few places.
Column		Moderate distortion; some columns experience yielding. Some local buckling of flanges	No observable damage or distortion
Connection		Many fractures (X% of total ?) and/or extensive yielding	No observable fractures; minor yielding at some connections
Panel Zone		Extensive distortion	Minor distortion
Column Splice	Ductile Splices	Fractures at some locations	No yielding
Base Plate		Extensive yielding of anchor bolts and base plate	No observable damage or distortion
Drift	Inter-story	5% transient or permanent	0.7% transient; negligible permanent

3.2.2.1.1 Incipient Damage Performance Level

Structural Performance Level, Incipient Damage,, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to re-occupancy.

3.2.2.1.2 Collapse Prevention Performance Level

Structural Performance Level, Collapse Prevention, 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, aftershock activity could credibly induce collapse.

Commentary: When a building is subjected to earthquake ground motion, a pattern of lateral deformations that varies with time is induced into the structure. At any given point in time, a particular state of lateral deformation will exist in the structure, and as some time within the period in which the structure is responding to the ground motion, a maximum pattern of deformation will occur. At relatively low levels of ground *motion, the deformations induced within the building will be limited, and the resulting stresses which develop within the structural components will be within the elastic range of behavior. Within this elastic range, the structure will experience no damage. All structural components will retain their original strength, stiffness and appearance, and when the ground motion stops, the structure will return to its pre-earthquake condition.*

At more severe levels of ground motion, the lateral deformations induced into the structure will be larger. As these deformations increase, so will demands on the individual structural components. At different levels of deformation, corresponding to different levels of ground motion severity, individual components of the structure will be strained beyond their elastic range. As this occurs, the structure starts to experience damage in the form of buckling, yielding and fracturing of the various components. As components become damaged, they degrade in stiffness, and some elements will begin to lose their strength. In general, when a structure has responded to ground motion within this range of behavior, it will not return to its pre-earthquake condition when the ground motion stops. Some permanent deformation may remain within the structure and damage will be evident throughout. Depending on how far the structure has been deformed, and in what pattern, the structure may have lost a significant amount of its original stiffness and, possibly, strength.

Brittle elements are not able to sustain inelastic deformations and will fail suddenly; the consequences may range from local and repairable damage to collapse of the structural system. At higher levels of ground motion, the lateral deformations induced into the structure will strain a number of elements to a point at which the elements behave in a brittle manner, or as a result of the decreased overall stiffness, the structure loses stability. Eventually, partial or total collapse of the structure can occur. The structural performance levels relate the extent of a building's response to earthquake hazards to these various possible damage states.

At the Incipient Damage Level, damage is relatively limited. The structure retains a significant portion of its original stiffness and most if not all of its strength. At the Collapse Prevention level, the building has experienced extreme damage. If laterally deformed beyond this point, the structure can experience instability and collapse. The range between these two levels represent conditions in which there is some identifiable margin for additional lateral deformation before collapse would occur. FEMA-273 also includes consideration of a Life Safety level, intermediate between the damage states represented by Incipient Damage and Collapse Prevention. The Life Safety level is defined in FEMA-273 as occurring at 75% of the lateral displacement at which Collapse Prevention occurs.

Given this circular definition of the Life Safety level, and the fact that the NEHRP Provisions have moved towards designing for Collapse Prevention, as opposed to Life Safety performance, this performance level has been omitted from these guidelines.

3.3 Evaluation Approach

The basic process of performance evaluation, as contained in these guidelines is to develop a mathematical model of the structure and to evaluate its response to the earthquake hazards by one or more methods of structural analysis. 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. For each performance level, these guidelines specify acceptance criteria for each of the design parameters indicated above. Acceptance criteria are limiting values for the various design parameters, at which damage corresponding to the specific 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 a structure to actually provide specific performance at selected probability of exceedance.

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 provided by a design, with regard to specific performance objectives, based on the ratio of factored capacity to factored demand. If the predicted level of confidence is inadequate, then either more detailed investigations and analyses should be performed to improve the level of confidence attained with regard to performance, through the attainment of better understanding of the structure's behavior and modification of the load and resistance factors, or alternatively, the structure should be upgraded such that a sufficient level of confidence can be attained given the level of understanding obtained. If it is deemed appropriate to upgrade a structure to improve its probable performance, an iterative approach consisting of trial designs, followed by verification analyses, evaluation of design parameters against acceptance criteria, and calculation of confidence level is repeated until an acceptable design solution is found.

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 3.4 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 performance evaluation. The purpose of this LRFD approach is to develop estimates of the confidence level inherent in a design with regard to a specific performance objective (probability of exceeding a specified performance level, within a 50 year period).

The basic process starts with the selection of a performance objective. This consists of the specification of a performance level and a desired probability of exceedance for this performance level in a 50 year period (P_{E50}). Once this probability of exceedance is selected, two hazard parameters are determined, from the site hazard curve. These are the value of spectral response acceleration S_a at the fundamental period of the structure for the selected hazard level (P_{E50}) and the slope of the hazard curve, k, in logarithmic coordinates, evaluated at the P_{E50}.

Using the S_a value appropriate to the hazard probability, a structural analysis is performed to determine the maximum inter-story drift demand for the structure. 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:

$$\lambda = \beta e^{\left(\frac{k}{2b}\Sigma \sigma_i^2\right)} \tag{3-1}$$

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.*

The factored demand, calculated from the analysis represents a mean estimate of the probable maximum inter-story drift demand. These guidelines also tabulate permissible inter-story drifts for the various performance levels, dependent on frame and connection configuration, as well as capacity factors, that similarly adjust the estimated capacity of the structure to a mean value. Guidelines are provided in Chapter 6 for determination of ϕ *factors for connections for which project specific qualification testing is performed.*

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

$$\gamma_{con} = \frac{\phi c}{\lambda D} \tag{3-2}$$

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.*

3.4 Analysis

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 3.5.

3.4.1 Alternative Procedures

Four alternative analytical procedures are available for use in systematic performance evaluation of WMSF structures. The basic analytical procedures are described in Section 3.4, which provides supplementary guidelines on the applicability of the *FEMA-273* procedures and also provides supplemental modeling recommendations. The four basic procedures are:

- Linear static procedure (LSP) - an equivalent lateral force technique, similar, but not identical to that contained in the building code provisions
- Linear dynamic procedure (LDP) - an elastic, modal response spectrum analysis or an elastic time history analysis
- Nonlinear static procedure (NSP) - a simplified nonlinear analysis procedure in which the forces and deformations induced by a monotonically increased pattern of lateral loading considering the degradation of strength and stiffness associated with material and element nonlinearity.
- Nonlinear dynamic procedure (NDP) - 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 performance evaluation process is to predict the values of key response parameters, that are indicative of the structure's performance, when it is subjected to a specific ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy using the basic equation:

$$\gamma_{con} = \frac{\phi c}{\lambda D} \tag{3-3}$$

where:

λ = *a load factor to account for uncertainty in the prediction of demands (the value of the response parameters)*
D = *the predicted demand*
ϕ = *a capacity reduction factor to account for uncertainty in the capacity of the structure*
C= *the capacity for the 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 performance evaluation, 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 under the design ground motion.

The ability of the performance 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. 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.

3.4.2 Procedure Selection

Table 3-3 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 3-3 - Recommended Analysis Procedures

Performance Level	Analysis Procedure			
	Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Incipient Damage	Permitted for	Permitted for	Permitted for	Permitted for

	regular structures, per the NEHRP Provisions $\lambda = 1.3$	structures of any configuration $\lambda = 1.0$	structures of any configuration $\lambda = 1.2$	structures of any configuration $\lambda = 1.0$
Collapse Prevention	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 $\lambda = 1.0$

3.4.3 Linear Static Procedure (LSP)

3.4.3.1 Basis of the Procedure

Linear static procedure analysis of 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 to be checked using the applicable acceptance criteria of Section 3.5. 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 3.5.

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 equivalent static lateral forces. The magnitude of the equivalent lateral forces is selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating, on average, maximum displacements that are *expected during the earthquake ground motion. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the yielding building will be less than the internal forces calculated on an elastic basis.*

The performance of MRSF structures is most closely related to total 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.

3.4.3.2 Modeling and Analysis Considerations

3.4.3.2.1 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:

$$T = C_t h_n^{3/4} \tag{3-4}$$

where

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

C_t = 0.035 for welded 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:

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \tag{3-5}$$

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.

3.4.3.3 Determination of Actions and Deformations

3.4.3.3.1 Equivalent Lateral Load

An equivalent lateral load, given by equation 3-6, shall be independently calculated for each of two orthogonal directions of building response, and applied to a mathematical model of the building structure.

$$V = C_1 C_2 C_3 S_a W \tag{3-6}$$

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 3-4

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

T = Fundamental period of the 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.

C_2 = Modification factor to represent the effect of fractures, stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Performance Levels are listed in Table 3-4. 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 3-7) 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 4-4 may be used.

$$\theta = \frac{P\Delta}{VH} \qquad (3\text{-}7)$$

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 3-4 - Modification Coefficients for Linear Static Procedure

Performance Level	C1	C2	C3
Immediate Occupancy			
PR Connections	1.0	1.2	1.2
FR Connections	1.0	1.0	1.0
Collapse Prevention T< 1.0 Sec T ≥ 1.0 Sec	 2.0 1.0		
PR Connections		1.2	1.0
Ductile FR Connections		1.1	1.2
Brittle FR Connections		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 (3-6) may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in Section 3.5 are developed to take this aspect into account.

3.4.3.3.2 Vertical Distribution of Seismic Forces

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

$$F_x = C_{vx} V \tag{3-8}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{3-9}$$

where

k = 1.0 for T ≤ 0.5 second

= 2.0 for T ≥ 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 = Equivalent lateral load from Equation (3-6)

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

3.4.3.3.3 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building should be distributed according to the distribution of mass at that floor level.

3.4.3.3.4 Floor Diaphragms

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

3.4.3.3.5 Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral loads in accordance with Equations 3-6, and 3-8 and stiffnesses obtained from Section 3.5. 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 3-2.

3.4.3.3.6 Determination of Column Demands

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

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \qquad (3\text{-}10)$$

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 3-5

Table 3-5 Value of Load Factors λ_c for Columns - Linear Static Procedure

Column Located In	$\overline{M/M_p}^{\,1}$

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

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.

3.4.4 Linear Dynamic Procedure (LDP)

3.4.4.1 Basis of the Procedure

Linear dynamic procedure analysis of 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 3-4.

> *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.*

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 3.5. Calculated displacements are factored by the applicable load factor, λ, obtained from Table 3-3 and compared with factored acceptable values, per Section 3.5. 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 3.5.

3.4.4.2 Modeling and Analysis Considerations

3.4.4.2.1 General

The LDP should conform to the criteria of this section. The analysis should be based on appropriate characterization of the ground motion. The modeling and analysis considerations set forth in Section 3.5 should apply to the LDP but alternative considerations are presented below.

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.

3.4.4.2.2 Ground Motion Characterization

The horizontal ground motion should be characterized by one of the following methods:

- An elastic response spectrum, developed in accordance with the Guidelines of FEMA-273 for the appropriate hazard return period
- Ground acceleration time histories that are compatible with such a response spectrum, as indicated in FEMA-273

3.4.4.2.3 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 the base shear for 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.

3.4.4.2.4 Response-History Method

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 time histories prepared according to the guidelines of FEMA-273, 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.

3.4.4.3 Determination of Actions and Deformations

3.4.4.3.1 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 3.4.3.2 and by the applicable λ obtained from Table 3-3.

3.4.4.3.2 Determination of Column Demands

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

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \tag{3-10}$$

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 3-6

Table 3-6 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.

3.4.5 Nonlinear Static Procedure (NSP)

3.4.5.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the nonlinear material and geometric response characteristics is displaced to a target displacement, and resulting internal deformations and forces are determined. The nonlinear load-deformation characteristics of individual components and elements of the building are modeled directly. The mathematical model of the building is subjected to a pattern of monotonically increasing lateral forces 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, during the design earthquake. 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 3.4.5.3. 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 design earthquake, 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 3.5. Calculated inter-story drifts and column and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

3.4.5.2 Modeling and Analysis Considerations

3.4.5.2.1 General

In the context of these *Guidelines*, the NSP involves the monotonic application of lateral forces 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 3-12. Performance evaluation 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, and connections.

3.4.5.2.2 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.

3.4.5.2.3 Lateral Load Patterns

Lateral loads should be applied to the building in profiles that approximately bound the likely distribution of inertia forces in an earthquake. 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 3-9, 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.

3.4.5.2.4 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:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (3\text{-}11)$$

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 3-1 for further information.

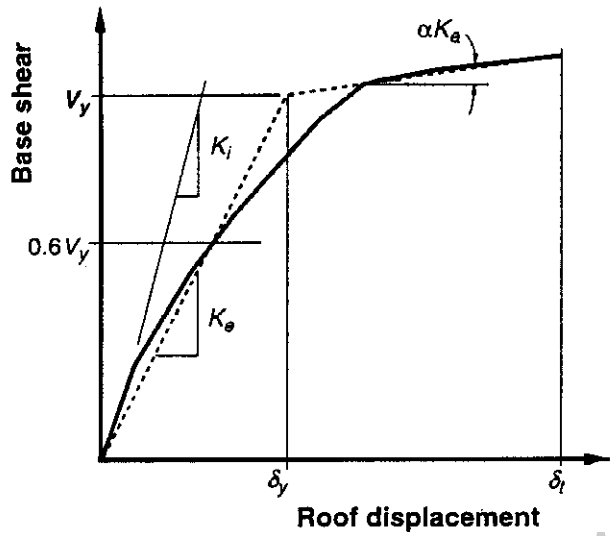


Figure 3-1 - Calculation of Effective Stiffness, K_e

3.4.5.2.5 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.

3.4.5.2.6 Analysis of Two-Dimensional Models

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

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.

3.4.5.3 Determination of Actions and Deformations

3.4.5.3.1 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:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \qquad (3\text{-}12)$$

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 3-7

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 3.4.3.3. 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 3.4.3.3.

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 3.4.3.3.

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:

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \quad (3\text{-}13)$$

Table 3-7 **- 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 3-1).

W = Total dead load and anticipated live load, as calculated in Section 3.4.3.3.

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

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \quad (3\text{-}14)$$

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 3-1)

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 3-12. 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 3-12 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 design response spectrum. The target displacement so calculated should be no less than that displacement given by Equation 3-12, 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 3.5 to account for system torsion.

3.4.5.3.2 Floor Diaphragms

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

3.4.5.3.3 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 3-3.

3.4.5.3.4 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 3-3.

3.4.6 Nonlinear Dynamic Procedure (NDP)

3.4.6.1 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 neccesary 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 the design earthquake.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 3.5. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values for the applicable performance level.

3.4.6.2 Modeling and Analysis Assumptions

3.4.6.2.1 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 3.5 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.

3.4.6.2.2 Ground Motion Characterization

The earthquake shaking should be characterized by ground motion time histories, prepared in accordance with the recommendations of FEMA-273. A minimum of three pairs of ground motion records shall be used..

3.4.6.2.3 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 3.5. The requirements of Section 3.5 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.

3.4.6.3 Determination of Actions and Deformations

3.4.6.3.1 Modification of Demands

The effects of torsion should be considered according to Section 3.5.

3.4.6.3.2 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 3-3.

3.4.6.3.3 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 3-3.

3.5 Mathematical Modeling

3.5.1 Basic Assumptions

3.5.1.1.1 Modeling Approach

In general, a 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.

3.5.2 Frame Configuration

The analytical model should accurately account for the stiffness effects of frame connections. Element and component stiffness properties and strength estimates for both linear and nonlinear procedures can be determined from information given in Chapter 6 for pre-qualified connections.

3.5.3 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.

3.5.4 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.

3.5.5 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.

3.5.6 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.*

3.5.6.1 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:

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \qquad (3\text{–}15)$$

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 $\theta = mg/(Kh)$ 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 $\theta > \alpha$', 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.

3.5.6.2 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.

3.5.7 Elastic Section 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 the earthquake. 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 flexural 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.

3.5.8 Connection Flexibility

Panel zone stiffness may be considered in the frame analysis by adding a panel zone element to the model. The beam flexural stiffness may also be adjusted to account for panel zone stiffness or flexibility and the stiffness of concrete encasement. Centerline analysis shall be used for other cases.

The modeling of stiffness for connections for Fully-Restrained (FR) connections is not required since, by definition, the frame displacements are not significantly (<5%) affected by connection deformation. The strength of the connection must be evaluated to determine if it can carry the expected moment and shear demand generated in the beam and the beam-to-column connection.

3.5.9 Nonlinear Properties

The elastic component properties, as outlined in section 3.5.7., shall be used. Appropriate nonlinear moment-rotation and interaction relationships should be used for beams and beam-columns to represent plastification.

3.5.9.1 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.

3.6 Acceptance Criteria

Acceptability of building performance shall be determined through evaluation of the relationship:

$$\gamma_{con} = \frac{\phi C}{\lambda D} \tag{3-16}$$

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

for each of the performance parameters indicated in Table 3-8. 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 3-9, or through direct calculation of confidence level through the procedures of Section 3.7. 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 3-8 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 3.6.1
Column Axial Load	The adequacy of each column to withstand the calculated maximum compressive load for that column shall be evaluated. Refer to Section 3.6.2
Column Splice Tension	The adequacy of column splices to withstand calculated maximum tensile demands for the column shall be evaluated. Refer to Section 3.6.3

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.*

Table 3-9 Confidence Levels for Various Values of γ_{con} for Different Analytical Approaches

Analysis Procedure	Linear Static Procedure					Linear Dynamic Procedure					Nonlinear Static Procedure					Nonlinear Dynamic Procedure				
Confidence Level	50	65	84	90	95	50	65	84	90	95	50	65	84	90	95	50	65	84	90	95
Geographic Region																				
California	.2	.5	1	1.2	1.5	.3	.6	1.1	1.3	1.5	.4	.7	1.1	1.3	1.5	.5	.8	1.2	1.3	1.5
Pacific N.W.	.6	.9	1.3	1.4	1.6	.5	.8	1.3	1.5	1.7	.4	.7	1.3	1.5	1.8	.3	.6	1.2	1.5	1.9
Intermountain	.6	.9	1.3	1.4	1.6	.5	.8	1.3	1.5	1.7	.4	.7	1.3	1.5	1.8	.3	.6	1.2	1.5	1.9
Central U.S.	.6	.9	1.4	1.6	1.7	.5	.9	1.5	1.7	2.0	.5	.8	1.5	1.8	2.2	.4	.8	1.6	1.9	2.4
Eastern U.S.	.6	.9	1.4	1.6	1.7	.5	.9	1.5	1.7	2.0	.5	.8	1.5	1.8	2.2	.4	.8	1.6	1.9	2.4

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.

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 3-3, the load factors associated with nonlinear analysis approaches are generally lower than those associated with the linear approaches. Similarly, as reflected in Chapter 6, connections that have exhibited consistent behavior in laboratory tests are generally assigned larger resistance factors, than do connections with inconsistent behaviors, to reflect the reduced uncertainty with regard to predicting their behavior.

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 at the target probability of exceedance, 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 at the target annual probability of exceedance. 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 average regional values for the hazard parameters. Section 3.7 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}. Chapter 6 presents procedures for determining resistance factors, based on connection behavior. 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.

3.6.1 Interstory Drift Capacity

Inter-story drift capacity may be limited either by the global response of the structure, or by the local behavior of beam-column connections. Factored inter-story drift capacity, ϕC, shall be taken as the lesser of the product of the resistance factor ϕ and capacity C, obtained from Table 3-10, based on global response, or the product of the resistance factor ϕ and capacity C, obtained from Chapter 6 for the beam-column connections incorporated in the structure. In lieu of the values contained in Table 3-10, the more detailed procedures of Section 3.7.1 may be used to determine inter-story drift capacity as limited by global building response.

3.6.2 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.

3.6.3 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:

$$x=a+b \tag{3-17}$$

Table 3-10 Inter-story Drift Capacity and as Limited By Global Response, and Associated Resistance Factors

Structure Type	Incipient Damage		Collapse Prevention	
	Inter-story Drift Capacity	Resistance Factor ϕ	Inter-story Drift Capacity	Resistance Factor ϕ
Low Rise -(3 above grade stories or less)	0.015	.75	.10	.6
Mid Rise - (4 or more above grade stories, but not more than 12 above grade stories)	0.015	.75	.08	.6
High Rise - More than 12 above grade stories	0.015	.75	.05	.6

3.7 Detailed Procedure for Determination of Performance 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 procedures may be used 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 3.7.1
- Development of a suite of ground motion accelerograms in accordance with Section 3.7.2
- Performance of a suite of dynamic pushover analyses in accordance with Section 3.7.3
- Calculation of factored drift capacity in accordance with Section 3.7.4
- Calculation of confidence index, γ_{con}, and inherent confidence in building performance, in accordance with Section 3.7.5

3.7.1 Hazard Parameters

A median hazard curve shall be developed for the site, indicating the annual probability of exceedance for various values of 5% damped spectral response acceleration at the fundamental period of the structure. The hazard curve shall be constructed using standard ground motion attenuation relationships, considering the activity rate of each of the faults and seismic source zones that contribute to the hazard at the site, and considering the affect of site response on the spectral character of ground shaking at the site. The slope of the hazard curve, *k*, in logarithmic (log - log) coordinates shall be determined.

Alternatively, a generalized 5% damped response spectrum, at the desired hazard level (annual probability of exceedance) may be constructed using the procedures of FEMA-273 and the slope of the hazard curve, *k*, may be approximately determined from Table 3-11.

Table 3-11 Approximate Hazard Parameter, *k*

Geographic Region	*k*
California	3
Pacific Northwest and Intermountain	2
Central U.S.	1
Eastern U.S.	1

3.7.2 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 3.7.1. The accelerograms shall be scaled to achieve spectral compatibility in accordance with the guidelines of FEMA-273.

3.7.3 Dynamic Pushover Analysis

A nonlinear mathematical model of the building shall be constructed. 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 3.7.3, 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. This plot is termed a dynamic pushover plot.

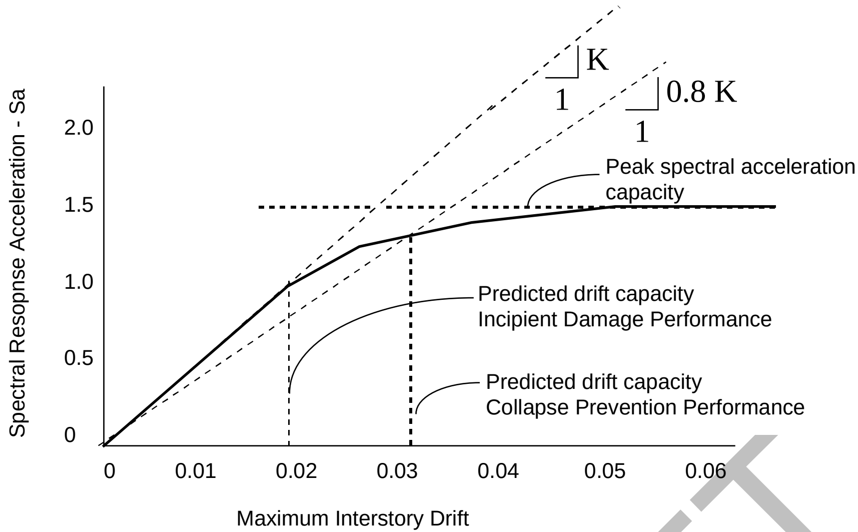


Figure 3-2 Dynamic Pushover Curve

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 collapse prevention performance, for this ground motion. The inter-story drift at which the slope of the global pushover curve deviates from the slope of the initial portion of the curve shall be taken as the inter-story drift capacity for incipient damage performance. Refer to Figure 3-2. The inter-story drift capacity for collapse prevention performance shall not be taken as greater than 0.1.

3.7.4 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:

$$\phi = e^{-k\sigma_{\ln\delta}^2 / 2b} \qquad (3\text{-}18)$$

where: k = the slope of the hazard curve, determined in accordance with Section 3.7.1

b = a hazard parameter that may be taken as 1

$\sigma_{\ln\delta}$ = 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 3-18 and the median inter-story drift capacity determined from the dynamic pushover analyses.

3.7.5 Determination of Confidence Level

A performance confidence index, γ_{con}, shall be determined in accordance with Section 3.6, for each of the controlling performance parameters. The confidence parameter K_x, shall be determined from the equation, using the smallest of the values γ_{con}:

$$K_x = \frac{\ln(\gamma_{con})}{b\sigma_{UT}} + \frac{k\sigma_{UT}}{2} \qquad (3\text{-}19)$$

where:
k = the slope of the hazard curve, determined in accordance with Section 3.7.1
b ;= a hazard parameter that may be taken as 1.0
σ_{UT} = is a measure of the uncertainty related to prediction of drift demand, taken from Table 3-12.

