Seismic Design Criteria for New Moment-Resisting Steel Frame Construction

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.

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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 (NEHRP Provisions).* 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 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 design 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 this *Seismic Design Criteria for Moment-Resisting Frame Construction* is to provide engineers and building officials with guidance for reliable earthquake-resistant design of new structures incorporating moment-resisting steel frames. It is one of a series publications prepared by the SAC Joint Venture addressing the issue of the seismic performance of moment-resisting steel frame buildings. Companion publications include:

- *Post-earthquake Evaluation and Repair Criteria for Welded Steel Moment-Resisting 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.

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

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

1.2 Intent

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

a) Engineers engaged in the design of new steel frame structures that may be subject to the effects of earthquake ground shaking.

b) Regulators and building departments responsible for control of the design and construction of structures 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.

The fundamental goal of the information presented in these guidelines is to help identify and reduce the risks associated with the earthquake-performance of moment-resisting steel frame structures. The information presented here primarily addresses the issue of beam-to-column connection integrity under the severe inelastic demands that can be produced by building response to strong ground motion. Users are referred to the applicable provisions of the locally prevailing building code for information with regard to other aspects of building construction and earthquake damage control.

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.

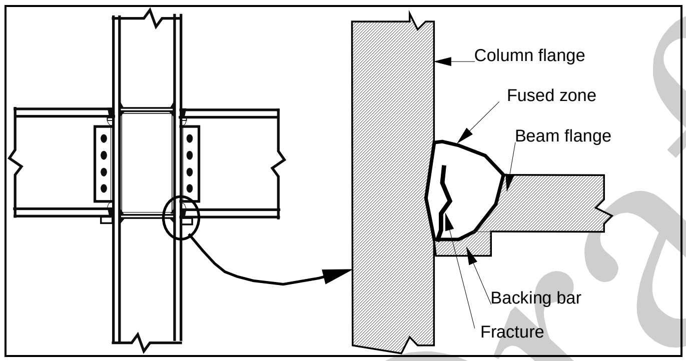
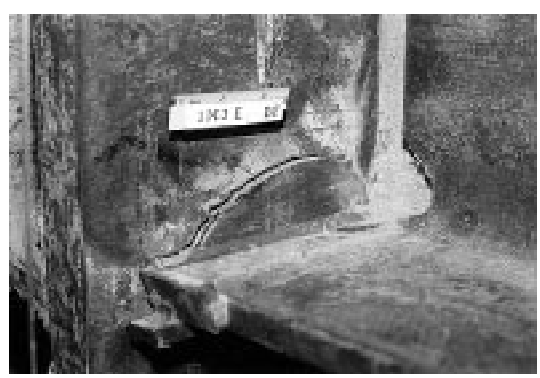


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

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.

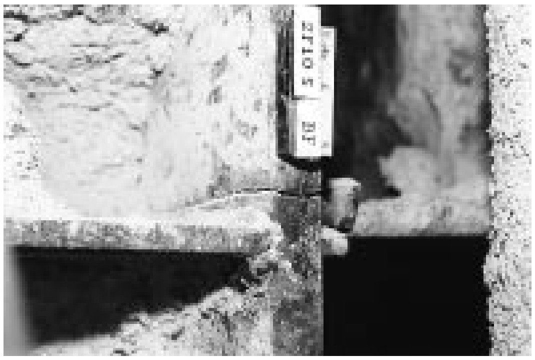
A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). In some cases, these fractures extended into the column web and progressed across the panel zone Figure (1-3b). Investigators have reported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

a. Fracture at Fused Zone

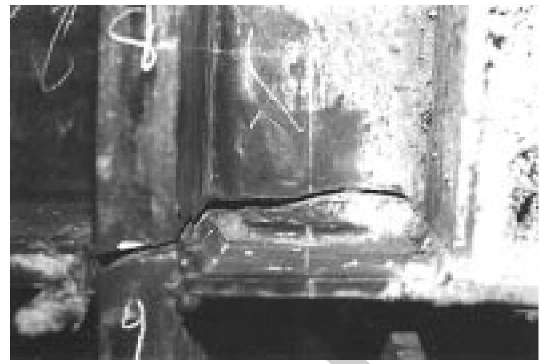


b. Column Flange “Divot” Fracture

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

1.5 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 *NEHRP Provisions*. 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 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 - General Requirements**. This chapter, together with Chapter 3, are intended to supplement the building code requirements for design of moment-resisting steel frame structures. This chapter includes discussion of referenced codes and standards; design performance objectives; selection of structural systems; configuration of structural systems; and analysis of structural frames to obtain response parameters (forces and deflections) used in the code design procedures. It also includes discussion of an alternative, performance-based design approach that can be used at the engineer's option, to design for superior or more reliable performance than is attained using the code based approach. Guidelines for implementation of the performance-based approach are contained in Chapter 4.

- **Chapter 3 - Connection Qualification**. Moment-resisting steel frames can incorporate a number of different types of beam-column connections. Based on research conducted by the SAC Joint Venture, a number of connection details have been pre-qualified for use with different structural systems. This chapter provides information on the limits of this pre-qualification for various types of connections and specific design and detailing recommendations for these pre-qualified connections. It also includes performance data on these connections for use with the performance-based design procedures of Chapter 4. In some cases it may be appropriate to use connection details and designs which are different than the pre-qualified connections contained in this Chapter, or to use one of the pre-qualified connection details outside the range of its pre-qualification. This chapter provides guidelines for project-specific qualification of a connection in such cases. It also includes reference to several proprietary connection types that may be utilized under license agreement with individual patent holders. When proprietary connections are used in a design, qualification data for such connections should be obtained directly from the licenser.

- **Chapter 4 - Performance Evaluation** - This chapter provides a performance evaluation procedure that may be used in the performance-based design process. This procedure allows the probability that a structure will exceed one of several performance states to be estimated, together with a level of confidence on this estimate. The guidelines of this chapter are intended to be optional and apply only to the use of performance-based design approaches.

- **Chapter 5 - 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**.

- **Chapter 6- 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.