Table 3-12 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

Table 3-13 - Values of Kx for Various Levels of Confidence

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

The level of confidence with regard to the target performance shall be determined by interpolation from, Table 3-13.

4. Loss Estimation

4.1 Scope

This section provides two alternative methods of performing financial loss estimates for WSMF buildings. When a building is damaged by an earthquake, there are a number of potential sources of loss. The principal sources of this loss are the costs associated with repairing the damage and restoring the building to service as well as the loss of revenue resulting from an inability to occupy space in the damaged structure until it is repaired. Losses could also occur from other sources including the need to rent space for temporary or alternate quarters, relocation costs, litigation, devaluation of property values and a general decline in the economic welfare of the affected region. These guidelines provide methods for estimating only the following losses:

- probable repair costs for a structure, expressed as a percentage of building replacement value
- probable time to conduct repairs

It should be recognized that actual repair cost and time to effect repairs is a function of a number of complex factors including the severity of damage, the availability of design professionals and contractors to work on a building, the availability of funds to implement the repairs, ability to relocate building tenants, etc. Consequently, there is a significant scatter to data on actual repair costs and repair times and any estimate of loss must inherently include significant uncertainty.

4.2 Loss Estimation Techniques

Guidelines for two alternative techniques are provided to estimate probable repair costs for WSMF buildings, in the event that they are affected by future strong earthquake ground motion. The first technique, a Rapid Loss Estimation method, permits estimates of losses to be developed based on limited data on the building size and configuration and estimates of ground motion intensity. The second technique utilizes engineering data obtained from a detailed performance evaluation of the specific building, conducted in accordance with Chapter 3.

Commentary: Several different methodologies are commonly used to perform loss estimates. These may be termed actuarial, expert opinion, and engineered. All of these methods inherently include significant uncertainty with regard to the predicted repair costs. Actuarial estimates are developed based on historical data on the actual costs incurred in the repair of structures of a given class, when subjected to ground motion of a certain intensity. When such a data base is available, it is possible to *determine the distribution of losses over the population contained in the database, including a median (best estimate of the loss for any structure in the class) and a coefficient of variation. This permits the loss for a structure similar to those contained in the database to be estimated within a level of confidence, using the median and coefficient of variation. This is the approach adopted in the rapid loss estimation methodology. Although it is potentially the most accurate of any of the approaches, there are several significant sources of uncertainty including the completeness (or incompleteness) of the database, the similarity of the structure being evaluated to structures included in the database, and the similarity of economic conditions at the time the data base losses occurred to those that may exist when the building experiences and event. Such data bases are rarely complete or accurate. The best data bases are obtained from individual insurance companies. However, there databases include data only on claims they experienced, which may not be representative of the regional experience, may be exaggerated, or under-represented, depending on the company's underwriting and adjustment policies, and may not include data on undamaged buildings. The data base used in this study was obtained by conducting a survey of engineering and construction firms in the Los Angeles region.*

The most commonly used loss estimation methodology is based on distributions of expert opinion of probable repair costs. ATC-13 and other similar studies have developed damage functions by obtaining opinions from structural engineers and other experts on typical levels of damage for various classes of structures when subjected to different intensities of ground motion. Statistical data from such a database can then be used in the manner previously described to derive loss estimates for other buildings. This approach has greater uncertainty associated with it and no direct tie to actual losses experienced in past events, other than as perceived by the experts at the time they provided the opinions which form the data base. Often the experts involved in this process had different concepts as to the "model building' for which they provided data and it may be difficult to determine if a building for which this methodology is used, is matched by the model buildings included in the experts' consideration.

In the third approach, engineering calculations are performed to estimate the types of damage actually likely to be experienced by the structure and probable repair costs are then determined based on this damage. Such an approach has not been widely used in the past. However, within the last few years, the National Institute of Building Sciences has prepared a general loss estimation methodology, HAZUS, that employs a generalized version of this approach. In the HAZUS *model, a standard pushover curve has been developed for each model building type and a loss function has been fitted to each pushover curve. Loss estimates are determined by estimating a displacement demand on the structure, based on ground motion parameters, determining where on the pushover curve for the model building this displacement demand occurs, and converting this to a loss using the loss function. Uncertainty is included in this calculation through the function that converts the pushover information into loss estimation. This approach is appealing in that it allows detailed analytical data on the structural behavior of a building to be directly used in the loss estimation process, a capability that neither of the other two loss estimation approaches permits. However, unless the loss conversion functions are benchmarked against actual loss data, estimates derived from this methodology are still very much dependent on expert opinion for conversion of structural performance to building loss. This approach has been adopted for the detailed loss estimation method contained in these Guidelines.*

4.2.1 Use of Loss Estimation Data

The information obtained from these techniques may be considered, together with other data, when making investment decisions relative to such buildings, or when conducting cost-benefit studies to determine if structural upgrade of existing buildings is economically justified.

4.2.2 Damage Data Included in the Methodology

Economic losses resulting from earthquake induced building damage include direct costs resulting from inspection to determine the extent of damage, engineering design fees, actual costs related to the structural repairs, demolition and replacement costs for architectural finishes and utilities (that must be removed to allow access for inspection and repair), and repair of damaged non-structural components, as well as indirect costs resulting from loss of use, lost income from rents that are not collected on spaces vacated during the repair period, and project financing costs. The loss estimation data obtained in accordance with the methodologies of this section only includes consideration of the direct damage repair costs. It does not include consideration of indirect costs related to lost rents, interruption of business and similar issues. These indirect costs often result in a greater economic impact than do the actual costs of repair, but are difficult to estimate on a general basis. Allowance for such indirect costs should be made in any economic analysis conducted for individual buildings.

4.3 Rapid Loss Estimation

For the purpose of this draft, the loss estimation methodology contained in FEMA-267 has been carried forward as a placeholder. It is intended to replace this model with an updated version, based on more comprehensive data from the Northridge earthquake.

4.3.1 Introduction

The loss estimation data presented in this section is compatible with that presented in *ATC-13* (Applied Technology Council -1985), a document frequently used as the basis for loss estimation studies. In that document, vulnerability functions are presented for broad classes of buildings, based on the expert opinion of groups of individuals familiar with the performance of those structures. The vulnerability functions relate the expected repair costs, expressed as a percentage of building replacement value, to a ground motion parameter (Modified Mercalli Intensity), and a level of confidence.

Commentary: Both ATC-13 and the FEMA-267 model employ MMI as the basic ground motion parameter. MMI is a highly subjective parameter intended to be determined after the fact of an earthquake, based on observation of actual damage. Several models have been developed that allow estimation of MMI, before an earthquake occurs, based on acceleration attenuation information and site soil type. Recently developed loss estimation methodologies, including HAZUS have begun to abandon MMI and move towards spectral response parameters for representation of ground motion strength. This is appealing in that it accounts for some of the peculiar characteristics of ground motion that are destructive to buildings in a more direct way. However, spectral response parameters do not account for duration of strong shaking, where MMI, at least in an indirect manner, does. In the simplified methodology, spectral response displacement will be used as the primary ground motion index, rather, than MMI, perhaps with a modified to include duration effects.

Table 4-1 presents a proposed vulnerability function for WSMF mid-rise buildings typical of California construction prior to the Northridge Earthquake. Each column of the table provides an estimate of the percentage of the total population of these buildings within a region affected by ground motion of defined intensity, expected to have repair costs “d,” expressed as a percentage of building replacement value, within the indicated ranges. Figure 4-2 provides a plot of this data in a format which may be more useful for application to loss estimation estimates. The statistics contained in the table were calculated using a loss estimation model developed by Thiel and Zsutty (Thiel and Zsutty - 1987), and data obtained on the performance of 89 buildings affected by the Northridge Earthquake (Bonowitz and Youssef - 1995).

Table 4-1 - Estimated Distribution of WSMF Buildings[1] by Severity of Damage in Regions of Varying Ground Motion Intensity

Damage d[2]	Modified Mercalli Intensity			
	VII	VIII	IX	X
d≤5%	71%	57%	40%	30%
5%<d≤25%	21%	29%	34%	35%
25%<d≤50%	7%	12%	20%	26%
50%<d≤75%	1%	2%	5%	8%
75%<d≤100%	0%	0%	1%	1%

Notes:

1. WSMF buildings conforming to pre-Northridge Earthquake design and construction practice for regions of high seismicity (*UBC* seismic zones 3 and 4) {*NEHRP* Map Areas 6 and 7}.
2. "d' is the direct damage repair cost, expressed as a percentage of building replacement cost

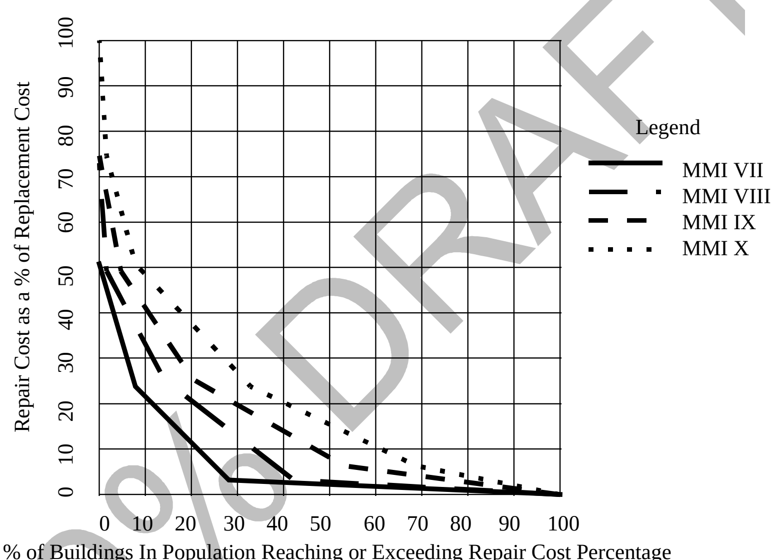


Figure 4-1 - Vulnerability Estimates for WSMF Buildings Conforming to Typical California Practice Prior to the Northridge Earthquake

4.3.2 Limitations of Approach

These loss estimation statistics should be used with caution, when applied to individual buildings. The unique characteristics of any individual building, including the strength and stiffness of its lateral force resisting system, its inherent redundancy, its condition, and the quality of its construction, will affect the relative vulnerability of the building. The statistics presented may be considered as representative of average buildings, in general conformance with the applicable building code provisions. Buildings that have substantial deficiencies relative to those provisions would be expected to be significantly more vulnerable. Similarly, buildings that have superior earthquake resisting characteristics, relative to the requirements of the building code, would be expected to be less vulnerable.

4.3.3 Connection Damage Costs

The statistics contained in Table 4-1 were established based on case studies conducted by SAC of the damage experienced by selected buildings affected by the Northridge Earthquake. It appears that typical repair costs for structural damage to connections can range from about $7,000 per connection to approximately $20,000. These costs are dominated not by the structural work, but rather by costs related to mobilizing into discrete areas of the building, performing local demolition of finishes and utilities as required to gain access and to create a safe working environment, and reconstruction of these finishes and utilities upon completion of the structural work. The cost of the structural work itself tends to vary from about $2,000 for the simplest repairs of damage (type W1 and W2) to perhaps $5,000 or more for repairs of the most complex types. These cost estimates do not include allowances for hazardous materials abatement, which will be required if either asbestos containing materials or lead based paint are present in the original construction. Such materials are likely to be present in buildings constructed prior to about 1980. The above costs relate only to the restoration of connections. They do not include costs related to re-establishing vertical plumbness of the building, which may be impractical to accomplish, or costs related to repair of architectural, mechanical, and electrical components which are directly damaged by the building’s response to the ground motion. These statistics assume that the building is repaired, rather than demolished and reconstructed. It should be noted that at least one building, in Santa Clarita, was demolished and reconstructed rather than repaired. A number of factors may have contributed to the owner’s decision to take such action, however, it is clear that the cost associated with this decision was much greater than would be indicated by the statistics presented in this Section.

Commentary: The damageabilities indicated in Table 4-1 and Figure 4-1 were estimated based on statistics available on a data set of 89 buildings (Bonowitz and Youssef - 1995). From this data set, it was possible to establish the probability of a building incurring damage to a given percentage of its total connections. This data set also allowed estimation of the number of connections per square foot of floor space provided by a building. From these statistics, an estimated average repair cost per connection of $12,500 was applied against the probable number of damaged connections per square foot of floor space. Building value was taken as $125/square foot of floor space. This computation permitted calculation of the expected loss percentage to a typical building. This data was then entered into a loss estimation model developed by Thiel and Zsutty (Thiel and Zsutty - 1987). The model was developed to replicate damage statistics observed in historic earthquakes and extended to current construction types using, in part, the expert opinion results of ATC-13.

Ground motion is characterized in Table 4-1 and Figure 4-1 using Modified Mercalli Intensity (MMI). Although MMI has been the most common ground motion parameter used for loss estimation studies in the past, it is subjective and interpretation can be varied. MMI can only be assigned after an earthquake has occurred and is based on observation of damage and other effects that have actually occurred. It is dependent, to a very great extent, on the types of construction which are present in the affected region. The distributions of damage indicated in Table 4-1 and Figure 4-1 are considered appropriate for California, and other regions with similar seismic design and construction practices. However, these data may not be appropriate for other regions.

It should be noted that when the repair cost for a building approaches 60 per cent or more of its replacement value (d$\geq$60%) many owners will determine, based on a number of factors, that complete building replacement, rather than repair is warranted. Therefore, it is probable that the actual costs for repair of some buildings will be 100 per cent of the replacement value. This possibility has not been reflected in the development of the damage repair cost distributions presented in Table 4-1.

It should also be noted that the statistics used to develop the above vulnerability estimates were taken from an incomplete data set of buildings. The data set may or may not have been representative of the distribution of damage in the total set of buildings affected by the Northridge Earthquake. If the data set is biased, this is likely to be a bias towards buildings that are more heavily damaged, since the data was collected soon after the earthquake, when only those buildings most likely to have been damaged had been inspected. A review of the applicability of the statistics used for generating the vulnerability estimates should be conducted, when more complete data on the distribution of damage becomes available.

4.3.4 Adjustment Factors

The damage data presented in Table 4-1 and Figure 4-1 have been developed for regular, mid-rise WSMF buildings constructed in the Los Angeles area prior to 1994. Adjustment factors, which take into account the height of the building, its age, the level of redundancy and the existence of irregularities are presented in the following sections. These factors have been developed to directly modify the repair cost factor, “d,” presented in Table 4-1.

Commentary: The values presented below represent, to a large extent, placeholders for values to be identified as part of the study of existing buildings (Thiel). The "text' portion of the adjustment factors will, obviously, be adjusted depending on the results of the research. This draft includes a brief case for the factors and requires significant substantiation. Four adjustment factors are presented currently-the final number will likely vary depending on the results of the investigations.

4.3.4.1 Building Height Adjustment Factor

Performance of steel moment frames in recent earthquakes (Kobe 1995, Northridge 1994, and Mexico City 1985) indicates that damage to WSMF is likely to accumulate in specific stories of the building. Hence, damage to high-rise WSMF, if limited to a few, specific stories, is likely to result in lower damage factor relative to low-rise buildings, which are likely to sustain damage throughout. Using this logic, the following adjustment factors are recommended for low-, mid- and high-rise WSMF, as shown in Table 4-2.

Table 4-2 - Building Height Adjustment Factor

Building Description	Number of Stories	Adjustment Factor
Low-rise	1-3	1.2
Mid-rise	3-10	1.0
High-rise	> 10	0.9

Commentary: The definition of low-, mid-, and high-rise buildings needs to be correlated with the building damage statistics that are available. The adjustment factors above assume the "base" factors included in Table 4-1 represent mid-rise WSMF. Also, the factors, themselves, need to be validated based on the research findings.

4.3.4.2 Age Factor

Damage to older WSMF, for a variety of possible reasons, appears to be less than those constructed in the 1980's and 1990's. There are many reasons for the age factor values listed in Table 4-3, below. These include the following:

- Welding process (SMAW as compared to FCAW).
- Material properties of the steel, including actual tensile strength to yield ratio.
- Overmatch of the weld relative to the yield strength of the steel members.

Table 4-3 - Age Adjustment Factor

Building Age	Adjustment Factor
> 1980	1.0
< 1980	0.9

Commentary: The justification for the Age factor needs to validated through the research findings, including the state-of-art reports being developed by Roeder, Foutch, and Thiel. These reports may also indicate that the age factor be further subdivied into the factors "bulleted" above, depending on whether there is enough information to justify this level of refinement.

4.3.4.3 Redundancy Adjustment Factor

Research of damaged building indicates that the number of moment frame bays is an indicator of the damage level. There are several reasons why redundancy is an important factor relative to building damage. The key reason appears to be the size of the members which frame redundant buildings. The more frame bays in a building, the smaller the member sizes that are typically found. Connection tests of these smaller member sizes (Roeder) indicate that the connections of smaller beams are likely to perform better, and, hence, result in less damage, than their larger counterparts. An approach used in the design of new buildings correlates well with the observed damage data that has been collected and analyzed. A brief discussion of the redundancy/reliability factor, ρ, for new buildings is presented below.

The seismic provisions for new buildings, included in the 1997 Uniform Building Code and the 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings, include a measure of redundancy (and reliability) through the introduction of the ρ factor. This variable is determined based on the percentage of seismic force that is resisted by a frame bay relative to the based shear and the area of the diaphragm at the level under consideration. Values or ρ which exceed 1.5 indicate a system that, essentially, lacks redundancy. Values of ρ less than 1.0 indicate a highly redundant structure. The redundancy adjustment factors are given in Table 4-4.

Table 4-4 - Redundancy Adjustment Factor

Value of ρ	Adjustment Factor
> 1.5	1.2
1.0 - 1.5	1.0
< 1.0	0.9

Commentary: The validity of the redundancy adjustment factor needs to be confirmed. Discussions over the past few years indicate that there will likely be a correlation between redundancy and damage level, mainly due to the smaller members sizes that usually accompany designs that have a significant level of redundancy (ρ < 1.0). If further research bears this out, factors similar to those listed in Table 4-4 will be important to include. It is unlikely that the UBC code representation of rendundancy will be adequate for our purposes but is included as a starting point for our efforts.

4.3.4.4 Irregularity Adjustment Factor

Building irregularity, whether horizontal or vertical, can have a significant effect on the performance and, hence, damageability of buildings, including WSMF. The study of the existing buildings indicates that conditions such as soft stories and torsional irregularities has an effect on the final loss estimation estimates.. Factors which take into account system irregularity are included in Table 4-5, below. The irregularity categories are based on those included in the 1997 NEHRP Provisions.

Table 4-5 - Irregularity Adjustment Factor

Irregularity	Adjustment Factor
Stiffness Irregularity-Soft Story	1.2
Weight Irregularity	1.1
Vertical Irregularity	1.1
In-plane Discontinuity in Vertical Lateral-force-resisting Element	1.2
Discontinuity in Capacity-Weak Story	1.2
Torsional Irregularity	1.1
Reentrant Corners	1.0
Diaphragm Discontinuity	1.1
Out-of-plane Offsets	1.2
Nonparallel Systems	1.1

Commentary: In addition to determining the adjustment factors, the category of irregularity needs to be reviewed. It may turn out that only a few of these irregularities are important or can be isolated from the available data. The values shown below, as with all of the tables, are to be considered as placeholders.

4.4 Detailed Loss Estimation

4.4.1 Introduction

The methodology contained in this section is based on a model developed for the HAZUS project of the National Institute of Building Sciences. In this methodology, building vulnerability is represented by a pushover curve that relates building lateral deformation to spectral displacement demand produced by ground motion. Vulnerability is then converted to loss through a loss function. In the HAZUS model, a series of standard pushover curves have been developed representing the behavior of different classes of model buildings, such as WSMF - low rise, WSMF - mid-rise, etc. The methodology presented in this section is based on the HAZUS approach, however, rather than providing default, model building pushover curves, these guidelines require that a building specific pushover curve be developed.

Guidelines for development of pushover curves for buildings are contained in FEMA-273 and the supplemental instructions of Chapter 3 of this document. In lieu of developing a full pushover curve for use in the loss estimation methodology, it is permissible to develop an approximate pushover curve, using the guidelines of section 4.4.2. The following sections provide guidelines on conversion of either a detailed pushover curve, developed in accordance with the guidelines for the Nonlinear Static Procedure, of Chapter3, or an approximate pushover curve developed in accordance with Section 4.4.2, into a loss curve.

4.4.2 Approximate Pushover Curve

Development of an approximate pushover curve, for use in loss estimation entails the following steps:

a) A mathematical model of the building is developed and an elastic response spectrum analysis of the model is performed. The displacement of the roof at the center of mass of the roof, Δ_{RE}, and the total base shear, V_E obtained from the response spectrum analysis are noted.

b) Demand capacity ratios are calculated for all participating elements of the lateral-force-resisting frame, using the forces predicted by the response spectrum analysis. The demand capacity ratio is calculated as the moment in each element predicted by the analysis, divided by the expected plastic moment capacity of the element, as given by the equation:

$$DCR_i = \frac{M_{E\,i}}{Z_i F_{ye}} \qquad (4\text{-}1)$$

where:

DCR_i = the demand capacity ratio for member "i"

M_{Ei} = the demand on member "i" predicted by the response spectrum analysis

Z_i = the plastic modulus for member "i"

F_{ye} = the expected yield strength for member "i" determined in accordance with the recommendations of Chapter 2.

c) The largest of the demand capacity ratios, determined for all of the members is determined. The displacement at which first major yielding occurs is computed as:

$$\Delta_y = \frac{\Delta_{RE}}{DCR_{\max}} \tag{4-2}$$

d) A plastic mechanism analysis of the structure is performed, using the upper bound theorem, to determine the lateral side-sway mechanism that results in the least base shear capacity, assuming that the lateral shears are distributed in the same pattern as the inertial forces predicted by the response spectrum analysis. This base shear is denoted as V_{max}.

e) An approximate pushover curve is drawn, as indicated in Figure 4-1 and the following:

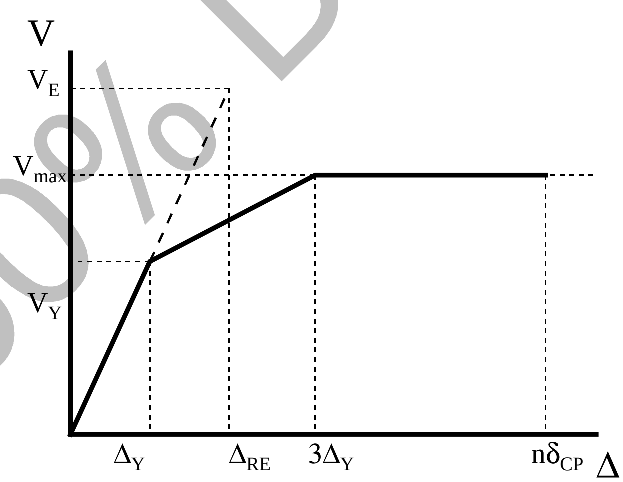


Figure 4-1 Approximate Pushover Curve

i) A domain consisting of a base shear versus roof deflection is drawn

ii) A broken line is drawn from the origin of this plot to the point V_E, Δ_{RE}.

iii) A solid line is drawn along this broken line to deflection , Δ_y. This defines the first yield force, V_y.

iv) A horizontal broken line is drawn at a force demand, V_{max}.

v) The solid line is extended from the coordinate V_y, Δ_y to the point V_{max}, 3 Δ_y.

vi) The solid line is extended horizontally from the point V_{max}, 3 Δ_y to the point V_{max}, $n\phi\delta_{CP}$, where n is the number of stories in the structure and $\phi\delta_{CP}$ is the mean value of the collapse prevention drift, determined in accordance with the procedures of Chapter 3.