2. GENERAL REQUIREMENTS

2.1 Scope

This Chapter presents overall guidelines for the design of moment-resisting steel frames (MRSF) for new buildings and structures. Guidelines are provided for three different MRSF systems, each with different levels of inelastic deformation capability. Included herein are guidelines on applicable codes and standards, recommended performance objectives, system selection, system analysis, frame design, connection design, specifications, quality control and assurance, and other structural systems.

2.2 Applicable Codes and Standards

MRSF systems should, as a minimum, be designed in accordance with the applicable provisions of the prevailing building code and these Guidelines. The Guidelines are specifically written to be compatible with the *1997 NEHRP Provisions* (FEMA 302). Where these Guidelines are different than the prevailing code, these Guidelines should take precedence. The following are the major references:

FEMA 302	NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition
AWS D1.1	Structural Welding Code, 1996 Edition
AISC Seismic	Seismic Provisions for Structural Steel Buildings, April 15, 1997
AISC-LRFD	Load and Resistance Factor Design Specifications for Structural Steel Buildings

Commentary: The 1994 and 1997 Uniform Building Codes, as well as the 1997 AISC Seismic Provisions (AISC), provide design requirements for MRSF structures, including a requirement that connection designs be based on tests. The 1997 NEHRP Recommended Provisions (NEHRP) adopt the 1997 AISC Seismic Provisions by reference as the design provisions for seismic force resisting systems of structural steel. The International Building Code (IBC), scheduled for publication in the year 2000, is expected to be based generally on the NEHRP Provisions, and is expected to have design requirements for steel structures primarily based on the AISC provisions. It is anticipated that by the time the IBC is published many of the recommendations of these guidelines will be incorporated therein as modifications of the AISC or that the AISC will be modified and incorporated by reference. These guidelines are written to be compatible with the AISC and NEHRP Provisions and reference will be made to sections of those documents where appropriate herein.

2.3 Design Performance Objectives

Under the 1997 *NEHRP Provisions*, each building and structure must be assigned to one of three Seismic Use Groups (SUGs). Buildings are assigned to the SUG's based on their intended occupancy and use. Most commercial, residential and industrial structures are assigned to Seismic Use Group I. Buildings occupied by large numbers of persons, or by persons with limited mobility, or house large quantities of potentially hazardous materials are assigned to Seismic Use Group II. Buildings that are essential to post-earthquake disaster response and recovery operations are assigned to Seismic Use Group III. Buildings in SUG II and III are respectively intended to provide better performance, as a class, than buildings in SUG-I. As indicated in the *NEHRP Provisions Commentary*, each SUG is intended to provide the performance indicated in Figure 2-1.

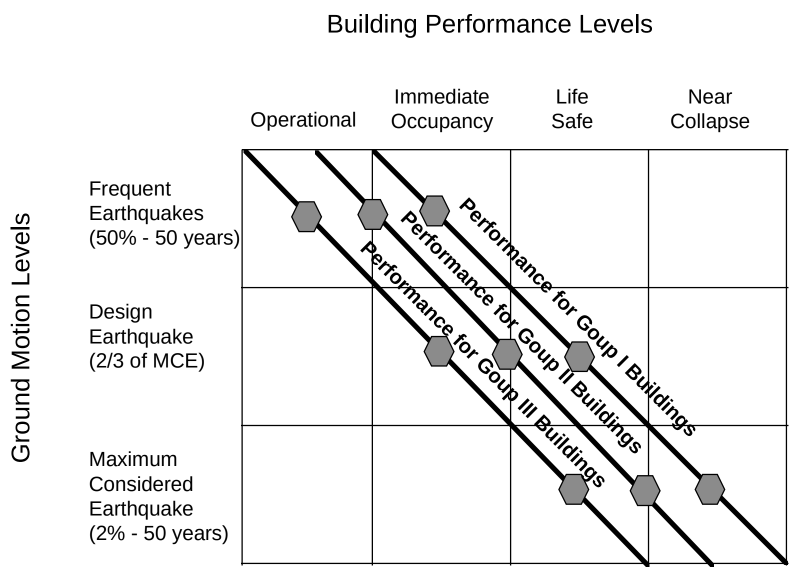


Figure 2-1 - NEHRP Seismic Use Groups and Performance

The NEHRP Provisions attempt to obtain these various performance characteristics through regulation of design force levels, limiting lateral drift values, system selection, and detailing requirements, based on the SUG, the seismicity of the region containing the building site and the effect of site specific geologic conditions. Structures should, as a minimum, be assigned to an appropriate SUG, in accordance with the building code, and be designed in accordance with the applicable requirements.

Although the *NEHRP Provisions Commentary* implies that buildings designed in accordance with the requirements for the various SUG's are capable of providing the alternative performance capabilities indicated in Figure 2-1, the *NEHRP Provisions* do not contain direct methods to evaluate and verify the actual performance capability of structures, nor do they provide a direct means to design for performance characteristics other than those implied for each of the SUGs. It is believed, based on observation of the performance of modern, code conforming construction in recent earthquakes, that the *NEHRP Provisions* provide reasonable reliability with regard to attaining Life Safe performance for SUG-1 structures subjected to rare events, as indicated in Figure 2-1. However, the reliability of the *NEHRP Provisions* with regard to attainment of other performance objectives for SUG-1 structures, or for reliably attaining any of the performance objectives for the other SUGs seems less certain and has never been quantified or verified.

Chapters 2 and 3 of these Guidelines, present code-based design recommendations for MRSF structures. All buildings should, as a minimum, be designed in accordance with these recommendations. For buildings in which it is desired to attain other performance than implied by the code, or for which it is desired to have greater confidence that the building will actually be capable of attaining the desired performance, the Guidelines of Chapters 4 and 5 may be applied.