4.4.3 Structural Vulnerability

Structural vulnerability is represented by a pushover curve, expressed in Spectral acceleration vs. spectral displacement coordinates. The standard pushover curve derived from a nonlinear static procedure (NSP) analysis of a structure, as outlined in Chapter 3, or an approximate pushover curve, developed in accordance with Section 4.4.2, above, should be used as the basis for this vulnerability curve. The standard pushover curve derived from an NSP is plotted as a function of base shear and lateral roof displacement. In order to obtain a vulnerability curve it is necessary to transform the pushover curve from these coordinates, respectively, into spectral acceleration, S_a, and spectral displacement S_d. This is done in a point by point manner, using the following equations:

$$S_{ai} = \frac{V_i / W}{\alpha_i} \qquad (4\text{-}3)$$

$$S_{di} = \frac{\Delta_{i\,roof}}{PF_1 \phi_{1,roof}} \qquad (4\text{-}4)$$

where:
S_{ai}= the spectral acceleration for point “i” on the pushover curve
S_{di}= the spectral displacement for point “i” on the pushover curve
V_i= the base shear at point “i” on a pushover curve
W= the weight of the structure
Δ_{iroof}= the displacement of point “i” on the pushover curve
α_i= the modal mass coefficient for the first mode given by the expression:

$$\frac{\sum \left[w_i \phi_{1,i} \right]^2}{\left[\sum w_i \right] \left[\sum w_i \phi_{1,i}{}^2 \right]} \qquad (4\text{-}5)$$

PF_i= modal participation factor for the first mode, given by the equation:

$$\frac{\sum w_i \phi_{1,i}}{\sum w_i \phi_{1,i}{}^2} \qquad (4\text{-}6)$$

$\phi_{1,roof}$ = amplitude of mode 1 at the roof level

$\phi_{1,I}$= amplitude of mode 1 at level "i"

w_i= weight assigned to level "i"

4.4.4 Building Loss

The building loss curve provides estimates of the expected cost to restore a structure to pre-earthquake condition, expressed as a percentage of building replacement value, at various levels of confidence, as a function of spectral displacement demand, at the fundamental period of the structure. The greater the level of confidence expressed in the estimate, the less likely that actual losses would experience the indicated amount. A 50% confidence level effort would be expected to be exceeded by half of the buildings for which an estimate is prepared. A 90% confidence level estimate would be expected to be exceeded by only 10% of the buildings for which an estimate is prepared. Loss curves are generated from the vulnerability curve and have the form indicated in Figure 4-2, in which loss, at a confidence level, is expressed as a function of estimated spectral response acceleration demand at the fundamental period of the structure.

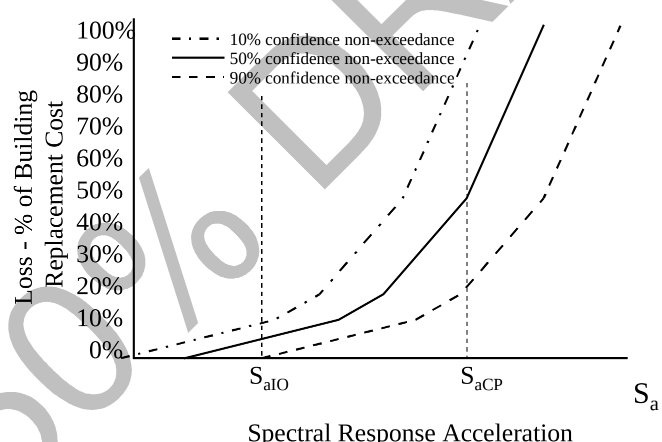


Figure 4-3 Standard Loss Curve

Note - the procedure for generating a loss curve, from a vulnerability curve is still under development. More detailed information will be provided in the next draft.

To estimate the probable loss for a structure, at a desired level of confidence, it is necessary to estimate the spectral response acceleration produced by the earthquake for which the structure is being evaluated, at the fundamental period of the structure. Estimates of ground motion parameters, such as spectral response acceleration, also have levels of confidence associated with them. For the purposes of this methodology, a median, 5% damped response spectrum for the earthquake of interest is used to determine the spectral response acceleration, at the fundamental period of the structure. The estimate of loss may then be read directly from the loss curve, at that spectral response acceleration, and the desired confidence level.

4.5 Repair Time

The amount of time a building, or portions of a building are out of service, while they are being repaired, is a critical factor in the actual losses experienced. The guidelines of this section provide a rough method of estimating potential repair times for buildings based on the estimate of direct monetary losses due to damage repair, as obtained either from the rapid loss estimation methodology of Section 4.3 or the detailed loss estimation methodology of Section 4.4.

Table 4-6 presents estimates of probable loss of service time, during repair, based on limited data from the Northridge earthquake and expert opinion. It is important to note that the actual lost service time during repair of a building is dependent on its size, the availability of the necessary resources (contractors, engineers, inspectors), the ability of the owner to bear the related expenses, and the efficiency of all parties involved in performing the work. These many complex factors are accounted for in the Table in only a general way and result in considerable amount of the uncertainty expressed in the table.

Table 4-6 - Probable Repair Times for Damaged WSMF Buildings

Loss Percentage - %	Estimated Days Out of Service for Repair	Affected Area
0	0	no loss of use
10	0.00067 - 0.0015 days/sq ft.	loss of use in area being repaired
20	0.001 - 0.002 days/sq ft.	loss of use in area being repaired
30	0.003 - 0.005 days/sq ft.	complete loss of use during repair
40	0.0033 - 0.0075 days/sq ft.	complete loss of use during repair
50	building not repaired	complete loss of use
60	building not repaired	complete loss of use
70	building not repaired	complete loss of use
80	building not repaired	complete loss of use
90	building not repaired	complete loss of use
100	building not repaired	complete loss of use

5. SEISMIC UPGRADE

5.1 Scope

Rehabilitation measures for steel components and elements of WSMF structures are described in this chapter. Information needed for simplified and systematic rehabilitation of steel buildings is presented herein.

5.2 Codes and Standards

The following codes and standards are applicable to seismic upgrades for steel frames, to the extent indicated below:

FEMA 273	Guidelines for Seismic Rehabilitation of Buildings - (provides general performance-based design methodology, as modified by these guidelines)
FEMA-301	NEHRP Recommended Provisions for Seismic Regulation of New Buildings and Other Structures (governing the detailing, materials and workmanship for new construction employed in an upgrade design)
AWS D1.1	Structural Welding Code (governing requirements for welding)
AISC	Seismic Design Provisions (as referenced herein)
AISC/LRFD	Specifications for the Design of Steel Structures (for requirements such as bolting, welding, computation of member nominal capacities)

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

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

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

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

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

5.3 Upgrade Objectives and Rehabilitation Criteria

Two approaches are available for seismic upgrade of WSMF structures - a Simplified approach and a Systematic approach.

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

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

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

5.3.1 Simplified Rehabilitation

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

In simplified rehabilitation, the individual beam-column connections of the existing lateral force-resisting system for the WSMF structure are modified to provide equivalent inter-story drift capacity to that required for a new WSMF structure having the same structural system. Existing WSMF structures will typically have been designed, either as Ordinary Moment Resisting Frames (OMFs) or Special Moment Resisting Frames (SMFs). The required inter-story drift capacities for these systems shall be as indicated in Table 5-1. Chapter 6 of these guidelines provides pre-qualification requirements for selected connection upgrades, that are accepted generically as being capable of providing these inter-story drift capacities indicated in Table 5-1, providing the requirements of the pre-qualification are complied with. Chapter 6 also provide project-specific qualification procedures that may be used to affirm that other connection upgrades provide the desired inter-story drift capacity.

Table 5-1 Factored Inter-story Drift Capacities for Simplified Upgrade

System	Inter-story Drift Capacity (Radians), $\phi\ \theta_i$
OMF	0.02*
SMF	0.04*

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

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

5.3.2 Systematic Rehabilitation

In systematic rehabilitation, a detailed performance evaluation of the structure is performed in its existing condition and its ability to meet desired performance objectives is determined. If the structure is found to be incapable of meeting the desired performance objectives, then structural modifications are performed to improve the probable performance. These modifications could include connection improvement measures, such as those available for simplified rehabilitation, but could also address systemic issues such as the basic strength and stiffness of the structure, the presence of irregularities or other vulnerabilities. An iterative process is followed in which a performance evaluation of the building is performed assuming proposed modifications are in place, and if the desired performance is not indicated, additional modifications performed.

Prior to performing a systematic seismic upgrade, one or more suitable performance objectives shall be selected as the basis for design. Performance objectives shall be selected in accordance with the guidelines of Section 3.2 of this document. A performance evaluation shall be conducted of the structure, to determine if it is capable of providing sufficient levels of confidence with regard to its ability to meet these performance objectives. If sufficient confidence is not attained, then upgrade modifications should be developed, either to reduce the response of the structure to earthquake ground shaking, such that acceptable confidence of achieving the desired performance is attained, or to increase the capacity of the structure to withstand earthquake response and provide acceptable confidence. Section 5.4 provides suggested upgrade strategies for use with systematic upgrade approaches.

Commentary: Performance objectives, selected in accordance with Section 3.2 of these guidelines are not completely compatible with those selected in accordance with FEMA-273. In FEMA-273, a performance objective is defined as consisting of two parts - a desired performance level, of which there are three (Immediate Occupancy, Life Safety, and Collapse Prevention) and a desired ground motion spectrum for which this performance level is not to be exceeded. In these guidelines, only two performance levels are defined (Incipient Damage and Collapse Prevention) and rather defining a specific ground motion spectrum for which the performance is to be attained (or not exceeded) the probability that the performance is to be attained (or not exceeded) in a defined number of years must be selected, together with a confidence level with regard to attainment of this performance.

The Incipient Damage level defined in these guidelines, may be taken as equivalent to the Immediate Occupancy level of FEMA-273. The Collapse Prevention level of these guidelines, may be taken as equivalent *to the Collapse Prevention level of FEMA-273. If it is desired to attain performance equivalent to the Life Safety level of FEMA-273, using these guidelines, this may be attained by using 75% of the acceptance criteria (drift capacities, strength capacities, etc.) specified in these guidelines for Collapse Prevention.*

To create performance objectives, using these guidelines, that are roughly equivalent to those contained in FEMA-273, it is necessary to associate a probability of exceedance, within a specified return period (e.g. 50 years) with the response spectrum used to define the hazard under the FEMA-273 criteria. Rehabilitation designs that provide a 95% confidence level for non-exceedance of the desired performance level at this probability shall be deemed equivalent to the intended performance of FEMA-273.

5.4 Upgrade Strategies

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

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

Commentary: A building's response to earthquake ground shaking results in the development of forces and deformations in the structure. In Chapter 3 of these guidelines, a procedure is defined for determining a level of confidence with regard to the ability of a structure to resist these forces and deformations with a defined probability of exceeding one or more performance levels. This confidence level is tied to the confidence parameter, γ_{con} calculated as the ratio of the factored capacity to resist these forces and deformations, ϕC to the factored demands (λD). Values *of the parameter γ_{con} larger than 1, indicate relative high confidence while values below 1, indicate progressively lower confidence.*

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

5.4.1 Connection Modifications

Connection modifications are intended to upgrade the ability of the individual connections to withstand expected rotational deformations with suitably low probability of unacceptable damage. This is judged to have been achieved when the ratio of factored inter-story drift capacity of the individual connections ($\phi\ \theta$) to withstand the factored demands ($\lambda\ \theta$) determined from an analytical evaluation of structural performance results in an acceptable confidence index, γ_{con}. Chapter 6 presents a series of pre-qualified connection upgrades, together with applicable capacity reduction factors, ϕ, and limiting inter-story rotation capacities, θ, for these various connection upgrades. Together with this data, design procedures for the connection upgrades and limiting parameters for which these upgrades are pre-qualified are presented. Chapter 6 also presents a project-specific connection qualification procedure for use in determining appropriate inter-story drift capacities and capacity factors, for connection upgrades that are not included in the pre-qualifications.

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

It is important to note that although connection upgrade strategies directly address the single most common vulnerability of WSMF structures - connections prone to premature brittle fracture, these upgrades can be quite costly, particularly in large structures with many connections. In some cases, it may be more cost effective to adopt other strategies, intended to reduce demands on connections than to increase individual connection capacities.

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

5.4.2 Lessening or Removal or Irregularities

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

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

5.4.3 Global Structural Stiffening

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

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

FEMA-273 provides modeling guidance and acceptance criteria for bracing and shear wall elements used to structurally stiffen a WSMF structure. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of FEMA-273. The performance of WSMF elements of the structure shall be than be evaluated using the procedures of Chapter 3, with the mathematical model modified to include the effects of the upgrade elements on structural response.

5.4.4 Global Structural Strengthening

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

FEMA-273 provides modeling guidance and acceptance criteria for bracing and shear wall elements used to structurally stiffen or strengthen a WSMF structure. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of FEMA-273. The performance of WSMF elements of the structure shall be than be evaluated using the procedures of Chapter 3, with the mathematical model modified to include the effects of the upgrade elements on structural response.

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

5.4.5 Mass Reduction

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

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

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

5.4.6 Seismic Isolation

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

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

FEMA-273 provides modeling guidelines and acceptance criteria for isolation systems for use in performance evaluation of isolated structures. Upgrades using this strategy shall be conducted by designing the upgrade elements using the guidelines of FEMA-273. The performance of WSMF elements of the structure shall be than be evaluated using the procedures of Chapter 3, with the mathematical model modified to include the effects of the upgrade elements on structural response.

5.4.7 Supplemental Energy Dissipation

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

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

The energy dissipated by a damping device is the integrated product of the amount of force the device exerts on the structure (or is exerted on the device by the structure) and the distance through which this force acts. In many ways, WSMF structures are ideal candidates for upgrade employing energy dissipation systems because they are inherently flexible structures permitting damper elements to dissipate large amounts of energy at relatively low force levels. This is important because whatever forces the dampers are subjected to, must also be resisted by the structure.

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

5.5 As-Built Conditions

5.5.1 General

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

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

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

5.5.2 Material and Section Properties

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

5.6 Upgrade Components

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

5.6.1 Material Specifications

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

5.6.2 Material Strength Properties

The AISC Seismic Provisions (Ref.) state:

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

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

For normal design purposes the AISC requirements should be followed as a minimum. Where a higher than normal reliability is desired, the designer should consider the variability of the properties and apply appropriate coefficients of variation.

Note: ASTM is about to issue a new, A992 specification for structural steels. This specification is similar to the ASTM A572 specification for Grade 50 steels, except that more restrictive limits apply to the permissible variation in yield strength, the ratio of yield to tensile strength and certain other properties, than contained in ASTM A572. This material specification was specifically developed by the steel industry in response to concerns raised by structural engineers with regard to the large variations in properties inherent in the A572 specification, and the difficulties this presented with regard to design for inelastic behavior and seismic resistance. The A992 material will be the recommended basic grade of steel for use in seismic force resisting systems. As this specification has not yet been officially adopted, it has not been addressed by this draft of the guidelines. However, later drafts will include reference and guidelines for use of this material.

Commentary: The SAC studies of rolled sections of Grade 50 steel indicates that the 1.1 value for R_y is a good representation of the mean value of yield strength. The study also developed statistics on the sectional properties of current rolled shapes. The statistics are given in the table below:

Statistic	F_m/F_y	*Area*	Z_x	Z_y
Mean	*1.09*	*0.990*	*0.987*	*0.984*
COV	*0.080*	*0.018*	*0.019*	*0.025*

In the relationship F_m/F_y, F_m represents the measured dynamic yield strength and F_y is, as usual, the "specified minimum yield stress" , or in this case 50 ksi.

We can see that in the mean, the expected yield strength, F_{ye}, is reasonably assumed to be $1.1R_y$. If a higher level of reliability is desired, values that account for the statistical variance may be used. The yield overstrength is somewhat offset by the fact that in the mean the cross *sectional properties are lower than the nominal. The mean value of the product of the yield strength statistic with the cross sectional properties can be estimated as the product of the means of the two values. The standard deviation of the product can be estimated as the sum of the squares of the standard deviation of each parameter. The estimated means and standard deviations and the mean +/-1 and 2 times the standard deviation are shown in the table below:*

Parameter	*Mean*	*Std. Dev.*	*Mean -1 Std. Dev.*	*Mean+ 1 Std. Dev.*	*Mean -2 Std. Dev.*	*Mean+ 2 Std. Dev.*
Squash Load $P_y = F_y A_{gross}$	*1.040*	*0.082*	*0.958*	*1.122*	*0.876*	*1.204*
Plastic Moment $M_{px} = F_y Z_x$	*1.039*	*0.082*	*0.957*	*1.121*	*0.874*	*1.203*
Plastic Moment $M_{py} = F_y Z_y$	*1.037*	*0.084*	*0.953*	*1.121*	*0.869*	*1.205*

It can be seen from the table that the Ry value of 1.1 for Grade 50 steel will give reasonable conformance with Mean + 1 standard deviation values. A reasonable estimate of the upper bound of the beam strength is 1.2 times the nominal value of the plastic moment. The designer may wish to use this value when seeking a higher than normal level of reliability.

Similar studies for the other grades of steel have not been performed as part of the SAC program. It is recommended that in the absence of specifically tested values for beam steels being used in the project, that the values for Grade 50 be used, unless steels with higher specified minimum yield stresses are being used, in which case, special qualification testing would be required.

5.6.3 Mathematical Modeling

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

6. CONNECTION QUALIFICATION

6.1 Scope

This section provides performance qualification data for various types of connections, together with guidelines for qualification and design of connections for the upgrade of existing WSMF structures. Included herein are guidelines for design of joints and conditions which are generic to most connection upgrade types, and guidelines for specific connection upgrade details of connections intended to be pre-qualified for use in seismic upgrades. Each of the connection pre-qualifications is limited to specific conditions for which they are applicable, including member size ranges and required inter-story drift capacity. Also included in this Chapter are guidelines for qualification of connections and connection upgrades, which have not been pre-qualified or are proposed for use outside the limits of their pre-qualification as set forth herein.

Commentary: The 1988 Uniform Building Code introduced a single pre-qualified moment connection design, representative of prevailing west coast practice at the time. The "qualification" of this connection was based primarily on the research of Popov and Stephen in the early 1970s. (Ref.) and the belief that this connection was capable of providing acceptable strength and ductility for service in all frames that otherwise met the provisions of the building code. The UBC pre-qualified connection was subsequently adopted into the 1992 AISC Seismic Provisions and then into model codes, nation wide. Although the building codes did not formally adopt the pre-qualification of this standard connection until the late 1980s and early 1990s, this connection detail had seen wide-spread use in WSMF construction since the 1970s.

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

Following the Northridge Earthquake, the pre-qualified connection contained in the building code was deleted by means of an emergency code change. In its place, a provision was substituted requiring that the designer demonstrate that whatever connection was used is capable of sustaining the necessary inelastic deformation demands. Qualification of *this capacity was by prototype testing. In the time since, a significant number of connection assemblies have been tested, allowing new pre-qualifications to be developed. Those pre-qualifications that are applicable to the upgrade of existing structures, incorporating the pre-Northridge style connection, appear in this guideline.*

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

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

6.2 Qualification Data for Existing Connections

This section provides guidelines for modeling and assessing the performance of typical moment-resisting and simple connections typically found in existing WSMF buildings.

6.2.1 Welded Unreinforced Fully Restrained Connection

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

Commentary: The data presented in this section is not specifically applicable to forms of this connection that employ weld metals with significant toughness. Some older building, particularly those erected prior to about 1970, may have welds deposited by the SMAW process. Such welds may have significant toughness, on the order of 20 ft-lbs at -

20^{o}F or more. Limited testing of such connections indicates that they may have substantially better inelastic deformation capacity than do connections employing weld material with lower toughness. Refer to Section 6.4 for data on connections with tough weld metal.

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

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

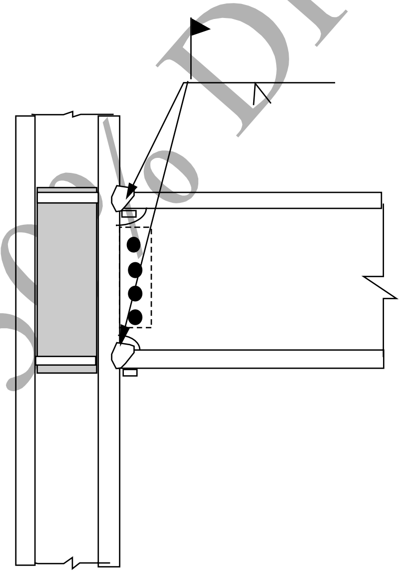


Figure 6-1 Welded Unreinforced Fully Restrained Connection (pre-1994)

6.2.1.1 Modeling Guidelines - Linear Analysis

Framing connected with welded unreinforced fully restrained moment-resisting 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 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.

6.2.1.2 Modeling Guidelines - Nonlinear Analysis

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}, determined in accordance with Chapter 2, 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.

6.2.1.3 Performance Qualification Data

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

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

Pre-qualified Drift Angle Capacity	0.020 radian - collapse prevention 0.010 radian - incipient damage
Capacity Reduction Factor ϕ	0.6 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	dependent on relative strength of panel zone, beam and column
Maximum beam size	unlimited
Beam Material	A36, A572, Gr. 50
Maximum column size	unlimited
Column Steel Grades	A36, A572, Gr. 50

6.2.2 Simple Shear Tab Connections - with slabs

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

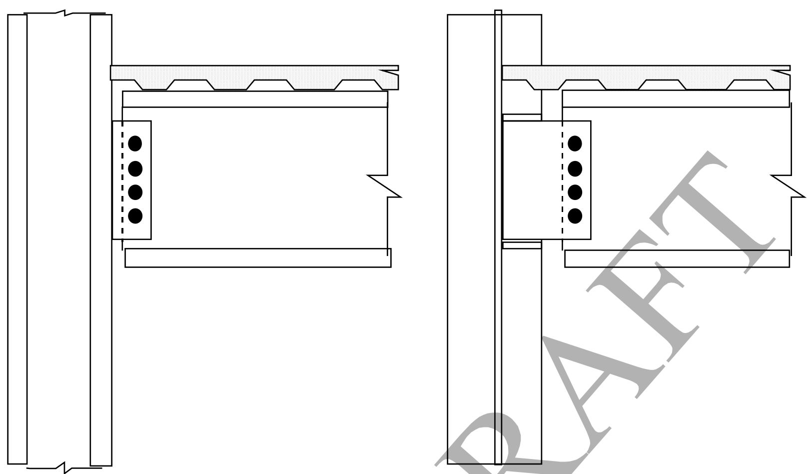


Figure 6-2 Typical Simple Shear Tab Connection with Slab

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

6.2.2.1 Modeling Guidelines - Linear Analysis

For purposes of linear analysis, simple framing with shear tab connections need not be included in the analytical model. If included, either of the following two approaches may be followed:

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.

6.2.2.2 Modeling Guidelines - Nonlinear Analysis

Framing connected with shear tabs, in structures with slabs present, shall be including in the analytical model. Framing should be modeled using center line to center line dimensions. Figure 6-3 presents a general hysteretic model that may be used for analysis of framing with these connections.

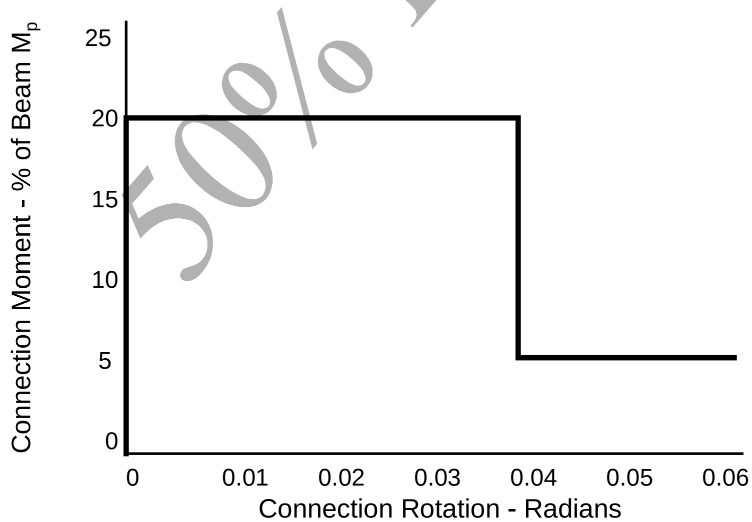


Figure 6-3 General Hysteretic Model for Shear Tab Connections with Slabs

6.2.2.3 Performance Qualification Data

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

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

Pre-qualified Drift Angle Capacity	0.15 radian - collapse prevention 0.02 radian - incipient damage
Capacity Reduction Factor ϕ	0.9 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	at center line of column
Maximum beam size	unlimited
Beam Material	A36, A572, Gr. 50
Maximum column size	unlimited
Column Steel Grades	A36, A572, Gr. 50

6.2.3 Simple Shear Tab Connections - without slabs

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

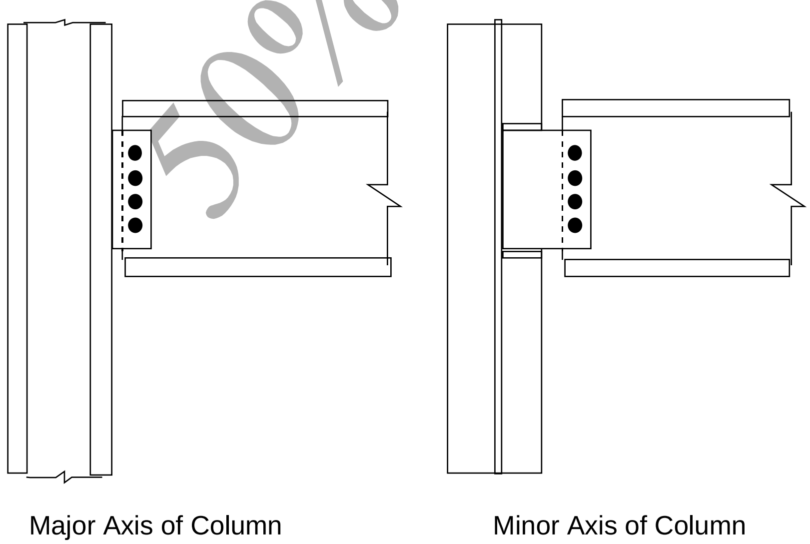


Figure 6-4 Typical Simple Shear Tab Connection without Slab

Commentary: Shear tabs of the type shown in Figure 6-4, 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.

6.2.3.1 Modeling Guidelines - Linear Analysis

For purposes of linear analysis, simple framing with shear tab connections and no slabs present need not be included in the analytical model. If included, 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.

6.2.3.2 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 inertial.

6.2.3.3 Performance Qualification Data

Table 6-3 presents the applicable performance qualification data for shear tab connections of beams to columns, with slabs present.

Table 6-3 Performance Qualification Data - Shear Tab Connections (no slab)

Pre-qualified Drift Angle Capacity	0.15 radian - collapse prevention 0.04 radian - incipient damage
Capacity Reduction Factor ϕ	0.9 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	at center line of column
Maximum beam size	unlimited

Beam Material	A36, A572, Gr. 50
Maximum column size	unlimited
Column Steel Grades	A36, A572, Gr. 50

6.3 Basic Design Approach for Connection Upgrades

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

6.3.1 Frame Configuration

Upgraded frames should be proportioned and detailed so that the required plastic deformation of the frame may be accommodated through the development of plastic hinges at pre-determined locations within the frame. Figure 6-5 indicates a frame in which plastic deformation is accommodated through the development of plastic flexural deformation (plastic hinges) within the beam span, remote from the face of the column, which is generally, the most desirable form of plastic frame behavior. Such behavior may be obtained by locally stiffening and strengthening type FR connections, using cover plates, haunches and similar detailing, such that the ratio of flexural demand to plastic section capacity is maximum at these interior span locations. This condition can also be obtained by locally reducing the section of the beam, at desired locations for plastic hinging to obtain a condition of maximum flexural demand to plastic section capacity at these sections. Other locations at which plastic deformation may take place in frames, depending on the configuration, detailing, and relative strength of the beams, columns, and connections include: within the connection assembly itself, as is common for type shear tab type framing connections; within the column panel zone, or within the column.

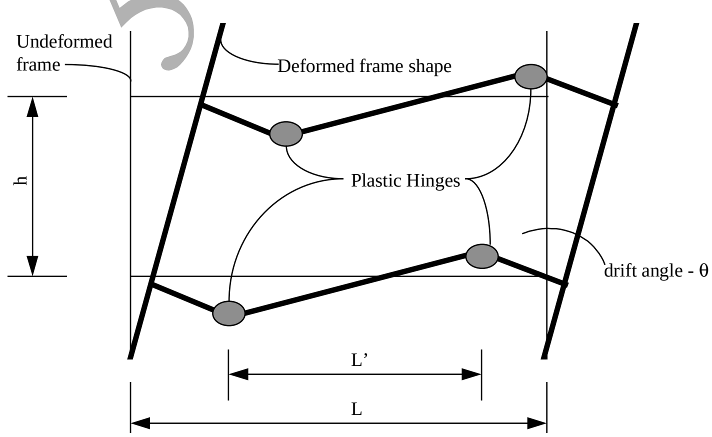


Figure 6-4 - Preferred Inelastic Behavior of Frames with Hinges in Beam Span

Commentary: Nonlinear deformation of frame structures is accommodated through the development of inelastic flexural or shear strains within discrete regions of the structure. At large inelastic strains these regions can develop into plastic hinges, which can accommodate significant concentrated rotations at constant (or nearly constant) load through yielding at tensile fibers and buckling at compressive fibers. If a sufficient number of plastic hinges develop in a frame, a mechanism is formed and the frame can deform laterally in a plastic manner. This behavior is accompanied by significant energy dissipation, particularly if a number of members are involved in the plastic behavior, as well as substantial local damage to the highly strained elements. The formation of hinges in columns, as opposed to beams, is undesirable, as this results in the formation of mechanisms with relatively few elements participating, so called "story mechanisms" and consequently little energy dissipation occurring. In addition, such mechanisms also result in local damage to the columns that are critical gravity load bearing elements.

The pre-qualified connection contained in the building codes prior to the Northridge Earthquake was based on the development of plastic hinges within the beams at the face of the column, or within the column panel zone itself. If the plastic hinge develops in the column panel zone, the resulting column deformation results in very large secondary stresses on the beam flange to column flange joint, a condition that can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in very large through-thickness strain demands on the column flange material and large inelastic strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle joint failure unless particular care is taken in fabricating the connection.

WSMF structures are expected to be capable of extensive amounts of energy dissipation through the development of plastic hinges. In order to achieve reliable performance of these structures, frame configurations should avoid a strong beam-weak column design to avoid the development of column hinging and story collapse mechanisms and beam-column connections should be configured to force the inelastic action (plastic hinge) away from the column face, where its performance is less dependent on the workmanship of the welded joint. This can be done either by local reinforcement of the connection, or local reduction of the cross section of the beam, at a distance away from the connection. Plastic hinges in steel beams have finite length, typically on the order of half the beam depth. Therefore, the location for the plastic hinge should be shifted at least that distance away from the face of the column. When this is done through reinforcement of the connection, the flexural demands on the *columns, for a given beam size, are increased. Care must be taken to assure that weak column conditions are not inadvertently created by local strengthening of the connections.*

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

It should also be noted that reinforced connection (or reduced beam section) configurations of the type described above, while believed to be effective in preventing brittle connection fractures, will not prevent structural damage from occurring. Brittle connection fractures are undesirable because they result in a substantial reduction in the lateral-force-resisting strength of the structure which, in extreme cases, can result in instability and collapse. Connections configured so as to force plastic hinging into the beam span should experience many fewer such brittle fractures than unmodified connections. However, the formation of a plastic hinge within the span of a beam is not a completely benign event. Beams which have formed such hinges may exhibit large buckling and yielding deformation, damage which typically must be repaired. The cost of such repairs could be comparable to the costs incurred in repairing fracture damage experienced in the Northridge Earthquake. The primary difference is that life safety protection will be significantly enhanced and most structures that have experienced such plastic deformation damage should continue to be safe for occupancy, while repairs are made.

If the types of damage described above are unacceptable for a given building, then alternative structural systems should be considered, which will reduce the plastic deformation demands on the structure during a strong earthquake. Appropriate methods of achieving such goals include the installation of supplemental braced frames, energy dissipation systems, base isolation systems, and similar structural systems.

6.3.2 Inter-story Drift Capacity

For systematic upgrade design, the factored inter-story drift capacity of connection assemblies should be sufficient to withstand the total (elastic and plastic) drift likely to be induced in the frame by earthquake ground shaking, as predicted by analysis, while providing sufficient confidence with regard to achievement of the desired performance, in accordance with the guidelines of Chapter 3. Sections 6.4 and 6.5 provide guidelines for determining the factored inter-story drift capacity of pre-qualified and qualified-by-test connection upgrades, respectively.

For the purposes of Simplified Rehabilitation, frames shall be classified either as Ordinary Moment Frames (OMFs) or Special Moment Frames (SMFs) and connection upgrade details that are pre-qualified for the appropriate system, as indicated in Sections 6.4 of these guidelines should be selected. For purposes of simplified upgrades, a frame should be considered an SMF system if the construction documents indicate it was designed as a Special Moment Resisting Frame, or a Ductile Moment Resisting Frame; or, if the original design documents indicate that any of the values indicated in Table 6-4 were used in determining the design seismic forces for the frame in the original design. A frame should be considered an OMF if the design documents indicate it was designed as an OMF or if any of the values indicated in Table 6-4 were used in determining the design seismic forces for the frame in the original design. If sufficient documentation is not available to permit determination of the original intended system for the structure, an SMF should be assumed.

Table 6-4 Design Coefficients for SMF and OMF Systems

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

Commentary: In Systematic upgrades, a complete analysis of the structure is performed, in accordance with the guidelines of Chapter 3. In this analysis, an estimate is developed of the forces and deformations induced by response to earthquake ground shaking, and based on these estimated forces and deformations, and the estimated capacity of the frame and its individual components to resist these demands, a level of confidence with regard to the ability of the frame to provide desired performance is estimated. Later sections of this chapter provide acceptance criteria (factored connection inter-story drift capacities) for various connection upgrade details that are used in the evaluations conducted in accordance with Chapter 3, in order to determine a level of confidence of performance capability.

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

Table 6-4 classifies framing systems using the terminology contained in the 1997 NEHRP Provisions and 1997 AISC Seismic Design Specification, as either an SMF or an OMF. A third system, termed Intermediate Moment Resisting Frame (IMF) was introduced for the first time in these design specifications and codes based upon them. Since frames designed in accordance with these later documents are unlikely to require upgrade, the Intermediate System is not referenced with regard to Simplified Upgrade.

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

6.3.3 Connection Configuration

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

a) Weak column - strong beam

b) Weak story

c) Soft story

d) Torsional Irregularity

For Systematic Rehabilitation a connection configuration that is capable of providing sufficient factored inter-story drift capacity should be selected . The connection upgrade configuration should also be compatible with the sizes of the framing elements. Section 6.4 presents data on a series of pre-qualified connection upgrade details, from which an appropriate detail may be selected. Alternatively, if project-specific connection qualification is to be performed, a connection of any configuration may be selected and qualified for acceptability using the procedures of Section 6.5.

6.3.4 Determine Plastic Hinge Locations

Based on the data presented in these Guidelines for pre-qualified connections, or data obtained from a qualification testing program for qualified-by-test connection configurations the location of expected plastic hinge formation, s_h,, as indicated in Figure 6-5 should be identified. The plastic hinge locations presented for pre-qualified connections are valid for beams with gravity loads representing a small portion of the total flexural demand and for conditions of strong column - weak beam. For frames in which gravity loading produces significant flexural stresses in the members, or frames that do not have strong column - weak beam configurations, locations of plastic hinge formation should be determined based on methods of plastic analysis.

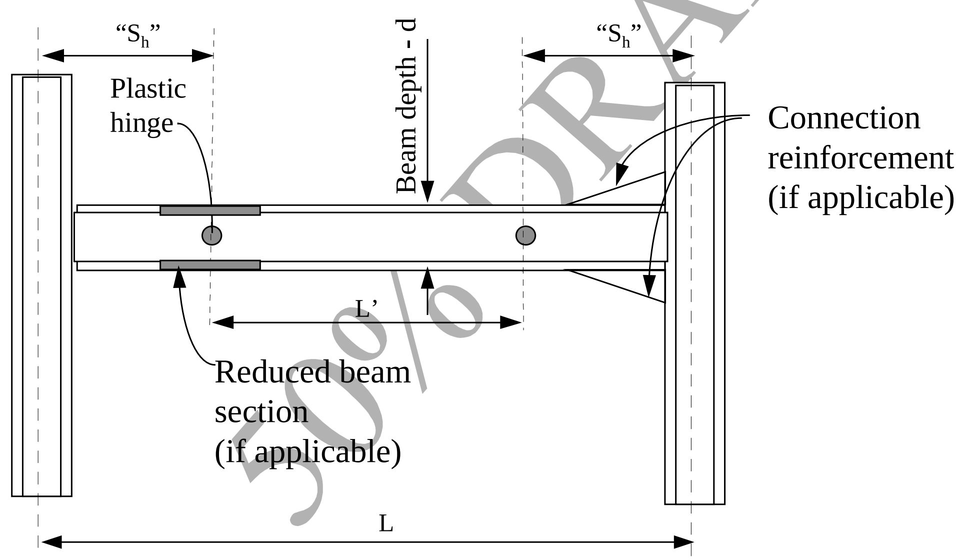


Figure 6-5 - Location of Plastic Hinge Formation

Commentary: The suggested location for the plastic hinge, as indicated by the parameter s_h in the pre-qualification data, is valid only for frames with limited gravity loading present on the frame beams, or for frames in which yielding will actually occur in the beam, rather than in the column panel zone or the column itself. If significant gravity load is present, or if panel zones or columns are the weak links in the frame, this can shift the locations of the plastic hinges, and in the extreme case, change the form of the collapse mechanism. If flexural demand on the girder due to gravity *load is less than about 30% of the girder plastic capacity, this effect can safely be neglected, and the plastic hinge locations taken as indicated, as long as beam flexure, rather than panel zone shear or column flexure is the dominant inelastic behavior for the frame. If gravity demands significantly exceed this level then plastic analysis of the girder should be performed to determine the appropriate hinge locations. In zones of high seismicity (S_{D1} greater than 0.2g) gravity loading on the girders of earthquake resisting frames typically has a very small effect.*

If frame yielding under lateral deformation is controlled by shear yielding of column panel zones, upgrades should include reinforcement of the panel zones to preclude this behavior mode. If frame yielding under lateral deformation is controlled by flexure of the columns, than either the columns should be reinforced to preclude this behavior mode or alternatively, supplemental lateral force resistance comprised of braced frames or shear walls should be provided.

6.3.5 Determine Probable Plastic Moment at Hinges

The probable plastic moment at the location of the plastic hinge should be determined as:

$$M_{pr}=1.1R_yZ_eF_y \qquad (6\text{-}1)$$

where:

M_{pr} = Probable plastic hinge moment, considering material strength variation, and strain hardening effects

R_y = A coefficient obtained from Table 2-1

Z_e = The effective plastic modulus of the section (or connection) at the location of the plastic hinge

F_y = the specified minimum yield strength of the material of the yielding element

6.3.6 Determine Shear at the Plastic Hinge

The shear at the plastic hinge should be determined by statics, considering gravity loads acting on the beam. A free body diagram of that portion of the beam between plastic hinges, is a useful tool for obtaining the shear at each plastic hinge. Figure 6-6 provides an example of such a calculation. For the purposes of such calculations, gravity load should be based on the load combinations required by the building code in use.

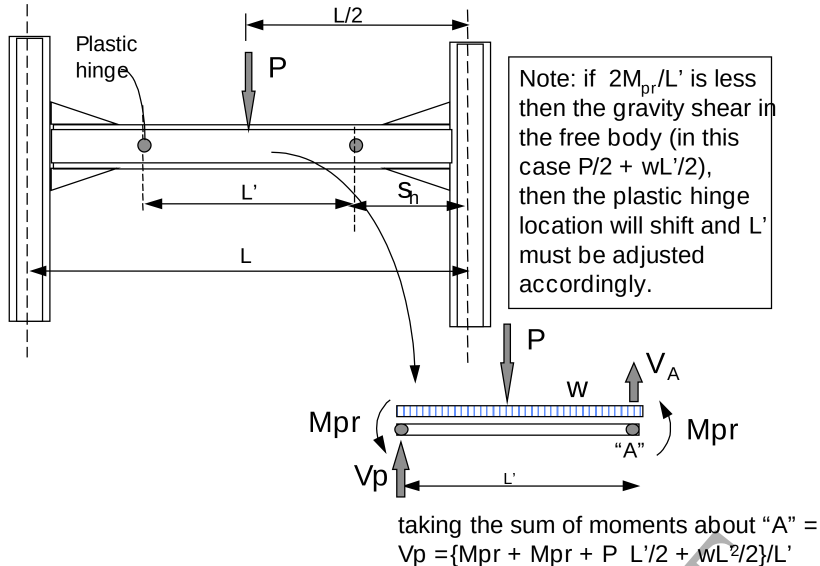
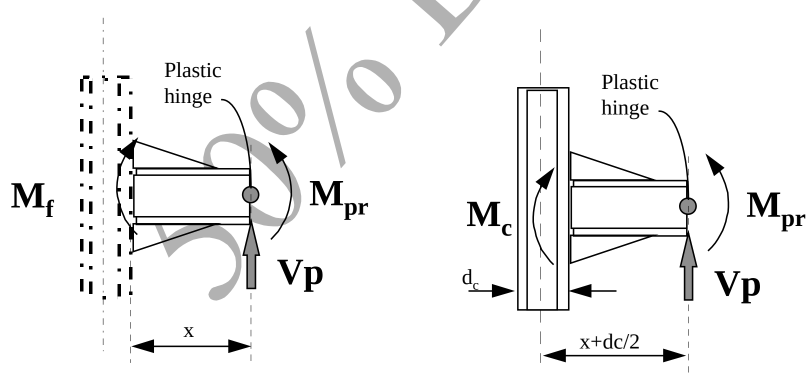


Figure 6-6 - Sample Calculation of Shear at Plastic Hinge

6.3.7 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection upgrade, including sizing the various plates, bolts, joining welds, etc. which make up the connection, it is necessary to determine the shear and flexural strength demands at each critical section. These demands may be calculated by taking a free body of that portion of the connection assembly located between the critical section and the plastic hinge. Figure 6-7 demonstrates this procedure for two critical sections for the beam shown in Figure 6-6.



$M_f = M_{pr} + V_p x$ $M_c = M_{pr} + V_p(x + d_c/2)$

Critical Section at Column Face Critical Section at Column Centerline

Figure 6-7 - Calculation of Demands at Critical Sections

Commentary: Each unique connection configuration may have different critical sections. The vertical plane that passes through the joint between the beam flanges and column (if such joining occurs) will typically define *at least one such critical section, used for designing the joint of the beam flanges to the column, as well as evaluating shear demands on the column panel zone. A second critical section occurs at the center line of the column. Moments calculated at this point are used to check strong column-weak beam conditions. Other critical sections should be selected as appropriate.*

6.3.8 General Requirements

This section provides guidelines for connection upgrade design conditions which are considered to be general, that is, those conditions which, when they occur in a connection upgrade, are considered to perform in a similar way, or at least to have the same requirements for successful performance, irrespective of the connection type being used. The designer should employ these guidelines in the design of all connection types, except when specific testing has been performed which qualifies the connection for use with different conditions, or unless otherwise specifically indicated in these guidelines.

6.3.8.1 Column Flange Through Thickness Strength

The through-thickness strength demands on existing column material should be limited to the values given in Table 6-5. Through thickness demands should be calculated using the procedures of Section 6.3.7.

Table 6-5 Column Flange Through-Thickness Strength

Column Flange Material Specification	F_{t-t}
Hot rolled wide flange columns conforming to ASTM A-572 Grade 50, or ASTM A-992, or ASTM A913 rolled later than 1994 and having sulfur content not in excess of 0.05% by weight.	No limit
All other material	$0.8F_u$

Commentary: Early investigations of connection fractures in the Northridge earthquake identified a number of fractures (types C3 and C5) that appeared to be the result of inadequate through-thickness strength of the column flange material. As a result of this, in the period immediately following the Northridge Earthquake a number of recommendations were promulgated that suggested limiting the value of through thickness stress demand on column flanges to a value of 40 ksi, applied to the projected area of the beam flange attachment. This value was selected to ensure that through-thickness yielding did not initiate in the column flanges of FR connections and often controlled the overall design of a connection subassembly.

It is important to prevent the inelastic behavior of connections from being controlled by through-thickness yielding of column flanges. This is because it would be necessary to develop very large local ductilities in the column flange material in order to accommodate even modest plastic rotation demands on the assembly. However, the actual cause for the type C3 fractures, that were initially identified as through-thickness failures of the column flange are now believed to be unrelated to this material property. Rather, it appears that C3 damage occurred when fractures initiated in defects present in the CJP weld root, not in the flange material (Ref). These defects sometimes initiated a crack that under certain conditions propagated into the column flange, giving the appearance of a through-thickness failure. Detailed fracture mechanics investigations by Dierelein suggest that the C3 damage will occur not as a result of inadequate through thickness strength, but rather as a result of certain combinations of material strength and toughness and conditions of stress in the connection.

As part of the SAC phase II program of research, extensive through thickness testing of modern steels, meeting the ASTM A-572, Gr. 50 and ASTM A913, Gr 65 specifications has been conducted to determine the susceptibility of modern column materials to through thickness failures. This combined analytical and laboratory research clearly showed that due to the restraint inherent in welded beam flange to column flange joints, the through thickness yield and ultimate strengths of the column material is significantly elevated in the region of the connection. Further, for the modern materials tested, these strengths significantly exceed those that can be delivered to the column by beam material conforming to these same specifications. For this reason, no limits are suggested in Table 6-5 for the through thickness strength of modern steel materials with controlled sulfur contents.

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

*Table 6-5 recommends a limit of 0.8*F_u *for through thickness stress on older steels, that may be susceptible to through thickness tearing, based on a statistical survey reported by Barsom (ref.).*

6.3.8.2 Base Material Notch-Toughness

When used as members in new WSMF construction, ASTM A6 Group 3 shapes with flanges 2-inch-thick and thicker, ASTM A6 Groups 4 and 5 shapes, and plates that are 1-½-inch-thick or thicker in built-up cross-sections should have a minimum Charpy V-Notch (CVN) toughness of 20 ft-lbs. at 70 degrees F. No limits are specified for existing material.

Commentary: The AISC LRFD Specification requirements (Ref) for notch toughness cover Groups 4 and 5 shapes and plate elements with thickness that is greater than or equal to 2-in. in tension applications. In these Guidelines, this recommendation is extended to cover: (1) all Group 4 and 5 shapes that are part of the WSMF; (2) ASTM Group 3 shapes that are part of the WSMF with flange thickness greater than or equal to 1-½-inch; and (3) plate elements with thickness greater than or equal to 2-in. that are part of the WSMF, such as the flanges of built-up girders. Because other shapes and plates are generally subjected to enough cross-sectional reduction during the rolling process that the resulting notch toughness will exceed that required above, specific recommendations have not been included herein.

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

6.3.8.3 Weld Filler Metal Notch-Toughness

All complete-joint-penetration groove welds made in the upgrade of WSMF connections shall be made with a filler metal that has a minimum CVN toughness of 20 ft-lbs. at minus 20 degrees F, as determined by AWS classification or manufacturer certification. This requirement for notch toughness shall also apply in other cases as recommended in these *Guidelines*. Existing welds need not be replaced with weld metal having this toughness, unless specifically indicated in these guidelines.

6.3.8.4 Weld Backing and Weld Tabs

6.3.8.4.1 Weld Tabs Existing weld tabs on existing joints need not be removed, unless specifically indicated otherwise in these guidelines.

Weld tabs used in new welded joints may be of any of the steels approved by the applicable building code for use in building structures. Weld shall be terminated at the end of a joint in a manner that will ensure sound welds. Whenever necessary, this shall be done by use of weld tabs aligned in such a manner as to provide an extension of the joint preparation. Weld tabs shall be removed upon completion and cooling of the weld, and the ends of the weld shall be made smooth and flush with the edges of abutting parts. The weld tabs may be removed by air arc or oxygen-acetylene burning (cutting) 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.

If weld tabs were used and are to be removed in conjunction with the removal of the weld backing, the tabs should be removed at the same time that the weld backing is removed and the fillet added.

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.

6.3.8.4.2 Weld Backing Existing weld backing on joints need not be removed unless specifically indicated otherwise, in these guidelines.

Weld backing for new welded joints may be of any of the steels approved by the applicable building code for use in building structures. Other materials may be used as weld backing as approved in the AWS D1.1 (Ref). Weld backing shall be used when the root opening cannot be closed to meet the requirements of the applicable AWS tolerance or the if required by the pre-qualified joint geometry.

Groove welds made with the use of steel backing shall have the weld metal thoroughly fused with the backing. Steel backing shall be made continuous for the full length of the weld. All joints in the steel backing shall be complete joint penetration welded butt joints meeting all of the requirements of the AWS code.

Steel backing, 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. The weld should be completed and reinforced with a fillet weld. Removal of the weld backing at the top girder flange is not required, but may be done at the discretion of the 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 D 1.1. 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 pre-qualified joint design of D 1.1. If for any reason the WPS does not meet the pre-qualified limits of AWS D1.1 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) that will be used to remove the weld backing, back gouge to sound metal and when during this process he will apply preheat.

Commentary: Although project conditions may vary, the following general guidelines may be considered. Steel backing may be removed by either grinding or by the use of air arc or oxygen acetylene burning (cutting). The zone just beyond the theoretical 90-degree intersection of the beam-to-column flange should be removed by either air arc or oxygen acetylene cutting followed by a thin grinding disk, or by a grinding disk alone. This shallow gouged depth of weld and base metal should then be tested by 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.