Commentary: The NEHRP provisions include three types of moment resisting steel frames (MRSF's) all of which are incorporated in these guidelines. The three types are: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). These systems are described in more detail in the section on system selection. In the NEHRP provisions, a unique R value is assigned to each of these systems, as are height limitations and other restrictions on use. Regardless of the system selected, the NEHRP provisions imply that structures designed to meet the requirements therein will be capable of meeting the Collapse Prevention performance level for a Maximum Considered Earthquake (MCE) ground motion level and will provide Life Safe performance for the Design Basis Earthquake (DBE) ground motion that has a severity of 2/3 of the severity of the MCE ground motion. This 2/3 factor is based on the assumption by the provisions that the Life Safety performance on which earlier editions of the provisions were based inherently provided a minimum margin of 1.5 against collapse. Except for sites located within a few kilometers of known active faults, the MCE ground motion is represented by a ground shaking response that has a 2% probability of exceedance in 50 years (2500 year mean return period). For sites that are close to known active faults, the MCE ground motion is taken either as this 2%/50 year spectrum, or as a spectrum that is 150% of that determined from a median estimate of the ground motion resulting from a characteristic event on a known active fault, whichever is less. This is compatible with the approach taken by the 1997 UBC for the definition of design ground motion on sites near major active faults.

The UBC and NEHRP provisions both define classes of structures for which performance superior to that described above is mandated. Additionally, individual building owners may desire a higher level of performance than that described. The UBC attempts to achieve higher performance through specification of an occupancy importance factor which increases the design force level; the NEHRP provisions attempt to improve performance through use of both an occupancy importance factor and of more *restrictive drift limits. The combination of increased design forces and more restrictive drift limitations leads to substantially greater strength in systems such as SMF's, the design of which is governed by drift.*

The NEHRP R factors, drift limits, and height limitations, as well as the inelastic rotation capability requirements corresponding to the R value for each moment frame type (SMF, IMF, or OMF), are based more on historical precedent and judgment than they are on analytical demonstration. In the research program on which these guidelines are based, an extensive series of nonlinear analytical investigations has been conducted to determine the drift demands on structures designed in accordance with the current code when subjected to different ground motions. The results of these investigations have led to these Guidelines recommending modifications to some of the NEHRP and AISC design provisions where there was concern that the intended performance would not be achieved.

It should be recognized that application of the modifications in these Guidelines, while considered necessary to achieve the indicated performance for moment frames, may make such systems perform better than some other systems which may not have had as significant an analytical base for their provisions. In other words, some other systems included in the NEHRP provisions, both of steel and of other materials, have provisions which may provide a lower level of assurance that the resulting structures will meet the intended performance level. It is also worthy of note that the three classes of steel MRSF systems contained in the NEHRP Provisions are themselves not capable of providing uniform performance. OMF structures will typically be stronger than either IMF or SMF systems, but can have much poorer inelastic response characteristics. The result of this is that OMF structures should be able to resist the onset of damage at somewhat stronger levels of ground shaking than is the case for IMF or SMF structures. However, as ground motion intensity increases beyond the damage threshold for each of these structural types, it would be anticipated that OMF structures would present a much greater risk of collapse than would IMF structures, which in turn, would present a more significant risk of collapse then SMF structures. For these reasons, the NEHRP Provisions place limitations on the applicability of these various structural systems depending on the height of a structure, and the seismic hazard at the site.

The reader is referred to Chapter 4 for more detailed discussion of recommended performance levels and their implications.

2.4 System Selection

2.4.1 Configuration and Load Path

Every structure should be provided with a complete load path, capable of transmitting inertial forces from the foundations to the locations of mass throughout the structure. For moment-resisting frame structures, the load path includes the foundations, the moment-resisting frames, floor and roof diaphragms and the various collector elements that interconnect these system components.

To the extent possible, the structural system should have a regular configuration without significant discontinuities in stiffness or strength and with the rigidity of the structural system distributed uniformly around the center of mass.

2.4.2 Selection of Moment Frame Type

The NEHRP Provisions define three types of MRSFs: Special Moment Frames (SMF), Intermediate Moment Frames (IMF), and Ordinary Moment Frames (OMF). Detailing and configuration requirements are specified for each of these three frame types to provide different levels of reliable ductility (inelastic rotation capability) and consequent drift angle capacity, varying from highest in SMF's to lowest in OMF's. The selection of moment frame type should be governed by the prevailing code and by the project conditions. Consideration should be given to using the more ductile systems.

Commentary: Although the NEHRP provisions, as modified by these guidelines, are intended to provide the same level of seismic performance for all three of the frame types given the conformance of all actual conditions to the limits of the assumed conditions, it is recognized that variations will occur in ground motions as well as in other conditions of design, and it is judged that higher ductility (higher inelastic rotation capability) is likely to provide a greater margin of safety if conditions beyond those anticipated should be experienced. For this reason, the NEHRP Provisions place limitations on the height and or relative importance or seismic exposure (Seismic Design Category) for structures which employ OMF's and IMF's as compared to those with SMF's. Because of the aforementioned higher margin, it is recommended that designers and owners consider the cost versus benefit of using systems with higher relative ductility whenever seismic forces govern the design.

The NEHRP Provisions and AISC Seismic use inelastic rotation demand as the primary design parameter for judging frame and connection performance, as did FEMA-267. SAC has decided to use interstory drift demand as the design parameter, because this parameter is analytically stable, will provide good correlation with performance, and is relatively simple to predict using common analysis methods.

2.4.3 Connection Type

Either Fully Restrained (FR) or Partially Restrained (PR) connections are permitted for all three MRSF systems in the *1997 NEHRP Provisions*. The provisions require that the connections meet minimum strength requirements and be demonstrated by test to be capable of providing minimum levels of rotational capacity. The provisions also require that the additional drift due to connection flexibility (for PR connections) be accounted for in the design, including P-Delta effects. In Chapter 3, design procedures are provided for several types of pre-qualified FR and PR connections, together with limitations on the applicability of the pre-qualification. Guidelines for analysis of frames comprising these connections are given in Chapter 4. Designs employing connections that are not pre-qualified under these Guidelines, should be demonstrated by test to be capable of providing the minimum levels of drift angle capacity required for the system being used.

Commentary: In many areas of the United States, modern era moment frames have been designed as type FR almost exclusively. On the other hand, in most areas, there are some older mid to highrise buildings designed with what would now be referred to as PR connections, and some engineers have a current practice of using PR connections in low to moderate seismic zones. Accordingly, research was undertaken as part of this project to permit development of rational guidelines for the design and analysis of such systems and to provide connection design guidelines which do not require project connection testing.