Removal of the weld backing from the top flange may be difficult, particularly along perimeter frames where access to the outer side is restricted. Since the potential stress riser produced by the unwelded portions of the weld backing are not located on the extreme outer fiber of the frame girder, the benefits of removal may be limited in repair *situations. 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.*

6.3.8.5 Reinforcing Fillet Welds

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

6.3.8.6 Weld Access Hole Size, Shape, Workmanship

There is little question that the size, shape, and workmanship of weld-access holes is a critical issue for performance of welded connections, particularly for those connections which do not utilize reinforcements of the flanges. Connection designs should utilize weld-access hole configurations and construction techniques that match those of the tested connections as indicated in the connection pre-qualification or as employed in the project-specific qualification testing.

Radii of access holes shall provide a smooth transition free of notches or cutting past the point of tangency between adjacent surfaces. The cut surface shall satisfy the surface requirements of AWS D1.1 Section 5.15.4.3.

Commentary: The size, shape, and workmanship of weld-access holes can affect the connection strength in several different ways, including the following:

- *Ease of making the weld and performing the NDT for bottom flange welds (and therefore their quality) is affected by the hole size and shape (bigger is better);*

- *The size and shape of the hole affects the stress distribution in both the flange and the web in the area of the hole (smaller may be better);*

- *The shape and workmanship of the hole affects the stress concentrations in flanges and the web in the area of the hole (smooth, semi-circular holes are better).*

Based on their finite analysis results El Tawil et al (Ref.) make the following statement: "Increasing the size of the web cope would permit *easier welding on the beam bottom flange, and possibly promote better weld quality. However, the analyses in this section suggest that it is important to use a small access hole in order to minimize the potential for ductile fracture at the root of the hole. The analyses further show that the access hole in which the web terminates perpendicular to the flange is clearly inferior to the semi-circular detail from the ductile fracture point of view."*

Since these results present a design dichotomy, some have suggested using large holes for purposes of welding and testing and then reinforcing the openings with cover plates. Others, including the Japanese, have suggested using small access holes and welding the holes up after the flange welding is completed. Murray (Ref.) prefers not to use access holes for welding of end plates to beams for end-plate connections, although this is a somewhat different case because the welds are made in the shop and can be made from the outside of both flanges.

For situations where extra large access holes are used, the designer is cautioned to consider the effect of disconnection of the flange from the web on the shear capacity of the resulting unstiffened web, where large shears must be transmitted to the column flange.

6.3.8.7 Continuity Plates

Continuity plates should be provided for all connection upgrades in which beam flanges are welded directly to the column flanges. For one-sided connections continuity plate thickness should be at least one half of the thickness of the beam flanges. For two-sided connections the continuity plates should be equal in thickness to the thicker of the beam flanges on the two sides. Continuity plates need not be provided when project-specific qualification testing indicates that such plates are not required.

Continuity Plates should be welded to column flanges using the same recommendations as for beam flanges, e.g. CJP welds should incorporate weld metal with rated toughness and backing and runoff tabs should be removed. Continuity plates should be welded to the web as required to transmit the shear forces corresponding to development of the axial strength of the CJP weld at one end of the connection, for one-sided connections, and that at both ends, for two-sided connections.

Commentary: FEMA267 says that continuity plates should be provided which match the thickness of the beam flanges. Several studies (Ref. Allen and Richard, Roeder) have shown that the absence of continuity plates significantly affects the stress distribution across the beam flange at the beam-to -column flange joint. Without continuity plates, the stresses opposite the column web may be multiple times larger than those at the flange tips, depending on the thickness of the column flange. Tremblay et *al (1995) reported that connections with continuity plates were found to have fewer connection failures.*

Studies by El-Tawil et al (Ref.) showed that the stress distribution was relatively insensitive to the thickness of the continuity plates in one sided connections. Analyses with continuity plates having thicknesses of approximately 60% of the thickness of the beam flanges resulted in almost no change in the stress and strain conditions at the connection as compared to the full thickness plates for the beam and column sizes studied [W36x150 (A36) Beam, W 14x257 (Grade 50) Column]. Further research is required to determine if this effect applies to conditions with thinner column flanges, higher strength beams, or two-sided connections. At this writing, it is assumed that thicker plates will be required for two-sided connections.

The CJP welds of continuity plates are expected to be subjected to conditions of stress similar to those of the CJP welds of the beam flange. For this reason, the same conditions should apply to their construction.

6.3.9 Bolted Joints

6.3.9.1 Existing Conditions

When evaluating existing structures, the condition of bolted or riveted connections shall be determined based on the appropriate AISC and RCSC Specifications and the following criteria:

- Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine tensile strength in accordance with ASTM F606 and the bolt classified accordingly. Alternatively, the assumption that the bolts are A307 shall be permitted. Rivets shall be assumed to be A502, Grade 1, unless a higher grade is established through documentation or testing.

- The edges of connection plates around bolted connections should be visually examined, and if necessary, inspected using NDT procedures such as magnetic particle (MT) to determine if any crack initiation occurred. Repairs to connection plates, if required, should be made using approved welding procedures as outlined in Section 8.3.

- Any evidence of yielding in the connection plates indicates that the high strength bolts are effectively in the snug tight condition regardless of their original installation condition. If bolts have been identified as ASTM A325 and are not in a snug tight condition they should be re-tightened or replaced.

If bolts have been identified as ASTM A490 and are not in a snug-tight condition, they should be replaced. Re-tightening or installation of bolts should be to a pre-tensioned condition in accordance with AISC or RCSC criteria (Ref. AISC & RCSC Spec).

6.3.10 Upgrading Connections

When upgrading existing connections, the capacity shall be determined based on the appropriate AISC and RCSC Specifications and the following criteria:

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

6.4 Pre-qualified Connection Upgrades

This section provides pre-qualification data for various alternative types of WSMF connection upgrades. Depending on the factored inter-story drift capacity required for the building, as indicated by an analysis in the Systematic approach, or in the case of Simplified Rehabilitation, by framing system type; and the member sizes, the designer may select a suitable connection upgrade detail from the following table.

Table 6-6 - Pre-qualified Welded FR Connection Upgrade Details

Connection Type	Criteria Section	Frame Type	Incipient Damage		Collapse Prevention	
			Limit Drift Angle (radians)	Capacity Reduction Factor Φ_I	Limit Drift Angle (radians)	Capacity Reduction Factor Φ_C
IWURF	6.4.1	OMF	0.01	.9	.02	0.6
WBH	6.4.2	SMF	0.01	.9	0.04	0.7
WTBH	6.4.3					
BBH	6.4.4					
BBTH	6.4.5					
BRBS	6.4.6					

6.4.1 Improved Welded Unreinforced Flange (IWURF)

This section provides guidelines for design of connection upgrades intended to improve existing unreinforced, welded flange connections without either strengthening the connection or locally reducing the beam section. Upgrade is accomplished through replacement of existing complete joint penetration groove welds of low toughness material and potentially having significant root defects, with new welds conforming to current construction requirements for WSMF construction. In addition, other elements of the connection are reinforced, as required. Table 6-7 provides performance qualification data for the reinforced connections.

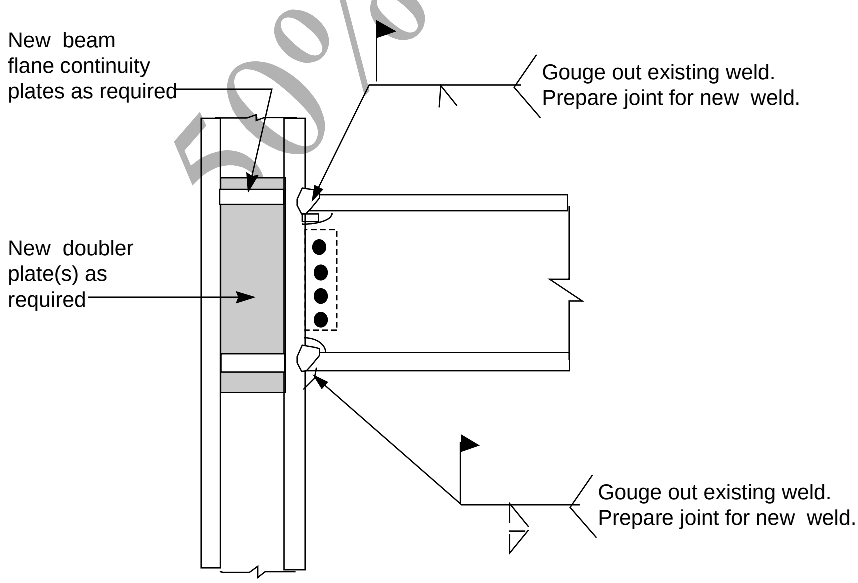


Figure 6-8 - Typical Detail - IWUF Connection

Table 6-7 - Pre-qualification Data WUF Connections

Applicable systems	OMF
Pre-qualified Drift Angle Capacity	0.02 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor ϕ	0.6 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	$d_c/2$
Maximum beam size	W36 x 150
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913

Commentary: There is more research information available on unreinforced beam-to-column connections than there is on any other type of steel moment frame connection. Not only were these connections studied prior to Northridge (Ref. Popov and Stephen, Popov and Amin, Englehardt and Husain, 1992 etc.), but they have been even more extensively studied in the aftermath. Many of the studies focused on the connection as used in pre-1994 practice, that, with bolted web connection and E70T-4 flange welds, with backing left in place (Refs..), while others have been focused on improvements to the connection, including those improvements recommended in this section.

Lu, Xue, Kaufmann and Fisher conducted a number of different tests at Lehigh, which were focused on fracture mechanics and the effects of notch toughness of welding electrodes. In one series of tests, four full scale specimens using W36x150 beams (A36) and W14x311 columns (Grade 50) were tested dynamically. Specimen A-1 was fabricated with E70T-4 electrodes with backing bars left in place, a bolted web connection, and no continuity plates. This connection was similar to some which fractured in the Northridge Earthquake. This specimen fractured at the bottom flange connection at 87% of the yield moment of the beam. Specimen A-2 was similar to A-1 except that backing was removed and small fillet welds were added to the back side of the welds. This connection showed a slightly improved performance, but still fractured at only 92% of the beam yield moment. Specimens A-3 and A-4 were similar to each other in that continuity plates were added, beam webs were attached to the column flanges with complete penetration welds, welding was performed with notch-tough electrodes, backing was removed and fillet welds were added to the back side of the flange welds. The difference between the two was that A-3 was welded with E7018 and A-4 was welded with E70TG-K2 flux cored electrode, both of which have good *notch toughness ratings. Specimens A-3 and A-4 achieved inelastic rotations in the range of .025 Radian. Based on these, and other similar tests they have conducted, the authors conclude that notch toughness of weld wire is a major factor in the performance of this type of connection and that the tests demonstrate "the need to impose fracture toughness requirements on weld metals for future construction in order to insure that premature weld fracture will not occur." (References..)*

Studies conducted as part of the FEMA/ SAC Phase II at the University of Michigan (Goel and Stojadinovic) are further examining these connections to provide better understanding of the following:

1. *Ductility provided by panel zone yielding;*
2. *Depth effects;*
3. *Range where FR connections with bolted webs and flanges welded with notch tough electrode can be used in the future;*
4. *Impact of changes in material properties of steel on connection ductility;*
5. *What went wrong with the pre-Northridge connection.*

In this series of projects, a number of specimens were constructed, using weld metal with rated toughness and W30x99 beams. While all of these specimens exhibited greater ductility than typical connections fabricated with low toughness weld metals, none were able to achieve the amount of ductility obtained in the Lehigh tests. All of the specimens developed brittle fracturing, extending across one of the beam flanges, approximately in line with the toe of the weld access hole. Finite element studies confirm that the beam flange at the toe of the weld access hole is subjected to very large stresses and that the severity of these stresses is dependent on a number of factors including the shape of the access hole itself, the depth of the beam section, the ratio of web section properties to total section properties and the relative strength and flexibility of the column panel zone. In the testing conducted at the University of Michigan, none of the specimens achieved drift angles in excess of 0.025 radians.

6.4.1.1 Column Panel Zones

Column panel zones shall provide adequate shear capacity to develop $\Sigma 0.8M_p$ for the beams framing to the column at the connection, in accordance with the AISC Seismic Specification. Where the panel zone does not have this capacity, it shall be reinforced with doubler plates.

6.4.2 Welded Bottom Haunch Connection (WBH)

This connection upgrade is accomplished by converting the existing WURF connection into a haunched connection, with a single haunch present at the bottom beam flange. In addition to welding the new haunch, at the bottom beam flange, if the weld of the top beam flange to the column is made with low toughness weld metal, this weld must be gouged out and replaced with new material having minimum rated toughness of 20 ft-lbs at -20°F. Figure 6-9 provides a typical detail for this connection. Table 6-8 presents performance qualification data for the connection. Refer to NIST-XXXX for the applicable design procedure.

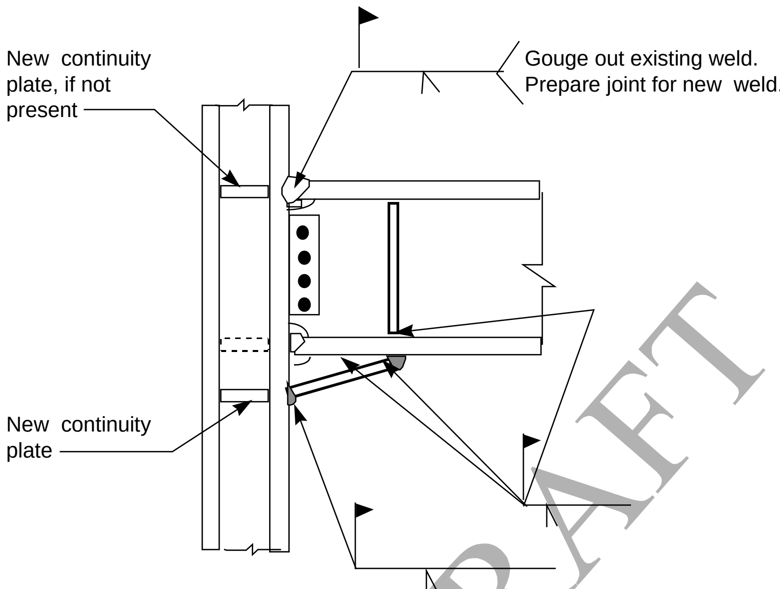


Figure 6-9 Typical WBH Connection Upgrade Detail

Table 6-8 Pre-qualification Data WBH Connections

Applicable systems	OMF SMF
Pre-qualified Drift Angle Capacity	0.05 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor ϕ	0.6 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	$d_c/2+d_b$
Maximum beam size	W36 x 150
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913

6.4.3 Welded Top and Bottom Haunch (WTBH)

This connection upgrade is accomplished by attaching a new welded haunch to both the top and bottom flanges of the existing beam connection. Existing welds in the connection need not be gouged out, or replaced. Design is accomplished to accommodate the general requirements of Section 6.3. Figure 6-10 shows a typical detail for this connection. Table 6-9 provides performance qualification data.

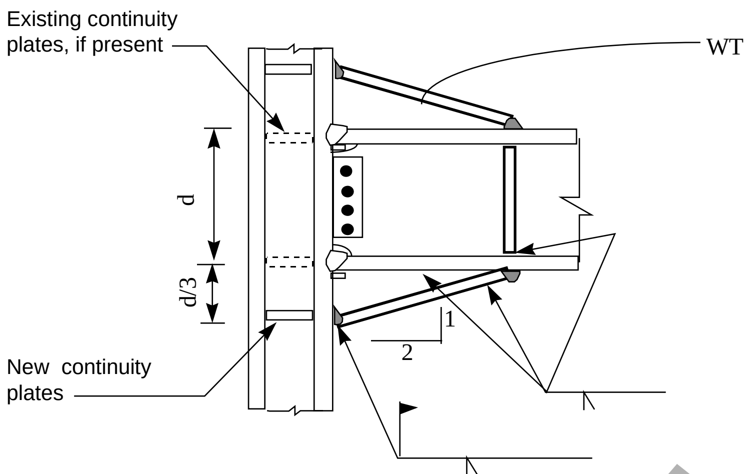


Figure 6-10 Typical Detail WTBH Connection Upgrade

Table 6-9 Pre-qualification Data WTBH Connections

Applicable systems	OMF SMF
Pre-qualified Drift Angle Capacity	0.10 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor ϕ	0.9 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	$d_c/2+d_b$
Maximum beam size	W36 x 150
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913

6.5 Non-pre-qualified Connections

This section provides guidelines for design and project-specific qualification of connection upgrade designs, either for use with performance evaluation or as a basis for upgrade design, for those cases in which there is no current pre-qualification for a connection configuration or for pre-qualified connections which are to be utilized outside the parametric limitations for the pre-qualification as indicated in the guidelines above. Project-specific qualification includes a program of connection assembly proto-type testing supplemented by a suitable analytical procedure that permits prediction of behaviors identified in the testing program.

Commentary: This suggests that for non-pre-qualified connections, both laboratory testing and the development of an analytical procedure that predicts the behavior is required. The intent is to provide a design *procedure applicable to the design of connections employing slightly different members than actually tested. This is similar to the intent of the County of L.A. requirements and more rigorous than contained in the FEMA-267 Guidelines.*

While it is not the intent of the Guidelines to require testing for most situations, there will arise circumstances where proposed connections do not satisfy pre-qualification requirements. In these situations, the requirement for testing reflects the view that the behavior of connections under severe cyclic loading cannot be reliably predicted by analytical means only.

Testing is costly and time consuming, and these Guideline recommendations attempt to keep testing requirements as simple as possible. These tests attempt to account for the behavior of many variables whose behavior is understood imprecisely, and the test conditions should match the conditions in the structure as closely as possible. Where conditions in the structure differ significantly from the conditions implied in this section, additional testing, which is beyond the scope of these Guidelines, may be required.

6.5.1 Testing Procedure

The testing program should replicate as closely as practical the anticipated conditions in the field, including such factors as:

a) Member sizes.

b) Material specifications.

c) Welding process, details and construction conditions.

d) Cover plates, continuity plates, web tabs, bolts, and doubler plates.

e) Connection configuration (e.g., beams on both sides).

f) Induced stresses because of restraint conditions on the welds and connection members.

g) Axial load, where pertinent.

h) Gravity load, where significant.

The testing program should be organized to provide as much information as possible about the capability of the connections selected. The following program is recommended:

a) Test at least two full size specimens representative of the larger beam/column assemblies in the project.

b) Test one additional full size specimen representative of other beam/column assemblies with significantly different interaction properties, such as beam b/t, panel zone stress/distortion, etc.

The minimum acceptable test setup shall consist of not less than a planar arrangement of a single column with one beam attached to the column. Loading shall be applied to the test specimen through the displacement of either the end of the beam or the end of the column. More comprehensive test subassemblies may be required to accurately model actual conditions.

Where two-sided connections are used in the structure, and the type of connection being used can be expected to perform differently in a two-sided use than in one-sided use, it should be tested in the two-sided configuration as well as the one-sided. Two-sided connection assemblies can be expected to behave differently than one-sided assemblies, for example, when panel zone distortions will be significantly different, or when systems involve transfer of stress to the column by plates, welds, or other elements which are connected to the beams on both sides of the column.

The inclusion of axial load should be considered when analysis indicates that significant tension can be expected to occur in a significant number of the columns represented by the specimen and where the connection type relies on the through-thickness strength of the column flanges. If the presence of a floor slab is anticipated to have significant influence on either the location or mechanism of the plastic hinge formed, than this should also be included in the test specimen.

Commentary: Most test specimens have been planar, consisting of a single column with a beam attached to either one or both sides of the column. The specimen is loaded by displacing either the end(s) of the beam or the end of the column. The specimen with a single column and one beam attached to the column is considered the minimum acceptable setup. Other specimen geometries may be necessary to adequately model actual conditions in the structure.

For the purposes of the Guidelines, the test specimens generally need not include a composite slab and need not include the application of axial load to the column. These conditions may have an influence on connection performance, and they can be included as a means of developing more realistic test conditions.

6.5.1.1 Essential Test Variables

The test specimen shall replicate as closely as practical the pertinent design, detailing, construction features, and material properties of the prototype.

(a) Sources of Inelastic Deformation

Behavior of the test subassembly shall reproduce that intended in the prototypical connection. At least 75 percent of the relative magnitude of inelastic deformation developed in the different members of the test subassembly as well as the in the different components of the connection shall reproduce that intended in the prototypical connection.

Commentary: This section is intended to insure that the inelastic rotation of the test specimen is developed in the same members and connection elements anticipated in the prototypical connection. For example, if the prototype connection is designed so that essentially all of the inelastic rotation is developed by yielding of the beam, then the test specimen should be designed to and perform in the same manner.

Variations in material strengths, member lengths, and testing conditions may make it difficult to reproduce precisely the same behavior in the test specimen as expected in the prototype. For this reason, it is believed that the 75 percent requirement provides sufficient latitude to produce reasonably similar results in the test specimens and the prototype. Variations between test specimen and prototype that exceed this limit would not be considered sufficiently similar to qualify under this section.

(b) Size of Members

Beams, columns, and connection components shall be full scale representations of the members used in the structure. The beam used in the test subassembly shall weigh at least 75 percent of those used in the structure. The beam used in the test subassembly shall have a depth of at least 90 percent of those used in the structure. Use of beam sizes outside of these limits will require additional testing.

Commentary: The intent of this section is to insure that test subassemblies match the member size and weight of those used in the actual structure. Members in the actual structure may be scaled up only to the limits specified above; however, it is not intended to limit the scaling down of members to shallower depths or lighter weights than those tested.

In addition to member depth and weight, other considerations exist when sizing members for test subassemblies. One of the most important is the width to thickness ratio of the webs and flanges. Selecting appropriate *width to thickness ratios applies to both the beams and to the columns because of the importance of local buckling and force distribution in controlling the behavior of the connection. It may require that a series of test be undertaken to appropriately bracket the range of beam sizes and width to thickness ratios present in the actual structure.*

(c) Material Strength

The material strengths used in the test subassembly shall match those used in the actual structure. For the purposes of these Guidelines, the beam yield strength in the test subassembly shall not be less than XX percent of that expected in the structure. The column strength shall not be more than XX percent of that expected in the structure.

6.5.1.2 Loading Protocol

Loading may be applied to the specimen in any manner that permits observation of the cyclic hysteretic behavior of the connection assembly, when subjected to multiple cycles, through the elastic and inelastic ranges of behavior. It is recommended that as a minimum, at least two cycles of loading be applied to the specimen at each of the following load increment steps:

1. One half the computed elastic capacity of the section
2. The computed yield capacity of the section
3. Angular displacements of the assembly equivalent to 1.5, 2, 2.5, 3, 3.5 ... times the computed yield angular displacement of the section. Loading should continue until assembly failure is obtained.

Dynamic loading is not required to satisfy these Guidelines.

6.5.1.3 Data Recording

As a minimum, the following data should be recorded throughout each test.

1. Angular rotation of assembly, as indicated in Figure 6-11, at each increment of load. Note - angular rotation is taken as equivalent to inter-story drift angle.
2. Applied loading at each increment, and computed moment at critical sections.
3. Any observable damage to assembly

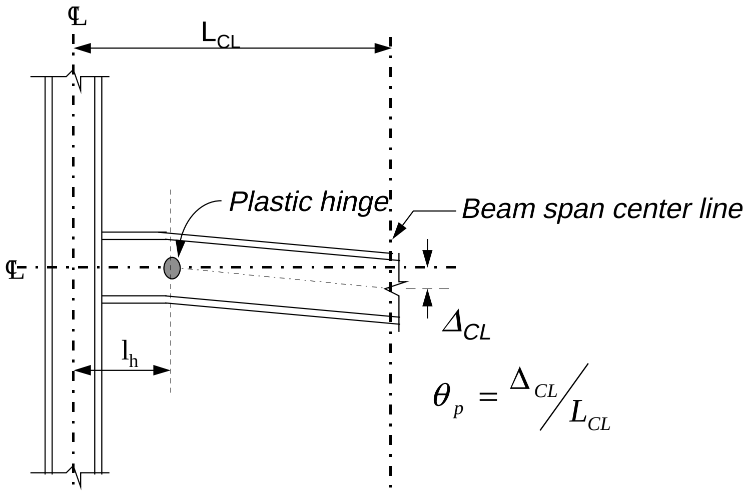


Figure 6-11 - Inter-story Drift Angle, θ

6.5.1.4 Data Reporting

Data reported to the building official should include a description of the test setup, the test specimen, the loading protocol employed, the location and date where the test was performed, the person in responsible charge of the test, hysteretic data for test expressed in an M-θ format and observations of damage at various load increments.

6.5.2 Acceptance Criteria

The inter-story drift capacity, at various performance levels shall be defined as indicated in Figure 6-11. The capacity shall be taken as indicated in Table 6-10.

Table 3-12 Inter-story Drift Capacity

Performance Level	Inter-story Drift Capacity
Incipient Damage	Taken at that value of θ, per Figure 6-11, at which peak load resistance occurs.
Collapse Prevention	Taken at that value of θ, per Figure 6-11, at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

6.5.3 Analytical Prediction of Behavior

Connection qualifications should include development of an analytical procedure to predict the behavior of the connection assembly, as demonstrated by the qualifications tests. The analytical procedure should permit identification of the strength and deformation demands on various elements of the assembly at the various stages of behavior and should identify the critical load limiting mechanisms. The analytical procedure should be sufficiently detailed to permit design of connections employing similar, but not identical, members to those tested.

Commentary: It is important for the designer to have an understanding of the limiting behaviors of any connection detail so that it may be designed and specified on a rational basis for assemblies that differ in configuration from those tested.

6.5.4 Determination of Resistance Factor

A resistance factor shall be determined for each performance level, using the procedures of this section. For each performance level, a tabulation shall be made of the inter-story drift obtained from each of the tests, together with the natural logarithm of these inter-story drift values. The median value shall be selected. The standard deviation of the natural logarithms of the test values, $\sigma_{\ln(t)}$, shall be determined. The resistance factor shall be calculated from the equation:

$$\phi = e^{-\frac{k\sigma_{\ln(t)}{}^{2}}{2b}\left(1+\frac{1}{N-1}\right)} \tag{6-1}$$

where: k = the slope of the hazard curve for the project site, plotted in natural logarithmic coordinates. The value of k may be taken as 3 for any site

$\sigma_{\ln(t)}$ = the standard deviation of the natural logarithms of the inter-story drift capacities obtained from the test program

b = is a parameter that relates increasing ground shaking intensity to increasing inter-story drift demand. The value of b may be taken as 1.0.

N = the number of tests of the connection assembly contained in the data base

The value of ϕ need not be taken as less than 0.75 for the incipient damage state, or less than 0.5 for the collapse prevention state, which values may be used for any connection.

Commentary: The procedure for calculation of the resistance factors contained in this section is based on Proposed Statistical and Reliability Framework for Comparing and Evaluating Predictive Models for Evaluation and Design, and Critical Issues in Developing such Framework. Report No. SAC/BD-9703, August 27, 1997, by Wen and Foutch, and on Performance Based Analysis and Design Procedure for Moment Resisting Steel Frames, September, 1998, by Hamburger and

Cornell.

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7. STRUCTURAL SPECIFICATIONS

7.1 Scope

This section provides guidelines for development of those divisions of construction specifications related to the fabrication and erection of structural steel for upgrade of existing WSMF structures. The section is written to be compatible with the standard format of the Construction Specifications Institute (CSI) **SECTION 05100 - STRUCTURAL STEEL**, which is outlined below. Similar language should be provided in specifications using other formats.

As contained in this draft, this specification is identical to that contained in the Criteria for New Buildings. It may be necessary to make some revisions to this specification as needed to suit the peculiar requirements of retrofit construction.

PART 1 - GENERAL

1.01 SUMMARY
1.02 REFERENCES
1.03 DEFINITIONS
1.04 SUBMITTALS
1.05 QUALITY ASSURANCE
1.06 SCHEDULING AND SEQUENCING

PART 2 - PRODUCTS

2.01 MATERIALS
2.02 FABRICATION
2.03 FINISHES
2.04 SOURCE QUALITY CONTROL

PART 3 - EXECUTION

3.01 EXAMINATION
3.02 PREPARATION
3.03 ERECTION
3.04 CLEANING
3.06 FIELD QUALITY CONTROL

Many of the noted subsections have no changes recommended. Where changes to typical code language are proposed, the changes proposed are emboldened. The reasons for the noted changes are provided in the commentary which follows each change.

Note: My original outline for this module suggested that its form be a complete CSI format Structural Steel specification section (05100 in CSI)

with emboldened new recommendations, and commentary. Having now started to follow this format, I think it will be better to provide only those sections for which special provisions are recommended. An alternate format would be to simply provide specification related information gleaned from FEMA 267 and 267A and from the SOA Reports and other research and leave it to the engineer to incorporate it appropriately in the specification. I would like feedback from others on which approach would be preferred. For purposes of this 25% Draft, I am attaching a copy of our firmís entire Section 05100 which reflects recommendations of FEMA 267 and 267A and is emboldened to indicate changes which were made in response to those documents. When it is determined that certain sections have no recommended changes, they can be removed from the document; for now it is best to consider the specification in its entirety. It is intended to include some of the commentary from FEMA 267 and 267A to make this a stand-alone document, along with new commentary for additional changes coming from the Phase II research and development. It should be noted that the CSI format does not make a clear distinction between Quality Control and Quality Assurance as we typically understand them. The quality control guidelines will hopefully provide definitions and a recommended approach to incorporate these into the specifications.

{Sources:

1. **FEMA 267 and 267A**
2. **ìSOA Report on Structural Steelî, Frank**
3. **ìSOA Report on Structural Welding for Seismic Applicationsî, Liu/Shaw}**

PART 1 - GENERAL

1.01 SUMMARY

A. Section Includes

1. Structural steel.

2. Reinforcing steel welded to structural steel.

3. Grout for baseplates and bearing plates.

B. Products Furnished But Not Installed Under This Section

1. Anchor bolts and steel fabrications cast into concrete are installed under Section 03100.

C. Related Sections

1. Section 05300 - Metal Decking: For shear connector studs attached to top flanges of beams for composite beam construction.

1.02 REFERENCES

A. ASTM - American Society for Testing and Materials

1. A6 - Specification for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use.

2. A36 - Specification for Steel.

3. A53 - Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless.

4. A123 - Specification for Zinc (Hot Dip Galvanized) Coating on Iron and Steel Products.

5. A153 - Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware.

6. A307 - Specification for Carbon Steel Externally Threaded Standard Fasteners.

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

8. A354 - Specification for Quenched and Tempered Alloy Steel Bolts, Studs and Other Externally Threaded Fasteners.

9. A449 - Specification for Quenched and Tempered Steel Bolts and Studs.

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

11. A500 - Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing.

12. A563 - Specification for Carbon and Alloy Steel Nuts, 1990 Edition.

13. A572 - Specification for High Strength Low Alloy Columbium-Vanadium Steel of Structural Quality.

14. A913 - Specification for High Strength Low Alloy Shapes of Structural Quality Produced by Quenching and Tempering Process.

15. Commentary: ASTM A913 Grades 50 and 65 are now accepted for seismic use in the AISC Seismic Provisons.

16. A615 - Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement.

17. A706 - Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement.

18. A780 - Specification for Repair of Damaged Hot-Dip Galvanized Coatings.

19. C1107 - Specification for Packaged Dry, Hydraulic-Cement Grout (Nonshrink).

20. F844 - Specification for Washers, Steel, Plain (Flat) Unhardened for General Use.

B. AISC - American Institute of Steel Construction

1. Specification - Load and Resistance Factor Design Specification for Structural Steel Buildings, December 1, 1993.

2. Specification - Seismic Provisions for Structural Steel Buildings, 1997

3. Commentary: The 1997 NEHRP and upcoming International Building Code (IBC) will be based on the provisions of the above specifications, therefore, it is appropriate to include them here.

4. Code - Code of Standard Practice for Steel Buildings and Bridges, 1992 Edition. Articles 3.2 and 3.3 and Section 4 and 9 of AISC Code are superseded by requirements of the General Conditions, Special Conditions and Contract Documents.

C. AWS - American Welding Society

1. D1.1 - Structural Welding Code, 1996 Edition.

2. D1.4 - Structural Welding Code - Reinforcing Steel, 1992 Edition.

D. ICBO - International Conference of Building Officials

1. UBC - Uniform Building Code, 1997 Edition.

E. SSPC - Steel Structures Painting Council's, "Systems and Specifications".

1. SP1 - Solvent Cleaning.

2. SP2 - Hand Tool Cleaning.

3. SP3 - Power Tool Cleaning.

4. SP6 - Commercial Blast Cleaning.

1.03 DEFINITIONS

A. Architecturally Exposed Structural Steel (AESS):

1. Structural steel framing exposed to view from the building exterior.

2. Structural steel framing noted as AESS on Drawings**.**

B. Heavy Shapes: ASTM A6, Group 3 shapes with flanges thicker than 1-1/2-inches and Group 4 shapes and Group 5 shapes; welded built-up members with plates exceeding 2-inches in thickness.

C. Commentary: The IG Section 8.1.4 recommends that toughness be specified for these sections, therefore, they need to be defined here.

D. Seismic Critical Weld:

1. Complete penetration welds in beam to column connections, including flange, flange reinforcement, stiffener plate and doubler plate welds.

2. Complete penetration welds of column splices and of columns to baseplates.

3. Other complete penetration welds indicated as "Seismic Critical" on Drawings.

4. Commentary: The IG recommends various new requirements for these welds, therefore, they are defined here.

1.04 SUBMITTALS

A. Shop Drawings:

1. Provisions of AISC Code, Section 4, are superseded by requirements of General Conditions, Special Conditions, and Section 01300 of these specifications.

2. Show size and location of structural members; give complete information necessary for the fabrication of members including cuts, copes, holes, stiffeners, camber, type and size of bolts and welds, surface preparation and finish; show methods of assembly.

3. Indicate welded connections using standard AWS symbols and clearly distinguish between shop and field welds.

4. Identify high strength bolted connections (snug-tight or slip-critical).

B. Certificates of compliance with specified standards.

1. All steel.

2. Fasteners, including nuts and washers.

3. Welding electrodes.

4. Studs.

5. Nonshrink Grout.

6. Reinforcing steel.

7. Primer Paint.

C. Certified manufacturerís test reports: Submit to Testing Laboratory for record purposes.

1. All Steel: Tensile tests and chemical analysis. welds. **Include all trace elements for steel to receive Seismic Critical Welds.**

2. Commentary: The IG section 8.1.2 commentary notes ìThe analysis and reporting of an expanded set of elements should be possible, and could be beneficial in the preparation of welding procedure specifications (WPSís) by the welding engineer if critical welding parameters are required.î

3. High Strength Bolts: As per ASTM A325-94, Section 14; or A490-93, Section 16.

4. Reinforcing Steel: Chemical, tensile and bend tests.

5. Heavy Shapes: Charpy V-Notch

6. Commentary: See commentary under 1.03 B. above.

D. Product Data

1. Welding Electrodes.

E. Welder Certification

F. Written Welding Procedure Specification (WPS) in accordance with AWS D1.1 requirements for each different welded joint proposed for use, whether prequalified or qualified by testing.

1. Indicate as-detailed configuration and also the maximum and minimum fit-up configurations.

2. Identify specific electrode and manufacturer.

3. List actual values of welding parameters to be used so that clear instruction is provided to welders.

G. Commentary: The IG section 8.2.2 provides extensive commentary on this issue.

H. Procedure Qualification Record (PQR) in accordance with AWS D1.1 for all procedures qualified by testing.

I. Samples: As requested by the Testing Laboratory.

1.05 QUALITY ASSURANCE

A. Code and Standards: Comply with provisions of following, except as otherwise indicated:

1. AISC "Code of Standard Practice for Steel Buildings and Bridges", 1992 Edition. Articles 3.2 and 3.3 and Sections 4 and 9 of AISC Code are superseded by requirements of the General Conditions, Special Conditions and Contract Documents.

2. AWS D1.1 "Structural Welding Code - Steel."

3. ICBO UBC Chapter 22, Division IX, "Allowable Stress Design and Plastic Steel Design for Structural Steel Buildings."

4. ICBO UBC Chapter 22, Division IV, "Structural Joints Using High Strength Bolts.

B. Qualifications for Welding Work: Qualify welding procedures and welding operators in accordance with AWS D1.1, "Qualification" requirements.

1. Qualify welders in accordance with AWS D1.1 for each process, position and joint configuration.

2. WPSís for each joint type shall indicate proper AWS qualification and be available where welding is being performed.

3. Welders who have not performed the required welding procedure for a period of six or more months shall be requalified.

4. Welders whose work fails to pass inspection shall be requalified before performing further welding.

5. If recertification of welders is required, retesting will be Contractor's responsibility.

C. Pre-Fabrication/Pre-Erection Conferences: Contractor shall schedule meeting with Architect, Testing Laboratory and the Steel Fabricator and Erector’s personnel supervising shop and field welding to review welding procedures and inspection requirements for "Seismic Critical Welds."

D. Commentary: The IG section 9.1.1 recommends that such a conference be held ìto plan and discuss the project and fabrication procedures.î

E. Welding Inspector Qualifications: All welding inspectors shall be AWS certified welding inspectors (CWI) as defined in AWS Standard and Guide for Qualification and Certification of Welding Inspectors, latest edition. Welding inspectorís qualifications shall be submitted to the Structural Engineer for approval. Inspectors shall be trained and thoroughly experienced in inspecting welding operations. Comply with AWS section 6.1.3.

F. Commentary: The IG section 10.1 provides recommendations for qualification of welding inspectors.

1.06 SCHEDULING AND SEQUENCING

A. Ensure timely delivery of items to be embedded in work of other sections such as cast-in-place concrete; furnish setting drawings or templates and directions for installation.

PART 2 - PRODUCTS

2.01 MATERIALS

A. General: All steel shall be identified as required by ICBO UBC Section 2202.2. Steel which is not properly identified shall be tested to show conformance with requirements of applicable ASTM Standard at Contractor’s expense.

B. Exposed Surfaces: For fabrication of work that will be exposed to view, use only materials that are smooth and free of surface blemishes including pitting, rust and scale seam marks, roller marks, rolled trade names, and roughness. Remove such blemishes by grinding or by welding and grinding, prior to cleaning, treating, and applying surface finishes.

C. Steel W Shapes: Dual Certified ASTM A36/A572

1. Heavy Shapes (see "Definitions" in this Section) shall be supplied with Charpy V-Notch testing in accordance with ASTM A6 Supplementary Requirement S5. The impact test shall meet a minimum average value of 20 ft-lbs absorbed energy at +70☜F and shall be conducted in accordance with ASTM A673, frequency H, with the following exceptions:

a) The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.

b) Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

c) Commentary: The above is recommended in section 8.1.4 of the IG.

D. Steel Channels and Angles: ASTM A36; or dual certified ASTM A36/A572.

E. Steel Plates and Bars:

1. ASTM A572, Grade 50, unless indicated otherwise.

2. ASTM A36 where designated on Drawings.

F. Steel Pipes: ASTM A53, Type S, Grade B.

G. Steel Tubing: ASTM A500, Grade B.

H. Standard Threaded Fasteners: ASTM A307, Grade A, bolts with ASTM A563 hex nuts.

I. High Strength Bolts:

1. ASTM A325, type 1; unless indicated otherwise.

2. ASTM A490 where designated on Drawings.

3. Nuts and washers conforming to ICBO UBC Section 2221.

J. Anchor Bolts (unless otherwise indicated on Drawings):

1. 1-inch diameter and smaller bolts: ASTM A307, Grade A.

2. Larger than 1-inch diameter bolts: ASTM A449.

3. Washers: ASTM F844; 5/16-inch minimum thickness.

4. Nuts: ASTM A563, heavy hex.

K. Anchor Bolts (where designated on Drawings):

1. ASTM A354, Grade BD, externally threaded rod; form head with ASTM F436 hardened washer between double ASTM A563, DH, heavy hex nuts.

2. Plate washer: ASTM F844; 1/2-inch minimum thickness.

3. Nuts: ASTM A563, Grade DH, heavy hex.

L. Welding Materials: AWS D1.1; type required for base metals being welded.

1. Electrodes shall be low hydrogen.

2. Electrodes for "Seismic Critical Welds" shall have a minimum Charpy V-notch toughness of 20 ft-lbs at -20☜F.

3. Commentary: The IG recommended that a notch toughness of 20 ft-lbs at 0 degrees F be used. Electrodes with toughness of 20 ft-lbs at -20 degrees are readily available and are specified in the AISC Seismic Provisions.

M. Shop Primer:

1. Type A Primer: Conforming to federal, state and local v.o.c. regulations; containing no lead or chromates; Tnemec Series FD88, or approved equal.

2. Type B Primer: Organic zinc-rich urethane; conforming to federal, state and local v.o.c. regulations; Class A coating in accordance with ICBO UBC Chapter 22, Division IV; Tnemec "90-97 Tneme-Zinc", or approved equal.

N. Studs:

1. Headed Shear Connector Studs; AWS D1.1, Type B; as-welded size as shown on Drawings.

2. General Purpose Studs; AWS D1.1, Type A; as-welded size and configuration as shown on Drawings.

O. Reinforcing Steel: ASTM A706, deformed.

P. Nonshrink Grout: Premixed, nonmetallic, noncorrosive product, complying with ASTM C1107, Class B or C, at flowable consistency for 30 minutes for temperature extremes of 45?F to 90☜F.

1. Products: Subject to compliance with requirements, provide one of the following:

2. Euco N.S., Euclid Chemical Co.
Masterflow 928, Master Builders.
Five Star Grout, U.S. Grout Corp.
Sika Grout 212, Sika Corp.

2.02 FABRICATION

A. Fabricate structural steel in accordance with AISC Specification and AISC Code.

1. Architecturally Exposed Structural Steel shall conform to Section 10 of AISC Code.

2. Fabricate joints in heavy shapes in accordance with additional requirements of Section A 3.1(c) of AISC Specification.

B. Connections: Where connection is not shown, design in accordance with standard practice unless otherwise directed by the Architect.

C. Drill, not punch, holes centered 6" or less from an edge to be complete penetration welded.

D. Commentary: Although not covered by the IG, it is recognized that punching of holes creates local embrittlement and sometimes cracks, which, when located near a welded edge, such as for erection bolts near a web CP weld, can lead to cracking of the base metal when high tensile stresses are resisted by the adjacent welds.

E. Assembly with High Strength Bolts

1. Construct connections in accordance with ICBO UBC, Chapter 22, Division IV, using provisions for slip-critical joints, unless snug-tight bolts are indicated on Drawings.

2. Use standard holes, unless otherwise indicated on Drawings.

F. Assembly with Standard Threaded Fasteners

1. Draw up tight, check threads with chisel or provide approved lock washers or self-tightening nuts.

2. Provide beveled washers under bolt heads or nuts resting on surfaces exceeding five percent slope with respect to head or nut.

G. Welded Construction

1. Examine fit-up of joint for conformance with welding procedure specification. Do not proceed with welding until fit-up is inspected by Testing Laboratory.

2. Weld in accordance with AISC Specification using manual shielded arc method or flux cored arc method in accordance with AWS D1.1. **Weld only in accordance with welding procedure specifications (WPS) for joint, which are to be available to welders and inspectors during the production process.**

3. Commentary: This is recommended by the IG section 8.2.2.

4. Groove welds shall be complete joint penetration welds, unless specifically designated otherwise on Drawings. Groove preparation is at Contractor's option, subject to qualification in accordance with AWS D1.1. Runoff plates shall be in accordance with AWS D1.1; **end dams shall not be used**.

5. Remove back-up plates for complete joint penetration welds where indicated in Contract Documents or when requested by Testing Laboratory to perform nondestructive testing. Remove at no additional cost to Owner.

6. Complete penetration groove weld Heavy Shapes in accordance with AISC Specification Section J1.7 for tension splices.

7. The following additional requirements apply to "Seismic Critical Welds":

a) Use electrodes specified for Seismic Critical Welds.

b) At beam flange to column welds, remove back-up plates, back gouge, clean by grinding and back weld with reinforcing fillet, unless Drawings specifically indicate that back-up bar may remain. Do not place reinforcing fillet until Testing Lab has inspected groove weld.

c) Cut off runoff plates 1/8-inch from edges and grind smooth (not flush).

d) Commentary: Electrode requirements are covered in the IG in section 8.2.3 and above in 2.01 L.2. The majority of *successful tests of connection specimens which require CP welding of the beam flange and/or cover plates to column flanges have had backing removed and reinforcing fillets added as described. Backing left in place frequently conceals incomplete fusion at the root of the weld, makes its detection by UT difficult, and represents a possible source of stress concentration (a notch) in itself. Removal of backing and back gouging eliminates concern about the weld and inspection and eliminates the stress concentration caused by the backing. The the back weld and reinforcing fillet fills the area of the back gouge and provides a smooth transition which reduces the stress concentration inherent in the connection of perpendicular members. The requirement to cut off the runoff plates and grind smooth provides a more gradual transition than leaving them in place and permits visual or NDT inspection of the end of the weld. The weld should not be ground flush as the grinding may gouge the base metal and cause a stress concentration.*

8. Weld reinforcing steel to structural steel in accordance with AWS D1.4 using prequalified procedures.

9. Grind exposed welds of Architecturally Exposed Structural Steel smooth and flush with adjacent finished surface.

H. Column Bases: Finish in accordance with AISC Specification. Lack of contact bearing with column shall not exceed 1/16 inch.

I. Bearing Plates: Provide for attached or unattached installation under beams, and girders resting on footings, piers, and walls.

J. Headed Studs: Automatically end weld in accordance with AWS D1.1 and manufacturer’s recommendations in such a manner as to provide complete fusion between the end of the stud and steel member.

2.03 FINISHES

A. Preparation of Surfaces

1. All surfaces shall be cleaned per SSPC-SP1 "Solvent Cleaning" to remove oil and grease prior to any other surface preparation.

2. After fabrication, prepare the following steel surfaces in accordance with SSPC-SP2 "Hand Tool Cleaning":

a) Steelwork to be spray-fireproofed.

b) Steelwork to be encased in concrete.

c) Steelwork to be hot-dip galvanized.

3. After fabrication, prepare the following steel surfaces in accordance with SSPC-SP3 "Power Tool Cleaning":

a) Interior steelwork to be painted with Type A Primer.

4. After fabrication, prepare the following steelwork in accordance with SSPC-SP6 "Commercial Blast Cleaning":

a) Exterior steelwork.

b) Architecturally Exposed Structural Steel.

c) Interior steelwork to receive Type B primer.

B. Painting

1. Apply one coat of primer to all structural steel surfaces unless otherwise noted. Do not paint the following surfaces:

a) Surfaces to be encased in concrete except initial two inches.

b) Surfaces to contact high-strength bolt connections, except surfaces painted with Type B Primer.

c) Surfaces to be field welded.

d) Surfaces to be spray fireproofed.

e) Top surfaces of beams to receive metal deck.

2. Use Type A Primer applied at 2.0 mils minimum dry film thickness on all normal environment interior steelwork.

3. Use Type B Primer applied at 2.5 mils minimum dry film thickness on all exterior steelwork and on interior steelwork subjected to wet conditions or corrosive fumes (noted on Drawings).

4. Permit thorough drying before shipment.

C. Hot dip galvanizing:

1. Hot dip galvanize the following items:

   a) Items noted on Drawings as galvanized.

   b) Fasteners which connect galvanized components, except A490 bolts shall not be hot-dip galvanized.

2. Galvanize in accordance with the following:

   a) Steel members and fabrications: ASTM A123.

   b) Bolts, nuts, washers: ASTM A153.

3. Treat faying surfaces of slip-critical high strength bolted connections to achieve Class C surface in accordance with ICBO UBC Chapter 22, Division IV.

2.04 SOURCE QUALITY CONTROL

A. Inspection and testing will be performed under provisions of Section 01400.

B. The Testing Laboratory will:

1. Review manufacturer's test reports for compliance with specified requirements.

2. Verify material identification.

3. Inspect high-strength bolted connections as required by ICBO UBC Section 1701 for conformance with ICBO UBC Chapter 22, Division IV.

4. Inspect welding as required by ICBO UBC Section 1701 in accordance with AWS D1.1. The following should be performed for each weld:

   a) Verify Welding Procedure Specification (WPS) sheet has been provided and has been reviewed with each welder performing the weld. Welds not executed in conformance with the WPS are rejectable.

   b) Verify fit-up meets tolerances of WPS and mark joint prior to welding.

   c) Verify welding consumables per Contract Documents and WPS.

d) Verify welder qualification and identification.

e) Verify amperage and voltage at the arc with hand-held meters.

f) Observe preheat and interpass temperatures, weld pass sequence and size of weld bead.

5. For Seismic Critical Welds, inspect removal of back-up and run-off plates, preparatory grinding and execution of reinforcing fillet.