2.4.4 Redundancy

The *1997 NEHRP Provisions* include a redundancy factor, ρ, with values between 1.0 and 1.5, which is applied as a load factor on calculated earthquake forces for structures categorized as Seismic Design Category (SDC) D, E, or F. Less redundant systems (frames with fewer participating beams and columns) will have higher values of the redundancy factor and therefore will require higher design forces to compensate for their lack of redundancy. Additionally, since the design of MRSFs is typically governed by considerations of drift control, rather than strength, MRSFs are required to be configured to qualify for a redundancy factor 1.25 or less (or 1.1 for SDC's E and F).

Designers should as a minimum, provide the level of redundancy required by the code. Whenever it is practical to do so, as many moment resisting connections as is reasonable should be incorporated into the moment frame.

Commentary: Redundancy has obvious advantages for structures subjected to random brittle fractures or failures resulting from occasional poor construction quality or an imbalance in material strengths of the various connected elements. If brittle connection fractures occur, it can be assumed that fractures, will not occur in all connections at the same time. Thus, more ductile connections will be available to dissipate earthquake energy, after a given number of fractures occur, in more redundant buildings. Cornell and Luco (Ref.) have done a limited study on this issue, which was not very conclusive. The effect of redundancy was not very strong, even when a rotation capacity benefit was given to the connections of the shallower beams of the more redundant structure.

Another important advantage of providing redundant framing systems is that the use of a larger number of frames to resist lateral forces often permits the size of the framing elements to be reduced. Laboratory research has shown that connection ductility generally decreases as the size of the framing increases.

2.5 Structural Materials

2.5.1 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.

2.5.2 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

$$F_{ye} = R_y F_y \qquad (2\text{-}1)$$

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.

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 $1.1F_y$. 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 variance of the product can be estimated as the sum of the squares of the variance of each parameter. The estimated means and variances and the mean +/-1 and 2 times the variance are shown in the table below:

Parameter	*Mean*	*Variance*	*Mean -1 Variance*	*Mean+ 1 Variance*	*Mean -2 Variance*	*Mean+ 2 Variance*
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 Variance 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 for the associated connections.

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.

2.6 Structural Analysis

An analysis should be performed for each structure to determine the distribution of forces and deformations under code specified ground motion and/or loading criteria. The analysis should conform, as a minimum, to the code specified criteria for equivalent lateral force (ELF), Modal Response Spectrum (MRS) or Response-history analysis, as applicable.

Chapter 4 provides guidance on analysis methods applicable to performance evaluation of WSMF structures.

Commentary: Seismic design forces for low to mid-rise buildings without major irregularities have traditionally been determined primarily by using the simple "equivalent static" method prescribed by the codes. Such methods are incorporated in the 1997 NEHRP Provisions and are permitted to be used for structures designated as regular up to 240 feet in height. Buildings which are over 5 stories or 65 feet in height and have certain vertical irregularities, and all buildings over 240 feet in height, require use of dynamic (modal or time history) analysis. The use of elastic or inelastic response history, or of non-linear static analysis is also permitted, though few guidelines are provided in the code for how to apply such analysis. Projects incorporating non-linear response-history analysis should be conducted in accordance with the performance evaluation provisions of Chapter 4.

2.7 Mathematical Modeling

2.7.1 Basic assumptions

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 design 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 framing in buildings with flexible diaphragms may be individually modeled, analyzed and designed as two-dimensional assemblies of components and elements, or a three-dimensional model may be used with the diaphragms modeled as flexible elements.

Explicit modeling of a connection is required for nonlinear procedures if the connection is weaker than the connected components, and/or the flexibility of the connection results in a significant increase in the relative deformation between connected components.

2.7.2 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 3 for pre-qualified connections. Guidelines for modeling structural components are given in Chapter 4.

Building irregularities are discussed in FEMA 302. Such classification should be based on the plan and vertical configuration of the framing system, using a mathematical model that considers relevant structural members.

2.7.3 Horizontal torsion

The effects of horizontal torsion must be considered. The total torsional moment at a given floor level includes 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.

Commentary: Actual torsion that is not apparent in an evaluation of the center of rigidity and center of mass in an elastic stiffness evaluation can develop during nonlinear response of the structure if yielding develops in an unsymmetrical manner in the structure. For example if the frames on the east and west sides of a structure have similar elastic stiffness the structure may not have significant torsion during elastic response. However, if the frame on the east side of the structure yields significantly sooner than the framing on the west side, then inelastic torsion will develop. Although the development of such inelastic torsion can be quite problematic, the NEHRP Provisions do not address this phenomena. Designers can avoid structures with severe inelastic torsion potential by providing framing layouts that are symmetrical about the center of mass, both with regard to stiffness and strength.

In buildings with diaphragms that are not flexible, 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%. Accidental torsion 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.

For the linear analysis of buildings with rigid diaphragms, when the ratio $\delta_{max}/\delta_{avg}$ due to total torsional moment exceeds 1.2, the effect of accidental torsion should be amplified by a factor, A_x:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \tag{2-2}$$

where:

δ_{max} = Maximum displacement at any point of the diaphragm at level x

δ_{avg} = Average of displacements at the extreme points of the diaphragm at level x

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 Linear Static Procedure (LSP) and the Linear Dynamic Procedure (LDP), the design forces and displacements should be increased by multiplying by the maximum value of η calculated for the building.

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

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

2.7.4 Foundation modeling

Foundations should be modeled considering the relative stiffness of the foundation systems and the rigidity of attachment of the structure to the foundation. 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.

Commentary: Most moment-resisting steel frames can be adequately modeled by assuming that the foundation provides rigid support for vertical loads. However, the flexibility of foundation systems (and the attachment of columns to those systems) can significantly alter the flexural stiffness at the base of the frame.

2.7.5 Diaphragms

Floor diaphragms transfer earthquake-induced inertial forces to vertical elements of the seismic framing system. Roof diaphragms are considered to be floor diaphragms. Connections between floor diaphragms and vertical seismic framing elements must have sufficient strength to transfer the maximum calculated diaphragm shear forces to the vertical framing elements. Requirements for design and detailing of diaphragm components are given in the *NEHRP Provisions*.