6. Nondestructive test all complete penetration groove welds larger than 5/16 inches by ultrasonic methods for conformance with the weld quality and standard of acceptance of AWS D1.1 for welds subject to tensile stress. Pass sound through the entire weld volume from two crossing directions to extent feasible.

7. Test column webs for cracking, using dye-penetrant or magnetic particle test, over 3" minimum zone above and below continuity plates after welding.

8. Commentary: This test is introduced to detect cracking which may occur in the ìK-Areaî as described in section 8.1.6 of the IG Advisory.

9. Ultrasonically inspect base metal thicker than 1?-inches for discontinuities behind welds in accordance with ICBO UBC Section 1703.3.

10. Periodically, inspect and test stud welding as required by ICBO UBC Section 1701 in accordance with AWS D1.1; review preproduction testing and qualification, periodically inspect welding and perform verification inspection and testing.

PART 3 - EXECUTION

3.01. EXAMINATION

A. Examine existing structure to support construction and verify the following:

1. Location and elevation of bearings and anchor bolts are correct.

2. Other conditions adversely affecting erection of steel are absent.

B. Do not begin erection before unsatisfactory conditions have been corrected.

3.02. PREPARATION

A. Supervise setting of anchor bolts and other embedded items required for erection of structural steel. Be responsible for correct bearing of steel and correct location of anchor bolts.

3.03. ERECTION

A. Erect structural steel in accordance with AISC Specification and AISC Code.

B. Grouting Baseplates and Bearing Plates: Prior to erection, clean and roughen concrete surface beneath baseplate to full 1/4" amplitude; clean bottom surface of baseplate of bond-reducing materials. After columns have been positioned and plumbed, flow nonshrink grout solidly between bearing surfaces to ensure no voids remain. Comply with manufacturer's recommendations for mixing, placing, finishing and curing of grout.

C. Where erection requires performing work of fabrication on site, conform to applicable standards of Fabrication Article.

D. Field corrections of major members will not be permitted without the Architect's prior approval.

E. Gas Cutting: Use of flame cutting torch will be permitted only after the Architect's prior approval and only where metal cut will not carry stress during cutting, stresses will not be transmitted through flame-cut surface and cut surfaces will not be visible in finished work.

1. Make cuts smooth and regular in contour.

2. To determine effective width of members so cut, deduct 1/8-inch from least width at cut edge.

3. Make radius of cut fillet as large as practical, but in no case less than one inch.

4. Do not use flame cutting torch to align bolt holes.

F. Field Touch-Up Painting: After erection, touch-up or paint field connections and abrasions in shop paint with same paint used for shop painting. Touch up galvanized surfaces in accordance with ASTM A780.

3.04. CLEANING

A. After erection, thoroughly clean surfaces of foreign or deleterious matter such as dirt, mud, oil, or grease that would impair bonding of fire-retardant coating, paint or concrete.

3.05. FIELD QUALITY CONTROL

A. Inspection and testing will be performed under provisions of Section 01400.

B. The Testing Laboratory will:

1. Inspect and test field high strength bolting and welding in accordance with SOURCE QUALITY CONTROL Article of this section.

END OF SECTION

8. MATERIALS AND FRACTURE-RESISTANT DESIGN

8.1 Scope

This section provides guidelines on materials selection and the basic properties and behavior of structural steel materials recommended for application in MRSF structures. Reference is made to standard industry specifications as well as recommended supplemental requirements for these materials. Guidance is provided on parent materials, welding materials and bolting. In addition, information is provided on the brittle fracture behavior of structural steel under certain conditions. Designers who are knowledge of the conditions that are conducive to the development of brittle fracture in steels can avoid many of these by applying appropriate practice in detailing and specifying materials and workmanship requirements.

8.2 Parent Materials

8.2.1 Steels

Designers should specify materials which are readily available for building construction and which will provide suitable ductility and weldability for seismic applications. Structural steels which may be used in the lateral-force-resisting systems for structures designed for seismic resistance without special qualification include those contained in Table 8-1. Refer to the applicable ASTM reference standard for detailed information.

Table 8-1 - Structural Steel Pre-qualified for Use in Seismic Lateral-Force-Resisting Systems

ASTM Specification	Description
ASTM A36	Carbon Structural Steel
ASTM A283 Grade D	Low and Intermediate Tensile Strength Carbon Steel Plates
ASTM A500 (Grades B & C)	Cold-Formed Welded & Seamless Carbon Steel Structural Tubing in Rounds & Shapes
ASTM A501	Hot-Formed Welded & Seamless Carbon Steel Structural Tubing
ASTM A572 (Grades 42 & 50)	High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
ASTM A588	High-Strength Low-Alloy Structural Steel (weathering steel)
ASTM A709	Structural Steel for Bridges
ASTM A913	High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching & Self-Tempering Process
ASTM A992	Standard Specification for Steel for Structural Shapes for Use in Building Framing

Structural steels which may be used in the lateral-force-resisting systems of structures designed for seismic resistance with special permission of the building official are those listed in Table 5-2. Steel meeting these specifications has not been demonstrated to have adequate weldability or ductility for general purpose application in seismic-force-resisting systems, although it may well possess such characteristics. In order to demonstrate the acceptability of these materials for such use in WSMF construction it is recommended that connections be qualified by test, in accordance with the guidelines of Chapter 3. The test specimens should be fabricated out of the steel using those welding procedures proposed for use in the actual work.

Table 5-2 - Non-pre-qualified Structural Steel

ASTM Specification	Description
ASTM A242	High-Strength Low-Alloy Structural Steel

Commentary: Many WSMF structures designed in the last 10 years incorporated ASTM A36 steel for the beams and ASTM A572 grade 50 steel for the columns. This provided an economical way to design structures for the strong column - weak beam provisions contained in the building code. Recent studies conducted by the Structural Shape Producers Council (SSPC), however, indicate that material produced to the A36 specification has wide variation in strength properties with actual yield strengths that often exceed 50 ksi. This wide variation makes prediction of connection and frame behavior difficult. Some have postulated that one of the contributing causes to damage experienced in the Northridge Earthquake was inadvertent pairing of overly strong beams with average strength columns.

The AISC and SSPC have been working for several years to develop a new specification for structural steel that would have both minimum and maximum yield values defined and provide for a margin between maximum yield and minimum ultimate tensile stress. AISC recently submitted and ASTM approved such a specification, A992, for a material with 50 ksi specified yield strength. The domestic mills began producing structural shapes to this specification late in 1998. It is expected this new material will replace A36 and A572 as the standard structural material for shapes for incorporation into lateral-force-resisting systems.

Under certain circumstances it may be desirable to specify steels that are not recognized under the UBC for use in lateral-force-resisting systems. For instance, ASTM A709 might be specified if the designer wanted to place limits on toughness for fracture-critical applications. In addition, designers may wish to begin incorporating ASTM A913, Grade 65 steel, as well as other higher strength materials, into projects, in order to again be able to economically design for strong column - weak beam conditions. Designers should be aware, however, that these alternative steel materials may not be readily available.

Note that ASTM A709 and A992 steel, although not listed in the building code as pre-qualified for use in lateral-force-resisting systems, actually meet or exceed all of the requirements for ASTM A36 and ASTM A572. Consequently, special *qualification of the use of this steel should not be required. Although A709 and A913 steels have not routinely been used in seismic applications they both have been approved by AWS (D1.1 and D1.5) as weldable in pre-qualified connections. Because of the superior welding properties as compared to A-36 and A-572, it is expected that A-992 will also be approved by AWS during the next approval cycle.*

8.2.2 Chemistry

ASTM specifications define chemical requirements for each steel. A chemical analysis is performed by the producer on each heat of steel. End product analyses can also be specified on certain products. A certified mill test report is furnished to the customer with the material. The designer should specify that copies of the mill test reports be submitted for his/her conformance review. In general, ASTM specifications for structural steels include maximum limits on carbon, manganese, silicon, phosphorous and sulfur. Ranges and minimums are also limited on other elements in certain steels. Chromium, columbium, copper, molybdenum, nickel and vanadium may be added to enhance strength, toughness, weldability and corrosion resistance. These chemical requirements may vary with the specific product and shape within any given specification.

Commentary: Some concern has been expressed with respect to the movement in the steel producing industry of utilizing more recycled steel in its processes. This results in added trace elements not limited by current specifications. Although these have not been shown quantitatively to be detrimental to the performance of welding on the above steels, the new A-992 specification for structural steel does place more control on these trace elements. Mill test reports now include elements not limited in some or all of the specifications. They include copper, columbium, chromium, nickel, molybdenum, silicon and vanadium. The analysis and reporting of an expanded set of elements is required, and could be beneficial in the preparation of welding procedure specifications (WPSs) by the welding engineer if critical welding parameters are required. Modern spectrographs used by the mills are capable of automated analyses. When required by the engineer, a request for special supplemental requirements beyond those listed above should be noted in the contract documents.

8.2.3 Tensile/Elongation Properties

Mechanical property test specimens are taken from rolled shapes or plates at the rolling mill in the manner and location prescribed by ASTM A6 and ASTM A370. Table 8-3 gives the basic mechanical requirements for commonly used structural steels. Properties specified, and controlled by the mills, in current practice include minimum yield strength, ultimate tensile strength and minimum elongation. However, there can be considerable variability in the actual properties of steel meeting these specifications. Table 8-4 presents statistical data on the range of strength values that may be expected of contemporary steels meeting the indicated specifications. This data is based on work performed by the Steel Shape Producers Council for the 1992 production year, supplemented by limited statistical surveys undertaken by the SAC project.

Table 8-3 - Typical Tensile Requirements for Structural Shapes

ASTM	Minimum Yield Strength, Ksi	Ultimate Tensile Strength, Ksi	Minimum Elongation % in 2 inches	Minimum Elongation % in 8 inches
A36	36	58-80[1]	21[2]	20
A242	42[4]	63 MIN.	21[3]	18
A572, GR50	50	65 MIN.	21[2]	18
A588	50	70 MIN.	21[3]	18
A709, GR36	36	58-80	21[2]	20
A709, GR50	50	65 MIN.	21	18
A913, GR50	50	65 MIN.	21	18
A913, GR65	65	80 MIN.	17	15
A992	50[5]	65 MIN.	21	18

Notes:
1- No maximum for shapes greater than 426 lb./ft.
2- Minimum is 19% for shapes greater than 426 lb. /ft.
3- Minimum is 18% for shapes greater than 426 lb./ft.
4. Minimum is 50 ksi for Shape Groups 1 and 2, 46 ksi for Shape Group 3
5. Yield to tensile ratio, max. of 0.85. Maximum yield strength 65 ksi.

Unless special precautions are taken to limit the actual strength of material incorporated into the work to defined levels, new material specified as ASTM A36 or A572 should be assumed to be A992 steel for connection demand calculations, whenever the assumption of a higher strength will result in a more conservative design condition.

Table 8-4 - Statistics for Structural Shapes

Statistic	A 36	Dual Grade	A572 Gr50	A913 Gr65	A922
Yield Point (ksi)					
Mean	49.2	55.2	57.6	75.3	
Minimum	36.0	50.0	50.0	68.2	No
Maximum	72.4	71.1	79.5	84.1	Data
Standard Deviation [s]	4.9	3.7	5.1	4.0	Available
Mean + 1 s	54.1	58.9	62.7	79.3	
Tensile Strength (ksi)					
Mean	68.5	73.2	75.6	89.7	
Minimum	58.0	65.0	65.0	83.4	
Maximum	88.5	80.0	104.0	99.6	
Standard Deviation [s]	4.6	3.3	6.2	3.5	
Mean + 1 s	73.1	76.5	81.8	93.2	
Yield/Tensile Ratio					
Mean	0.72	0.75	0.76	0.84	
Minimum	0.51	0.65	0.62	0.75	
Maximum	0.93	0.92	0.95	0.90	
Standard Deviation [s]	0.06	0.04	0.05	0.03	
Mean + 1 s	0.78	0.79	0.81	0.87	
Mean - 1 s	0.66	0.71	0.71	0.81	

Commentary: Design professionals should be aware of the variation in actual properties permitted by the ASTM specifications. This is especially important for yield strength. Yield strengths for ASTM A36 material have consistently *increased over the last 15 years so that several grades of steel may have the same properties or reversed properties, with respect to beams and columns, from those the designer intended. Investigations of structures damaged by the Northridge earthquake found some WSMF connections in which beam yield strength exceeded column yield strength despite the opposite intent of the designer.*

With the ASTM approval of the A992 structural steel, the production of dual certified steel (A36 and A572) probably will not occur. Similarly, it is unlikely that A36 or A572 steel will continue to be produced as structural shapes. Because it is produced as a single grade, it is unlikely that the A992 steel will have as much variation in properties as was experienced with the dual grade steels. However, it is uncertain as ot the future of A36 and A572 grades of steel in plate material. Because steel service centers carry inventories of A36, A572, and dual grade steel, it is advisable to be aware of the possibility that for a few years, structural shapes of this type may be incorporated in projects unless precautions are taken.

8.2.4 Toughness Properties

For critical connections, non-redundant components and unusual or difficult geometries involving Group 3 (with flanges 1-1/2 inches or thicker), 4 and 5 shapes and plates and built-up sections over two inches thick with welded connections, the designer should consider specifying toughness requirements on the parent materials. A Charpy V-Notch (CVN) value of 20 ft.-lb. at 70 degrees F. should be specified when toughness is deemed necessary for an application. Refer to Figure 8-1 for typical CVN test specimen locations. The impact test should be conducted in accordance with ASTM A673, frequency H, with the following exceptions:

a) The center longitudinal axis of the specimens should be located as near as practicable to midway between the inner flange surface and the center of the flange thickness at the intersection of the web mid-thickness. Refer to AISC LRFD specification, Section A3-1c, Heavy Shapes (American Institute of Steel Construction - 1993)

b) Tests should be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests. For the continuous casting process, the sample may be taken at random throughout the length of the beam or column. If rotary straightening is used to straighten the shape after cooling, test samples should be taken from the k-area as shown in Figure 8-1, and at least 10 feet from the end of a straightened piece.

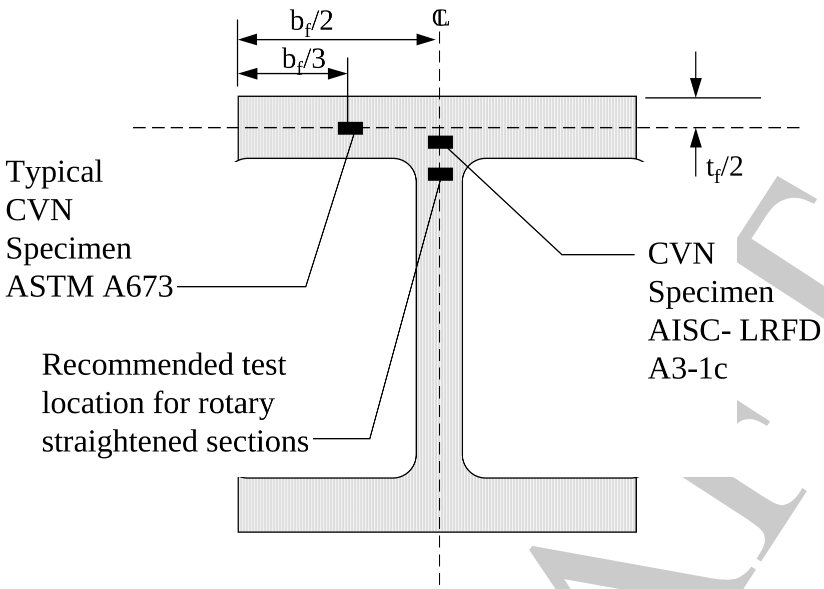

Figure 8-1 - Standard Locations for Charpy V-Notch Specimen Extraction, Longitudinal Only

Commentary: Many variables are recognized in analyzing the metallurgy of WSMF members. Until more research is available on the through-thickness properties of members thicker than two inches, a conservative approach is indicated. Specifying toughness properties in critical, unusual or non-redundant connections should be considered.

As temperature decreases or strain rate increases, toughness properties decrease. Charpy V-notch impact (CVN) tests, pre-cracked CVN tests and other fracture toughness tests can identify the nil ductility temperature (NDT) - the temperature below which a material loses all ductility and fractures in a brittle manner. On a microscopic level, this equates to a change in the fracture mechanism from shear to cleavage. Fracture that occurs by cleavage at a nominal tensile stress below yield is referred to as a brittle fracture. A brittle fracture can occur in structural steel when a particular combination of low temperature, tensile stress, high strain rate and a metallurgical or mechanical notch is present.

Plastic deformation can only occur through shear stress. Shear stress is generated when uniaxial or bi-axial straining occurs. In tri-axial stress states, the maximum shear stress approaches zero as the principal stresses increase. When these stresses approach equality, a cleavage failure can occur. Welding and other sources of residual stresses in combination with yield level seismic generated stresses can set up a state of tri-axial stress leading to brittle fractures if the connection is not properly detailed.

The necessity for minimum toughness requirements is not agreed to by all. There is also disagreement as to how much toughness should be required. The AWS Presidential Task Group recommended toughness values of 20 ft-lb. at different temperatures, depending on the anticipated service conditions. For base metal, a toughness of 15 ft-lbs at a temperature of 70 degrees F was recommended for enclosed structures and 40 degrees F for exposed structures. The 1993 AISC LRFD Specification, Section A3-1c, Heavy Shapes, requires toughness testing [Charpy V-Notch] under the following conditions for Group 4 and 5 shapes and plates exceeding 2 inches in thickness: a) When spliced using complete joint penetration welds; b) when complete joint penetration welds through the thickness are used in connections subjected to primary tensile stress due to tension or flexure of such members." Where toughness is required, the minimum value should be 20 ft-lb. at 70°F.

Plates thicker than two inches and sections with flanges thicker than 1-1/2 inches can be expected to have significantly variable grain sizes across the section. The slower cooling rate of the web-flange intersection in thick sections produces a larger grain size which exhibits less ductility and notch toughness.

ANSI/ASTM A673 and A370 establish the procedure for longitudinal Charpy V-notch testing. The impact properties of steel can vary within the same heat and piece, be it as-rolled, controlled rolled, or heat treated. Normalizing or quenching and tempering will reduce the degree of variation. Three specimens are taken from a single test coupon or location. The average must exceed the specified minimum, but one value may be less than the specified minimum but must be greater than the larger of two thirds of the specified minimum or 5 ft-lb. The longitudinal axis of the specimen is parallel to the longitudinal axis of the shape or final rolling direction for plate. For shapes, the specimen is taken from the flange 1/3 the distance from the edge of the flange to the web. The frequency of testing [heat or piece], the test temperature, and the absorbed energy are specified by the user. [NOTE: heat testing (frequency H) for shapes, means one CVN test set of samples from at least each 50 tons of the same shape size, excluding length, from each heat in the as-rolled condition. Piece testing (frequency P) for shapes, means one CVN test set of specimens from at least each 15 tons or each single length of 15 tons of the same shape size, excluding length, from each heat in the as-rolled condition.] Heat testing is probably adequate in most circumstances.

The specimen location required by ASTM A673 is not at the least tough part of a W shape. For a W shape, the volume at the flange web intersection has historically had the lowest ratio of surface area to volume and hence cools the slowest. This slow cooling causes grain growth and reduced toughness. The finer the grain, the tougher the material. Also, ASTM A673 does not specify where in the product run of an ingot to sample. Impurities tend to rise to the *upper portion of the ingot during cooling from molten metal. Impurities reduce the toughness of the finished metal. Hence, shapes produced from the upper portions of an ingot can be expected to have lower toughness, and samples should be taken from shapes produced from this portion of the ingot. In the continuous casting process, impurities tend to be more evenly distributed; hence, samples taken anywhere should suffice. The AISC LRFD specification requires testing from the upper portion of the ingot and near the web flange intersection. Even though the AISC LRFD specification does not require toughness testing for the typical WSMF connection, i.e., a Group 2 beam to a Group 4 column, recent tests at Lehigh (Ref.) on new continuous cast shapes and one old shape recovered from an existing building indicate that through-thickness properties of column flanges are not a concern.*

In response to concerns raised following the Northridge Earthquake, the AISC conducted a statistical survey of the toughness of material produced in structural shapes, based on data provided by six producers for a production period of approximately one year (American Institute of Steel Construction - 1995). This survey showed a mean value of Charpy V notch toughness for all shape groups that was well in excess of 20 ft-lb. at 70 degrees F. However, not all of the samples upon which these data are based were taken from the core area, nor at the k-area recommended by these Guidelines. Consequently, this survey does not provide definitive information on the extent to which standard material produced by the mills participating in this survey will meet the recommended values.

Rotary straightening of steel wide flange shapes produces large shear strains at the k-area that has been found to reduce the CVN toughness to low single digit toughness (Tide, 1997a, b). While this has not been demonstrated to have adverse affects on in-service performance of structural steel, it has been associated with fabrication related fractures.

8.2.5 Lamellar Discontinuities

For critical joints (beam to column CJP welds or other tension applications where Z-axis or tri-axial stress states exist), ultrasonic testing (UT) should be specified for the member loaded in the Z axis direction, in the area of the connection. A distance 3 inches above and below the location to be welded to the girder flange is recommended. The test procedure and acceptance criteria given in ASTM A898-91, Standard Specification for Straight Beam Ultrasonic Examination of Rolled Steel Structural Shapes, Level I, should be applied. This testing should be done in the mill or fabrication shop for new construction.

The possible occurrence of lamellar tearing can be minimized by following recommended procedures for welding highly restrained joints. These include detailing (AISC Ref.), preheating joint to temperatures in excess of the minimum requirements of AWS for the steel thicknesses involved in the connection and buttering layers of ductile and tough weld metal in the joint in the through-thickness direction.

Commentary: Prior to the Northridge earthquake very little test data existed on the through thickness properties of structural shapes nor were there any standard test methods for determining these properties. Nevertheless, the typical beam-column joints typically used in welded FR connections prior to the Northridge earthquake placed significant through-thickness demands on the flanges of columns and some fractures observed in damaged buildings following the Northridge earthquake were identified as potentially being the result of through-thickness failures of the material. Lamellar tearing, a form of through-thickness failure, had been a problem in the fabrication of heavy structural frames during the 1970s and this was again suspected to be a cause of some of these failures.

Extensive testing conducted as part of the SAC phase II investigations indicates that the through thickness strength of column shapes is not a significant limiting factor on connection behavior. Nevertheless, there is some potential for fabrication induced lamellar tearing of heavy weldments, particularly in steels having high sulfur contents. Laminations (pre-existing planes of weakness) and lamellar tearing (cracks parallel to the surface) will impair the Z axis strength and toughness properties of column material. These defects are mainly caused by non-metallic sulfides and oxides which begin as almost spherical in shape, and become elongated in the rolling process. When Z axis loading occurs from weld shrinkage strains or external loading, microscopic cracks may form between the discrete, elongated nonmetallic inclusions. As they link up, lamellar tearing occurs.

Longitudinal wave ultrasonic testing is very effective in mapping serious lamination discontinuities. Improved quality steel does not eliminate weld shrinkage and, by itself, will not necessarily avoid lamellar tearing in highly restrained joints. Ultrasonic testing should not be specified without due regard for design and fabrication considerations.

In cases where lamellar defects or tearing are discovered in erection or on existing buildings, the designer should consider the consequences of making repairs to these areas. Gouging and repair welding will add additional cycles of weld shrinkage to the connection and may promote crack extensions or new lamellar tearing. If weld repairs are attempted, carefully thought out repair detailing and weld procedure specifications should be prepared in advance.

8.3 Welding

8.3.1 Welding Process

Applicable welding processes for structural construction include shielded metal arc welding (SMAW), flux cored arc welding (FCAW), submerged arc welding (SAW), and gas metal arc weld (GMAW). Fabricators and erectors should be permitted to select the most appropriate process for each individual joint, given the limitations of access, production and worker qualifications. Contract documents should specify required strength and toughness properties for welding and usually should not attempt to limit process selection. Under some special conditions, including unusual base material chemistry and mixing of welding procedures (Ref.) or service conditions, some restrictions on weld processes or parameters may be appropriate and should be stated in the contract documents.

8.3.2 Welding Procedures

Welding should be performed within the parameters established by the electrode manufacturer and the Welding Procedure Specification (WPS), required under AWS D1.1. Either pre-qualified or qualified-by-test procedures may be utilized, if the procedure is capable of producing weld of the desired quality.

Commentary: Welding procedure specifications should be prepared by the fabricator and/or erector and should specify all parameters that must be controlled in making the weld. For example, the position (if applicable), electrode diameter, amperage or wire feed speed range, voltage range, travel speed range and electrode stickout (e.g. all passes, 0.072 in. diameter, 248 to 302 amps, 19 to 23 volts, 6 to 10 inches/minute travel speed, 170 to 245 inches/minute wire feed speed, 1/2" to 1" electrode stickout) should be established. Its importance in producing a high quality weld is essential. The following information is presented to help the engineer understand some of the issues surrounding these parameters.

The amperage, voltage, travel speed, electrical stickout and wire feed speed are functions of each electrode. If pre-qualified WPSs are utilized, these parameters must be in compliance with the AWS D1.1 requirements. For FCAW and SMAW, the parameters required for an individual electrode vary from manufacturer to manufacturer. Therefore, for these processes, it is essential that the fabricator/erector utilize parameters that are within the range of recommended operation published by the filler metal manufacturer. Alternately, the fabricator/erector could qualify the welding procedure by test in accordance with the provisions of AWS D1.1 and base the WPS parameters on the test results. For submerged arc welding, the AWS D1.1 code provides specific amperage limitations since the solid steel electrodes used by this process operate essentially the same regardless of manufacture. The filler metal manufacturer's guideline *should supply data on amperage or wire feed speed, voltage, polarity, and electrical stickout. The guidelines will not, however, include information on travel speed which is a function of the joint detail. The contractor should select a balanced combination of parameters, including travel speed, that will ensure that the code mandated weld-bead sizes (width and height) are not exceeded.*

8.3.3 Welding Filler Metals

The current AWS D1.1 requirements should be incorporated as written in the Code. The welding parameters should be clearly specified using a combination of the Project Specifications, the Project Drawings, the Shop Drawings and the welding procedure specifications, as required by AWS D1.1. For welding on ASTM A572 steel, the AWS D1.1 code requires the use of low-hydrogen electrodes. Low hydrogen practice should be specified regardless of the steel grade. With SMAW welding, a variety of non-low hydrogen electrodes are commercially available. These electrodes are not appropriate for welding on the higher strength steels used in building construction today, although they were popular in the past when lower strength steels were employed. All of the electrodes that are employed for flux cored arc welding (both gas shielded and self-shielded), as well as submerged arc welding, are considered low hydrogen. However, in some cases, the low hydrogen consideration is based on coupons that are artificially aged (Ref. AWS). Because deposited weld metal is not artificially aged, caution should be exercised and appropriate documentation obtained before automatically accepting a low hydrogen rating.

For critical joints (beam to column CJP welds or other tension applications where Z-axis loading or tri-axial stress states exist), toughness requirements for the filler metals should be specified. A minimum CVN value of 20 ft.-lb. at a temperature of -20 degrees F. should be required, unless more stringent requirements are indicated by the service conditions and/or the Contract Documents. The filler metal should be tested in accordance with the AWS A5 filler metal specification to ensure it is capable of achieving this level of notch toughness. The filler metal manufacturers Typical Certificate of Conformance, or a suitably documented test performed by the contractor, should be used to document the suitability of the electrode used. These tests should be performed for each filler metal by AWS classification, filler metal manufacturer and filler metal manufacturer's trade name. The sizes as specified by the AWS A5 document should be tested, although the exact diameter used in production need not be specifically tested. This requirement should not be construed to imply lot or heat testing of filler metals.

Electrode specification sheets should be provided by the Fabricator/Erector prior to commencing fabrication/erection.

Commentary: Although there are no notch toughness requirements for weld metal used in welding ASTM A 36 or A 572, Grade 50, A709, A913, and A992 steel under AWS D1.1, research conducted since the Northridge earthquake clearly demonstrates the benefits of incorporating notch tough weld metal in critical joints of MRSF construction. Most filler metals are fairly notch tough, but some will not achieve even a modest requirement such as 5 ft-lb. at + 70°F.

These guidelines recommend that critical joints be made with weld filler metal with rated notch toughness of 20 ft-lbs at -20^{o}F.

Welding electrodes for common welding processes include:

AWS A5.20: Carbon Steel Electrodes for FCAW
AWS A5.29: Low Alloy Steel Electrodes for FCAW
AWS A5.1: Carbon Steel Electrodes for SMAW
AWS A5.5: Low Alloy Steel Covered Arc Welding Electrodes (for SMAW)
AWS A5.17: Carbon Steel Electrodes and Fluxes for SAW
AWS A5.23: Low Alloy Steel Electrodes and Fluxes for SAW
AWS A5.25: Carbon and Low Alloy Steel Electrodes and Fluxes for Electroslag Welding

In flux cored arc welding, one would expect the use of electrodes that meet either AWS A5.20 or AWS A5.29 provided they meet the toughness requirements specified below.

Except to the extent that one requires Charpy V-Notch toughness and minimum yield strength, the filler metal classification is typically selected by the Fabricator. As an aid to the engineer, the following interpretation of filler metal classifications is provided below:

$E^1X^2X^3T^4X^5$ *For electrodes specified under AWS A5.20*
$E^1X^2X^3T^4X^5X^6$ *For electrodes specified under AWS A5.29*
$E^1XX^7X^8X^9X^{10}$ *For electrodes specified under AWS A5.1 or AWS A5.5.*

NOTES:

1. *Indicates an electrode.*

2. *Indicates minimum tensile strength of deposited weld metal (in tens of ksi, e.g., 7 = 70 ksi).*

3. *Indicates primary welding position for which the electrode is designed (0 = flat and horizontal and 1 = all positions).*

4. *Indicates a flux cored electrode. Absence of a letter indicates a "stick" electrode for SMAW.*

5. *Describes usability and performance capabilities. For our purposes, it conveys whether or not Charpy V-Notch toughness is required (1, 5, 6 and 8 have impact strength requirements while 2, 4, 7, 10 and 11 do not). A "G" signifies that the properties are not defined by AWS and are to be agreed upon between the manufacturer and the specifier. Impact strength is specified in terms of the number of*

foot-pounds at a given temperature (e.g., 20 ft-lb. at 0 degrees F). Note that for electrodes specified under AWS A5.20, the format for usage is "T-X".

6. *Designates the chemical composition of deposited metal for electrodes specified under AWS A5.29. Note that there is no equivalent format for chemical composition for electrodes specified under AWS A5.20.*

7. *The first two digits (or three digits in a five digit number) designate the minimum tensile strength in ksi.*

8. *The third digit (or fourth digit in a five digit number) indicates the primary welding position for which the electrode is designed (1 = all positions, 2 = flat position and fillet welds in the horizontal position, 4 = vertical welding with downward progression and for other positions.)*

9. *The last two digits, taken together, indicate the type of current with which the electrode can be used and the type of covering on the electrode.*

10. *Indicates a suffix (e.g., A1, A2, B1, etc.) designating the chemical composition of the deposited metal.*

Electrode Diameter: (See AWS D1.1 Section 4.14.1.2) Electrode diameter effects the rate of weld metal deposition and the heat imparted to the metal during welding. This can effect toughness of the completed joint. The following lists the maximum allowable electrode diameters for pre-qualified FCAW WPS's according to D1.1:

- *Horizontal, complete or partial penetration welds: 1/8 inch (0.125")**
- *Vertical, complete or partial penetration welds: 5/64 inch (0.078")*
- *Horizontal, fillet welds: 1/8 inch (0.125")*
- *Vertical, fillet welds: 5/64 inch (0.078")*
- *Overhead, reinforcing fillet welds: 5/64 inch (0.078")*

  * *This value is not part of D1.1-94, but will be part of D1.1-96.*

For a given electrode diameter, there is an optimum range of weld bead sizes that may be deposited. Weld bead sizes that are outside the acceptable size range (either too large or too small) may result in unacceptable weld quality. The D1.1 code controls both maximum electrode diameters and maximum bead sizes (width and thickness). Pre-qualified WPS's are required to meet these code requirements. Further restrictions on suitable electrode diameters are not recommended.

8.3.4 Preheat and Interpass Temperatures

The preheat temperatures and conditions given in AWS D1.1, Chapter 4 should be strictly observed with special attention given to Section 4.2, for the thickness of metal to be welded. For repair welding of earthquake damage, the *AASHTO/AWS D1.5 Bridge Welding Code* preheat requirements for fracture-critical, non-redundant applications should be considered.

Cracking of welds and heat affected zones should be avoided. One type of weld cracking is hydrogen induced cracking (HIC). For a given steel, variables that reduce HIC tendencies are prioritized as follows:

1. Lower levels of hydrogen.
2. Higher preheat and interpass temperatures.
3. Postheat.
4. Retarded cooling (insulating blankets).

Only low hydrogen electrodes should be used for fabrication and/or erection of seismically loaded structures. Proper preheat and interpass temperatures should be maintained. AWS D1.1 requirements are generally adequate for new construction.

Control of hydrogen and proper preheat and interpass temperature is much more powerful for overcoming HIC than postheat or retarded cooling methods. Retarded cooling has limited benefit if the entire piece is not preheated - obviously impractical for structural applications.

The engineer is encouraged to emphasize proper preheat and the use of low hydrogen electrodes and practice. If these measures are insufficient to prevent cracking, additional measures may be required to eliminate cracking. These measures may or may not call for additional preheat, postheat, or retarded cooling.

While low hydrogen electrodes and proper preheat is essential, postheat and retarded cooing is not generally required and should not be used for routine construction.

Commentary: There are two primary purposes for preheating and interpass temperature requirements:

(1) To drive off any surface moisture or condensation which may be present on the steel so as to lessen the possibility of hydrogen being introduced into the weld metal and HAZ, and

(2) To prevent the steel mass surrounding the weld from quenching the HAZ as cooling occurs after welding.

Virtually all weld repairs are made under conditions of high restraint. Consequently, higher preheat/interpass temperatures may be required for repair applications. As steel is cooled from the austenitic range (above about 1330 degrees F), it goes through a critical transition temperature. If it goes through that temperature range too fast, a hard, brittle phase called martensite forms (quenching). If it passes through that temperature range at a slower rate, ductile, tougher phases called bainite or ferrite/pearlite form. Preheating of the surrounding mass provides a slower cooling rate for the weld metal and HAZ.

The American Association of State Highway and Transportation Officials (AASHTO) recognizes repair welding as more critical in its guidelines for the repair of fracture-critical bridge members. The purpose, in part, is to allow more plastic flow and yielding, at welding temperatures, in the area near the weld. The requirements are given in Table 5-6:

Table 8-6 - AASHTO Preheat Requirements for Fracture Critical Repairs[1]

Steel	*Thickness, in.*	*Minimum Preheat/Interpass Temp., °F*
A36/A572	*to 1-1/2*	*325*
A36/A572	*>1-1/2*	*375*

1- Reference AASHTO/AWS D1.5-95 Bridge Welding Code

Preheat temperatures should be measured at a distance from the weld equal to the thickness of the part being welded, but not less than three inches, in any direction including the through thickness of the piece. Where plates are of different thicknesses, the pre-heat requirement for the thicker plate should govern. Maintenance of these temperatures through the execution of the weld (i.e. the interpass temperature) is essential. Maximum interpass temperatures should be limited to 550 degrees F for pre-qualified WPSs, for fracture-critical applications. Higher interpass temperatures could be employed if those higher temperature limits are qualified by test.

8.3.5 Postheat

Postheat is the application of heat in the 400 degrees F to 600 degrees F range after completion of welding. It may be helpful in mitigating some cracking tendencies.

Commentary: A postheat specification might require that complete joint penetration groove welds in existing buildings be postheated at 450 degrees F for two hours. The purpose of this postheat is to accelerate the removal of hydrogen from the weld metal and HAZ and reduce the probability of cracking due to hydrogen embrittlement. Hydrogen will migrate within the weld metal at approximately 1 inch per hour at 450 degrees F, and at about 1 inch per month at 70 degrees F. To the extent that hydrogen embrittlement is of concern, postheat is *one method of mitigating cracking. The use of low hydrogen electrodes, proper welding procedures, and uniformly applied and maintained preheat may represent a cost-effective method of addressing the problem of hydrogen embrittlement in lieu of postheat.*

When postheat is required, AASHTO/AWS D1.5-95 specifications require this to be done immediately upon completion of welding. The postheat is between 400 to 500 degrees F for one hour minimum, for each inch of the thickest member or for two hours, whichever is less.

8.3.6 Controlled Cooling

Most of the weldment cooling is effected by conductance within the steel rather than radiation. Retarded cooling should only be specified in cases where large weldments subject to significant residual stresses due to restraint (e.g. multiple members framing into one connection with Z axis loading) or ambient temperatures that would result in rapid cooling of large weldments. The length of time to cool down the weld and the level of insulation required are a function of weldment temperature, thickness of base metal and ambient temperature.

Commentary: Active systems of ramp-down cooling are generally not required; however, in highly restrained conditions they may offer an added advantage.

8.3.7 Metallurgical Stress Risers

Metallurgical discontinuities such as tack welds, air-arc gouging and flame cutting without preheating or incorporation into the final weld should not be permitted. Inadvertent damage of this type should be repaired by methods approved by the engineer, following the AWS D1.1 criteria and a specific WPS covering repairs of this type.

Commentary: Metallurgical stress risers may result from tack welds, air-arc gouging and flame cutting performed without adequate preheat. However, preheating is not necessarily required for air arc gouging or flame cutting used in the preparation of a surface to receive later welding. The subsequent heat input during the welding process should adequately anneal the affected area. The AWS D1.1 code requires the same preheating for tack welding operations as normal welding, with the exception of tack welds that are incorporated into subsequent submerged arc weld deposits.

Arc strikes can also be a source of metallurgical stress risers and should not be indiscriminately made. AWS D1.1 Section 3.10 indicates that "arc strikes outside the area of permanent welds should be avoided on any base metal. Cracks or blemishes caused by arc strikes should be ground to a smooth contour and checked to ensure soundness."

8.4 Bolting

Structural bolts employed in connections of MRSFs should conform to one of the standard types indicated in Table 8-7 and to the applicable requirements of the ASTM specifications.

Table 8-7 - Structural Bolts for Moment-Resisting Steel Frame Construction

Specification	Description	Remarks
ASTM A307	Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength	Should not be used in combination with welds on the same joint
ASTM A325	High Strength Bolts for Structural Steel Joints	Should not be used in the same plane in combination with welds to transfer loads
ASTM A490	Heat-treated Steel Structural Bolts, 150 ksi minimum tensile strength	

8.5 Fracture Mechanics Principles

This section provides basic information on the principles of fracture mechanics.

Commentary: Structural steel and weld metal are generally regarded as a ductile material capable of extensive inelastic deformation prior to development of tensile fractures. However, under certain condition, these highly ductile materials can behave in a brittle manner resulting in the development of unstable fractures with relatively little plastic deformation. The conditions that can lead to such brittle behavior and engineering approaches to judging the severity of these conditions are presented in this section.

8.5.1 Introduction

Brittle fracture can be described as a dynamic propagation of an unstable crack. Brittle fracture occurs when the state-of-stress at the crack tip reaches a critical magnitude resulting in an unstable crack. The relationship between stress, stress intensity factor and crack size is given by the relationship:

$$K = F\sigma\sqrt{\pi a}$$

where:

K = stress intensity factor, ksi $(in)^{1/2}$
F = non-dimensional constant
σ = nominal stress, ksi
a = crack size, in.

8.5.2 Crack Geometry

The non-dimensional term, F, allows for various geometric conditions in the vicinity of the crack (a) including crack location and size relative to the primary member. Evaluation of cracks located on the surface, subsurface, edge or through the full thickness, etc. of the member each require a different value of the coefficient F. Methods for determining F are documented in the literature (Barsom - 1987, Tada - 1985 and Fisher - 1984). In welded structures, initial cracks can result from weld discontinuities such as porosity, slag inclusions, lack-of-fusion, undercut and backing bar notches.

8.5.3 Stress Variables

Conventional engineering mechanics techniques are used to compute the nominal stress (σ) at the crack tip. In addition to stresses resulting from external forces, residual stresses from welding must be considered when welded connections are involved.

8.5.4 Stress Intensity Factor

The stress intensity factor (K) at the crack tip is calculated and compared to the notch toughness of the material in the vicinity of the crack. The appropriate notch toughness must be determined for the comparison to be valid. Specifically, it must be decided whether the stress intensity factor is compared to notch toughness based on a plane stress (K_c) or plane strain (K_{Ic}) condition for slow loading or a plane strain condition (K_{Id}) for dynamic loading. If the stress intensity factor is less than the material notch toughness, the crack will remain stable, and either elastic or plastic deformations will occur. Stress intensity factors greater than the material notch toughness indicate that brittle fracture is probable.

8.5.5 Temperature

Temperature and loading strain rate are variables that must be accounted for when determining notch toughness of a material. The relationship between notch toughness, temperature and strain rate is shown schematically in Figure 8-2. Typically, as temperature increases so does notch toughness and as the strain rate increases notch toughness decreases. This general statement is correct provided a lower transition temperature for notch toughness is exceeded. Similarly, the notch toughness increases until a limiting value is reached at some temperature and strain rate.

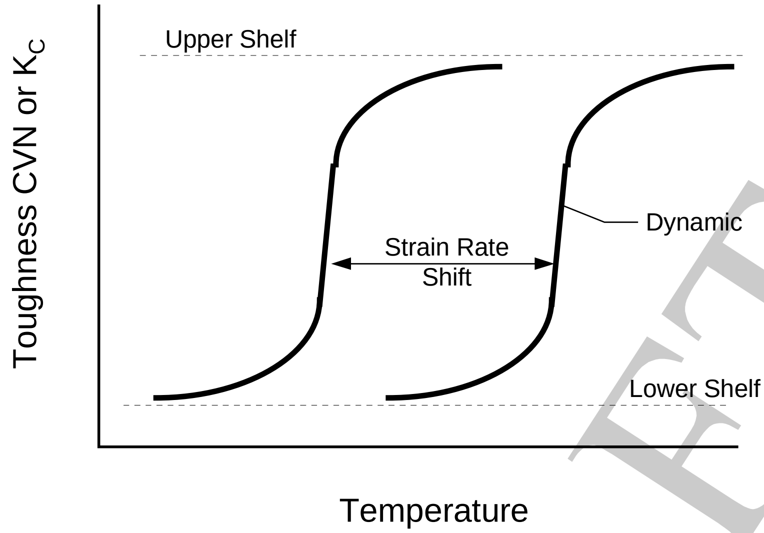

Figure 8-2 - Schematic Relationship Between Notch Toughness, Temperature and Strain Rate

8.5.6 Determining Notch Toughness

Over the years, numerous test methods have been developed to determine notch toughness. Many of these tests have been developed for specific purposes, others are more general but also more costly or difficult to perform. The Charpy V-notch (CVN) test fulfills several functions. Overall it is relatively inexpensive and therefore suitable for use as a quality control procedure. All specimens are identically manufactured with only the test temperature a variable. Provided reasonable care is exercised during production and testing, acceptable test repetitiveness can be accomplished. Conversion of CVN data to dynamic notch toughness and hence to static notch toughness or some intermediate strain rate is done using an empirical relationship such as:

$$K_{ID} = \sqrt{5E(CVN)} \Big/ 1000$$

where:

K_{ID} = dynamic notch toughness ksi $(in)^{1/2}$
E = modulus of elasticity, psi
CVN = Charpy V-notch, ft-lbs

and for structural steels:

$$T_{shift} = 215 - 1.5F_{ys}$$

where:

T_{shift} = temperature shift to convert K_{ID} to K_{IC}, ^{o}F
F_{ys} = room temperature yield strength, ksi

The original of these empirical equations is given by Barsom - 1987.

8.5.7 Roll of Notch Toughness

Structural steel during fabrication and subsequent use is subjected to various uses that result in irregular surface and loading conditions. Whenever the loading conditions and geometric arrangements result in tensile stresses and stress concentrations, brittle fracture is a possibility. Industry standards for material production and workmanship typically limit the size of discontinuities and cracks. Within these limits, nominally expected notch toughness is sufficient to ensure that yielding and plastic flow can occur before the onset of brittle fracture.

As the size of the crack increases, the criticalness of the notch toughness in the region of the crack tip becomes paramount. Combining natural cracks, such as backing bar geometry with a welding slag inclusion, compounds the problem and increases the need for notch tough material. Because there are going to be various levels of discontinuities, either from design or from workmanship, there must also be an expected and mandatory minimum level of notch toughness in the base metal and weld metal.

8.5.8 Base Metal and Weld Metal Notch Toughness

As construction of SMFs evolved from riveted and bolted connections to welded connections, the roll of notch toughness also evolved. Initially, welding was performed using shielded metal arc (SMAW) which was questionable concerning notch toughness and hydrogen levels. As better grades of SMAW electrodes evolved, such as E7018, with CVN toughness of at least 20 ft-lbs at -20^{o}F, notch toughness was not an issue of concern and hydrogen induced problems were essentially eliminated. With this type of welding material, the critical location for crack initiation and propagation was located in the heat-affected zone (HAZ).

Subsequently, as self-shielded flux cored arc welding (FCAW-S) was developed, the notch toughness and low hydrogen issues unexpectedly returned. Because of the high deposition rate and therefore greatly reduced cost, FCAW-S welding replaced SMAW for field applications. During the 20 years preceding the 1994 Northridge earthquake, the most commonly employed grade of FCAW-S wire was the American Welding Society (AWS) designation E70T-4 with properties specified in AWS A5.20: Carbon Steel Electrodes for Flux cored Arc Welding. Tests of this product indicate CVN toughness values in the low single digits at 70^{o}F can be expected. At this level of notch toughness the critical defect location is now in the weld metal and not the HAZ. Under these conditions, any weld root defect has the potential to become fracture critical and a potential source of brittle fracture initiation. Numerous examples extracted from Northridge earthquake damaged buildings confirm this scenario.

Commentary: The relationship between hydrogen level and notch toughness is not clearly identified in the literature and therefore there is no way to quantify the effects of hydrogen on notch toughness. Artificial aging of FCAW weld metal is not included in the AWS coupon preparation (AWS A5.20-95) for Charpy V-notch *samples. Artificial aging of tensile coupons (permitted by AWS) tends to decrease hydrogen levels and increase ductility. Because deposited weld metal in WSMF connections is not artificially aged, the use of any FCAW-S filler metal that does not have a specified CVN values in AWS A5.20 and A5.29 should not be used. Until familiarity with a specific FCAW-S filler metal is developed, supplemental CVN testing of as-deposited weld metal in accordance with ASTM 673 may be appropriate.*

8.6 Connections Conducive to Brittle Fracture

8.6.1 Loading Conditions

In typical welded, unreinforced beam-column joints, a critical state-of-stress occurs at the interface between the beam flange and the column flange under severe rotational loading of the connection. Such loading causes tensile stress in the beam flange and also produces tensile stress in the column flange. The same is true for compressive stress in the beam-flange to column-flange connection locations. The exact magnitude of the tensile stress in each flange is than dependent on the beam and column flange proportions. The vertical gravity stress on frame columns is usually not a significant factor because the columns are often sized for drift control under lateral load and not for live and dead load conditions.

Typically, for these connections, a plastic hinge is assumed to develop in the beam adjacent to the column under lateral loading. As a result, yield level stresses are expected to occur in the beam flange and large tensile stresses below yield are expected to occur in the column flange. These loading conditions produce a partially restrained stress condition with a high degree of tri-axial stress. Therefore, brittle fracture is a possible result in the presence of defects and low notch toughness material. Connections with base and weld metal, with adequate notch toughness, and the absence of rejectable notches or discontinuities will develop plastic flow (yielding) in the base metal adjacent to the beam-flange to column-flange weld and exhibit more ductile behavior.

8.6.2 Critical Connection Configurations

The loading condition and state-of-stress at the intersection of a beam and column has been described in the preceding section. Based on this information, various connection configurations can be described that are conducive to brittle fracture before adequate inelastic rotation can be sustained. The order in which they are listed generally, but not conclusively, reflect on ascending ability to deform inelastically.

1. Welded FR connections fabricated with low notch toughness weld metal, left-in-place backing bars and significant workmanship deficiencies.

2. Welded FR connections fabricated with low notch toughness weld metal, but with backing bars removed and with welds reinforced with large overlays of high toughness weld metal (Simon – 1997).

3. Welded FR connections fabricated using specified notch toughness base and weld metal and improved details and workmanship. Improved details include removal of backing bars and run-off tabs and incorporating large reinforcing fillet welds above and below the CJP. Continuous inspection from fit-up to weld completion to ensure strict compliance with an approved WPS.

4. Welded FR connections using reinforced beam-flange to column-flange details that result in plastic hinge formation away from the column face. The connection details and geometry are such that the column face weld stresses remain below the yield stress of the adjacent beam flange. This configuration can be accomplished using cover plates, vertical rib plates and several proprietary systems. In addition, the column-flange face stress levels equivalent to those produced by reinforcing plates can be achieved by the reduced beam section (RBS), or dogbone concept.