Floor diaphragms should be classified as either flexible, stiff, or rigid in accordance with the *NEHRP Provisions*. 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.

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.

2.7.6 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 vertical 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 to consider these effects when performing a nonlinear 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.

2.7.6.1 Static P-Δ Effects

The structure should be investigated to ensure that lateral drifts induced by earthquake response do not result in a condition of instability under gravity loads. At each story, the quantity θ_i should be calculated for each direction of response, as follows:

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} \tag{2–3}$$

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 and the design modified to reduce the computed lateral deflections in the story.

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

Commentary: For a bilinear SDOF system with mass m and height h the dimensionless parameter $\theta = mg/(Kh)$ can be used as indicator of the severity of P-Δ effects. The elastic stiffness K is reduced to $(1-\theta)K$, and the post-elastic stiffness $\alpha' K$ is reduced to $(\alpha' - \theta)K$. In this formulation α' is the strain hardening ratio of the system without P-delta effect, and $\alpha' - \theta$ 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.

2.7.6.2 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 $\alpha K = -0.03K$. 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 $\alpha = -0.03$ over displacement for $\alpha = 0.0$) 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.

2.7.7 Multidirectional excitation effects

Buildings should be designed for seismic forces in any horizontal direction. For regular buildings, seismic displacements and forces may be assumed to act nonconcurrently in the direction of each principal axis of a building. For buildings with plan irregularity and buildings in which one or more components form part of two or more intersecting elements, multidirectional excitation effects should be considered. Multidirectional effects on components should include both torsional and translational effects.

The requirement that multidirectional (orthogonal) excitation effects be considered may be satisfied by designing elements or components for the forces and deformations associated with 100% of the seismic displacements in one horizontal direction plus the forces associated with 30% of the seismic displacements in the perpendicular horizontal direction. Alternatively, it is acceptable to use SRSS to combine multidirectional effects where appropriate.

The effects of vertical excitation on horizontal cantilevers and prestressed elements should be considered by static or dynamic response methods. Vertical earthquake should be considered by static or dynamic response methods. Vertical earthquake shaking may be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum unless alternative vertical response spectra are developed using site-specific analysis.

2.7.8 Verification of analysis assumptions

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

Where moments in horizontally-spanning primary components, due to the gravity loads, exceed 50% of the expected moment strength at any location, the possibility for inelastic flexural action at locations other than components ends should be specifically investigated by comparing flexural actions with expected component strengths, 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.

2.8 Frame Design

The following provisions supplement the parallel provisions contained in the building code.

2.8.1 Strength of Beams and Columns

The AISC Seismic Provisions (Equation (9-3)) includes relationships which must be satisfied to provide for a nominal condition of columns being stronger than the beams connected to them (for SMF's and IMF's). The AISC equation uses the expected strength of the beams as described in Section 2.5.2, multiplied by a strain-hardening factor of 1.1 and compares it to the column strength as calculated using the nominal properties. The FEMA 267 relationship is formulated somewhat differently, but gives similar results. The relationship in the AISC Seismic Provisions should be satisfied, as a minimum.

Commentary: Non linear analyses have clearly shown that use of the provisions described above will not prevent hinging of columns. This is because the point of inflection in the column may move far from the assumed location at the column mid-height once inelastic hinging occurs, and because of global bending induced by the deflected shape of the building, of which the column is a part. The global bending problem will become more important if heavy (stiff) columns are used. If a story has a large drift compared to the story above, the moment at the top of the column will increase. If the drift is large compared to the story below, the moment at the bottom of the column will increase. If the drift is large compared to both of the adjacent stories (soft story condition), then a column hinge mechanism is possible.

Except for the column hinge mechanism case, column hinging is not a big problem, provided that the columns are designed as compact sections and are properly braced and axial loads are not too high. It is well understood that a column hinge will form at the base of columns which are continuous into a basement, or which are rigidly attached to a stiff and strong foundation.

In summary, column hinging is not judged to be a problem unless one of the following conditions occur:

1. *Noncompact sections are used, or inadequate lateral bracing at the hinge point is provided;*

2. The moment gradient within a story is very small (close to uniform moment), in which case member P-delta may become a problem;

3. Plastic hinges occur at the top and bottom of all columns in a story, in which case, a mechanism forms.

4. Axial loads in the column exceed about 50% of the column's buckling strength.

Based on the extensive analytical studies of prototype buildings, neither condition 2 or condition 3 above is expected to occur, except in very rare cases.

Review of Section 2.5.2 indicates that, even on a local level, avoiding hinges in all columns would require that the formula relationship be greater than about 1.5 (using mean +2 Variance and a strain-hardening factor of 1.1 on the beam strength and mean -2 Variance on the column strength). If it was desired to eliminate all column hinging, the factor would have to be even higher because of the conditions noted. It is not judged that such high factors are justified, but it may be desirable to increase the factor above 1.0. The New Zealand code (Ref.) uses higher factors.

Large axial loads reduce the ductility of column hinges. Consideration should be given to applying larger factors for columns with relatively higher axial loads.

The reader is referred to the State of the Art Report on System Performance (Ref.) for more information on this issue.

2.8.2 Panel Zone Strength

Panel zone strength is an issue affecting both global and connection performance. As a minimum, the requirements of the AISC Seismic Provisions should be met. Where relative panel zone strength affects the performance of a prequalified connection, limits on the strength are provided in the section where the connection design guidelines are given in Chapter 3. For non prequalified connections, the relative panel zone strength should be based on that of the tested connection.

Commentary: The 1988 UBC changed the panel zone strength requirements to recognize the contribution of thick flanges. It was recognized that the result would be some yielding of panel zones prior to the anticipated plastic hinging of the beams, but it was felt that strain hardening of the very ductile panel zones would ultimately lead to the beam yielding, and it was felt that some panel zone yielding would be desirable in increasing the ductility of the connection assembly without significant loss of strength of the frame. For the most part this concept has been supported by the current research, with the exception that some studies have shown that the panel zone yielding can lead to "kinking' of the column flange at the point of the beam flange weld, and that this condition can contribute to premature fracture of the weld. On the other hand, studies by Goel and

Stodajinovic (Ref.) of unreinforced connections have shown that panel zone yielding is responsible for about 50% of the plastic rotation evidenced in their tests.

Based on the evidence of current research, it is clear that connection performance can be affected either positively or negatively by panel zone strength. For this reason, the prequalified connection designs include a requirement for panel zone strength relative to beam strength. For some connection types strong panel zones are required, while for others an upper limit on panel zone strength is given.

With respect to the global affect of panel zone strength on frame performance, it is judged that the AISC requirements are adequate to prevent strength problems in the columns, and nonlinear analyses (See SOA report on System Performance, Ref.) have shown that weak panel zones have little effect on drift.

2.8.3 Connection Strength and Degradation

The AISC Seismic Provisions require that connection tests demonstrate “a flexural strength, determined at the column face, that is at least equal to the nominal plastic moment capacity of the beam, M_p, at the required inelastic rotation” except for conditions of Reduced Beam Section connections, connections which lead to flange buckling and PR connections for which somewhat reduced strengths are permitted.

Commentary: Since it is recognized that, when a frame reaches a state of declining strength with increased deformation, collapse can be expected, it is clearly undesirable for individual connections to reach such a state. Although declining strength of some individual connections does not indicate declining strength for the frame as a whole, clearly, if a sufficient number of connections reach such a condition, the frame as a whole will follow.

2.8.4 P-Delta Effects

Section 2.7.6 above gives analytical procedures for checking P-Delta effects on moment frame structures. These procedures are very complex for use in normal design. It is hoped that we can develop triggers and simple procedures for normal use, reserving the more complex analysis for use in special structures in near-fault zones. Such procedures have not yet been developed

2.8.5 Section Compactness Requirements

The AISC Seismic provisions provide section compactness requirements for beams used in moment frames, and for columns which may be subjected to hinging. Designers should follow the section compactness requirements of AISC as a minimum for purposes of selecting beam members for frame design. For columns, designers should select members which are compact, unless it can be shown by nonlinear analysis that the columns will not yield under the forces of the design earthquake. Alternatively, a minimum Column-Beam moment ratio of 2.0 should be used, rather than the 1.25 required by AISC. For guidelines for beam section compactness as it relates to connection design, see section 3.3.1.1.

Commentary: The effect of beam flange b/t as it relates to connection performance is discussed in section 3.3.1.1 Beam Flange Stability. The effect of beam section compactness on overall frame performance is directly related to how it affects strength degradation of individual connections in the frame. Flange local buckling and lateral torsional buckling are sources of strength degradation. The effect of strength degradation of individual beam column connection assemblies on the stability of the overall frame is discussed in the commentary under section 2.8.3 above.

The importance of column compactness for columns which may undergo plastic rotation is discussed in the commentary to section 2.8.1 above. That discussion clearly shows that the currently used ratio of 1.25, for defining columns which will not have to hinge, is far from being sufficient to prevent plastic hinging of columns, and the consequence of plastic hinging of non compact columns is judged to be highly undesirable.

2.9 Connection Design

The *NEHRP Provisions* require the use of tested connections to meet the requirements for design of SMF's, IMF's and also for OMF's, except where certain strength requirements are met based on calculations. Chapter 3 provides guidelines for the design of several types of pre-qualified Fully Restrained (FR) and Partially Restrained (PR) connections which are acceptable for use in MRSF systems, within the limitations expressed in the Chapter. It is intended that the testing of these connection types, performed as part of this program and prior to it, coupled with the detailed design guidelines herein, will satisfy the requirements of the *NEHRP Provisions* for these connection types without the need for additional specific testing. The connections covered include the following:

Welded Connections:

1. Welded Unreinforced Flanges (WURF);
2. Welded Cover Plated Flanges (WCPF);
3. Welded Flange Plates (WFP);
4. Reduced Beam Flanges (RBS, or Dog Bone);
5. Welded Single Haunch (WSH);
6. Welded Double Haunch (WDH).

Bolted Connections:

1. Bolted End Plate (BEP);
2. Welded Flange Plates with Bolted Beam (WFP/BB)

Partially Restrained Connections:

1. Double Split Tee (DST);
2. Single Tee Composite (STC);
3. Single Angle Composite (SLC);
4. Shear Tab Composite (SC).

For each connection type a complete set of design guidelines is presented along with the limiting inelastic interstory drift permitted for the connection type.. Knowing what interstory drift is required for a given frame type, the designer should select a suitable connection to meet the expected demands, then follow the design guidelines to complete the design. Connections contained in Chapter 3 may be used in applications outside the indicated range of pre-qualification provided that a project-specific qualification program is followed, as indicated in Chapter 3. Connection types not pre-qualified under the Guidelines of Chapter 3 may also be used, subject to the project specific qualification procedures.

Note that as of the time of publication of these 50% draft guidelines, insufficient connection testing has been completed to allow design guidelines for many of these connection types to be presented. A representative sampling of design guidelines, for certain connection types are included. Remaining connection types will be shown in later editions of these guidelines.

Commentary: Since the publication of the Interim Guidelines(FEMA 267) in August of 1995, a tremendous amount of research has been performed on steel moment frame connections. All of the connection types listed have been tested, some of them very extensively, and in addition, a great deal of subassembly testing has been performed to answer questions generic to the design of most or all of the connection types. All of the information derived from the tests has been incorporated into the design guidelines included in Chapter 3. The reader is referred to the commentary in that chapter for complete references and detailed discussion.

2.10 Specifications

The AWS, as well as FEMA, have developed significant modifications to recommendations for welding of steel moment resisting frames. Additionally, there have been developed several modifications to traditional steel specifications regarding material strength, material testing , and material shaping and repair. Modifications to quality control and assurance specifications have also evolved. Chapter 6 provides recommendations for construction specifications for projects incorporating MRSF construction.

Commentary: Chapter 6 includes guidelines for modifications to traditional structural steel specifications which reflect the knowledge gained from the FEMA/ SAC program. These guidelines incorporate those provisions demonstrated by research and/or judged by the writers to be essential to achieving the required or expected performance of the moment frame system and its connections.

2.11 Quality Control and Assurance

FEMA XXX - Quality Assurance Guidelines for Moment Resisting Steel Frame construction provides complete guidelines and commentary for Quality Control and Quality Assurance. The designer should utilize those guidelines to assure the proper selection and handling of materials and shop and field fabrication of moment frame connections.

Commentary: The reader is referred to the companion publication for a complete discussion of quality control recommendations and the reasons for them. The importance of quality control and quality assurance to the achievement of the intended performance can not be overemphasized.

2.12 Other Structural Systems

2.12.1 Column Splices

The AISC Seismic Provisions provide requirements for column splices for moment frame columns in Section 8.3 of that document. The requirements do not permit splices made with fillet welds or partial penetration groove welds to be located "within 4 feet nor one-half the column clear height of the beam-to-column connections, whichever is less." This requirement is not considered to be sufficiently conservative. The designer should use complete penetration groove welds for all moment frame column splices, except where inelastic analysis has been used to demonstrate that hinging will not occur in the region of the splice, and that the highest calculated moment at the location of the splice, together with any computed column tensile loads, can be resisted by the welds considering an appropriate factor for the stress concentration inherent in the welded joint type. For either CJP or Partial Penetration welds (if justified by analysis), weld metal with a minimum rated toughness of 20 ft-pounds at -20^{O}F should be used and runoff tabs should be removed. Backing need not be removed from column splice welds.

Commentary: The discussion in the commentary to section 2.8.1 above describes clearly why hinging can occur in the column in locations far from the beam-to-column connection. The AISC provisions attempt to avoid the concern about column moments at splices by defining the location of the column splices at locations near theoretical *inflection points. The provisions then are intended to provide adequate splices considering the worst case of axial loads which may occur. For the reasons described, the AISC provisions are not considered to be sufficiently conservative in this regard.*

Because bending and axial stresses at column splice welds may be high, it is recommended that notch tough weld wire be used for these splices and that runout tabs be removed. Removal of backing is not judged to be necessary because the configuration of backing for column-to-column flange welds is not conducive to crack formation, as it is for the right angle condition of beam-to-column flange joints.

2.12.2 Column Bases

Column bases can be of several different types, as follows:.

1. The column may continue into a basement, crawl space, or grade beam, in such a way that the column's fixity is assured without the need for a rigid base plate connection;

2. Large columns may be provided at the bottom level to limit the drift, and a "pinned base" may be utilized;

3. A connection which provides partial fixity may be provided, so that the column base is fixed up to some column moment, but the base itself, in some way, yields before the column hinges;

4. A heavy base plate assembly may be provided which is strong enough to yield the column.

In all of these cases, the designer should consider the base connection as similar to a beam-to-column connection and apply similar principles of design and detailing.

For the first case above, the designer should recognize that hinging will occur in the column, just above the first floor. The horizontal shear to be resisted at the ends of the column in the basement level should be calculated considering the probable overstrength of the framing.

For the "pinned base", the designer should ensure that the required shear capacity of the base can be maintained up to the maximum rotation which may occur.

In designing a base with partial fixity , the designer should consider the principles used in the design of PR type connections. This type of base may rely on bending of the base plate (similar to an end plate connection), bending of angles or tees, or yielding of anchor bolts. In the latter case, it is necessary to provide bolts or rods with appropriate elongation capacity to permit the required rotation and sufficient unrestrained length for the yielding to occur.

For the fully fixed base, the designer should employ the same guidelines as given for the rigid end plate connections. Such connections may employ thick base plates, haunches, cover plates, or other strengthening as required to develop the column hinge. Where haunched type connections are used, it must be recognized that the hinging will occur above the haunch, and appropriate consideration should be given to the stability of the hinge.

Commentary: It is well recognized that achievement of a mechanism in a moment frame requires a hinge at, or near to, the base of the column. The column base detail must accommodate the required hinging rotations while maintaining the strength required to provide the mechanism envisioned by the designer. These conditions are similar to the requirements for beam-to-column connections, as described.

(Note: in the Interim Guidelines, the discussion in section 7.10 provides guidelines for other structural systems which have moment resisting connections which perform similar functions to those employed in moment resisting frames. These systems include EBF's, Dual Systems, Welded Base Plates, Vierendeel Trusses, Tube Frame Systems, Collectors, Ties and Diaphragm Chords, Welded Column Splices, and Built-up Moment Frame Members. It is not clear at this time that we will have anything to add to these discussions, except for discussion on column splices and base plates as given.

3. CONNECTION QUALIFICATION

3.1 Scope

This section provides guidelines for connection qualification and design for new buildings. Included herein are guidelines for design of joints and conditions which are generic to most connection types, and guidelines for specific details of connections intended to be pre-qualified for use in OMF's, IMF's and SMF's. Also included are guidelines for qualification of connections which have not been pre-qualified or which are proposed for use outside the limits of their pre-qualification. Each of the pre-qualified connections has specific conditions for which it should be considered for use, including member size ranges and required drift angle capacity (and consequently moment frame type).

Commentary: The 1988 Uniform Building Code introduced a single pre-qualified ("prescriptive") moment- connection design for seismic applications, 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 1970's. (Ref. 12-1). The UBC pre-qualified connection was subsequently adopted into the 1992 AISC Seismic Provisions and then into model codes. After the 1994 Northridge earthquake demonstrated the ineffectiveness of the pre-qualified connection as it was being used in modern practice, enforcement agencies adopted emergency requirements for qualification of all moment frame connections by testing. The SAC Interim Guidelines and Interim Guidelines Advisory No. 1 (FEMA 267 and 267A, August 1995 and March 1997, respectively) continued and reinforced the recommendation for using only tested connections, while providing extensive guidance on how and under what conditions such testing should be required and how results might be interpolated or extrapolated. The 1997 NEHRP, AISC, and UBC, require that connections for all three types of moment frames (SMF, IMF, OMF) be qualified by test. Connections for OMF's are permitted to be designed based on calculations alone, if certain strength and detailing conditions are met.

It is the intent of these Guidelines to return the design of MRSF structures to the condition of being a straightforward and relatively simple task, while providing the reliability which was previously incorrectly assumed to exist. For the majority of structures and conditions of use, it is intended that the designer will be able to select, design, and detail moment frame connections appropriate for the intended structure by using the guidelines herein. For connection types not included herein, or which cannot be extrapolated from types herein, it is still intended that *qualification testing be used, and guidelines are provided for performance and acceptance of such testing.*

3.2 Basic Design Approach

This section provides guidelines on basic principles of connection design, including selection of an appropriate connection type, 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 types.

3.2.1 Frame Configuration

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 3-1 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. 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 PR connections; within the column panel zone, or within the column.

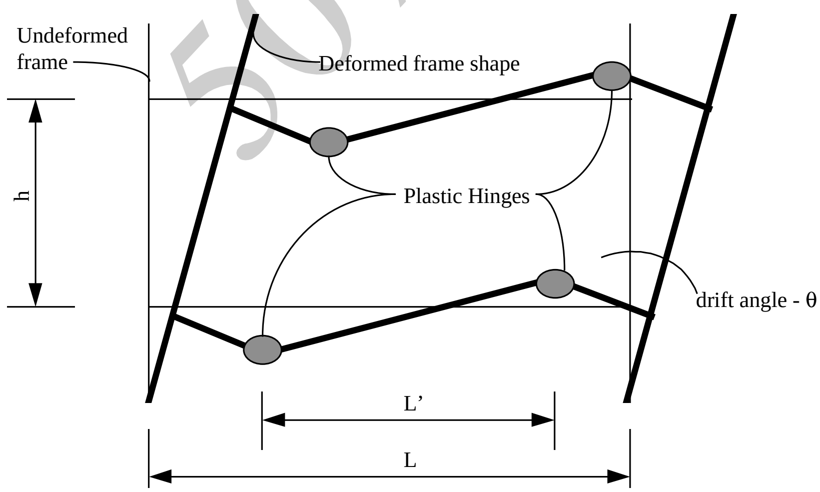


Figure 3-1 - Inelastic Behavior of Frames with Hinges in Beam Span

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 yielding 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 may result in the formation of mechanisms with relatively few elements participating, so called "story mechanisms," and consequently little energy dissipation occurring. In addition, such mechanisms may also result in local damage to the columns which are critical gravity load bearing elements.

The pre-qualified connection contained in the building code prior to the Northridge Earthquake was presumed to result in a plastic behavior that consisted of 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 may result in very large secondary stresses on the beam flange to column flange joint, a condition which for certain types of connections, can contribute to brittle failure. If the plastic hinge forms in the beam, at the face of the column, this can result in large inelastic strain demands on the weld metal and surrounding heat affected zones. These conditions can also lead to brittle joint failure even when particular care is taken in fabricating the connection.

SMF 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 incorporate a strong column-weak beam design to avoid the development of column hinging and story collapse mechanisms, and type FR beam-column connections should be configured to force the inelastic action (plastic hinge) away from the column face, where performance is less dependent on the workmanship of the welded joint. This can be done either by local reinforcement of the connection, or locally reducing 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 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 for several reasons. First, severe connection degradation can result in loss of gravity load carrying capacity of the framing at the connection and the potential development of local collapse. From a global perspective, the occurrence of many connection fractures results in a substantial reduction in the lateral-force-resisting strength and stiffness 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 experienced significant plastic rotation of such hinges may exhibit large buckling and yielding deformation, as well as concurrent localized damage to floor slabs and other supported elements. In severe cases, this damage must be repaired. The cost and difficulty of such repairs could be comparable to the costs incurred in repairing fracture damage of the type 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, that 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. Framing systems incorporating partially restrained connections may also be quite effective in resisting large earthquake induced deformation with limited damage.

OMF and IMF structures are intended to have less inelastic response capability than are SMF structures. For OMF systems, type FR connections that permit development of plastic hinges at locations other than within the beam span, e.g. in the panel zone, in the column, etc., are permitted.

Type PR connections are configured to be able to form plastic hinges through yielding of the connection elements themselves. The plastic moment capacity of these connections is typically a fraction of that of the connected framing elements, encouraging the inelastic behavior to occur within the connection. These connections must be configured to ensure that inelastic behavior occurs through ductile yielding of elements, rather than brittle failure, such as shearing or elongation of bolts, or tensile fractures through weak net-sections of connection elements. Frames employing properly designed PR connections can be capable of extensive inelastic response, with plastic hinges forming within the connection, adjacent to the face of the column. Because such connections are weaker and less stiff, systems using PR connections typically incorporate more of the framing members into the moment frame system than do frames using FR connections.

3.2.2 Inter-story Drift Capacity

The Inter-story Drift Capacity (IDC) of connection assemblies should reflect realistic estimates of the total (elastic and plastic) inter-story drift likely to be induced in the frame by earthquake ground shaking, considering the geometric configuration, strength, stiffness and hysteretic energy dissipation characteristics of the frame. For a specific frame configuration, and design ground motion, this can best be determined by using nonlinear response-history analyses, although nonlinear static methods can provide reasonable approximations. Current building codes do not require the application of such nonlinear analyses. For frames of typical configuration, conforming in all respects to the applicable code requirements, and for ground shaking of the levels anticipated by the building code, the default values contained in Table 3-1 may be used to be representative of the factored inter-story drift demands, ($\phi\theta$) experienced under design ground shaking levels.

Table 3-1 - Default Total Drift Capacities for Various Structural Systems

System	Factored Inter-story Drift Capacity (Radians), $\phi\theta_I$
OMF	0.02
IMF	0.03
SMF	0.04

3.2.3 Connection Configuration

A connection configuration should be selected that is compatible with the selected structural system, the anticipated inter-story drift demands and the sizes of the framing elements. Section 3.4 presents data on a series of pre-qualified connections, from which an appropriate connection type may be selected. Alternatively, if project specific connection qualification is to be performed, a connection of any configuration that provides the appropriate inter-story drift capacity may be selected.

3.2.4 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 configurations that are qualified on a project-specific basis, the location of expected plastic hinge formation, s_h,, as indicated in Figure 3-2 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. For frames in which gravity loading produces significant flexural stresses in the members, locations of plastic hinge formation should be determined based on methods of plastic analysis.

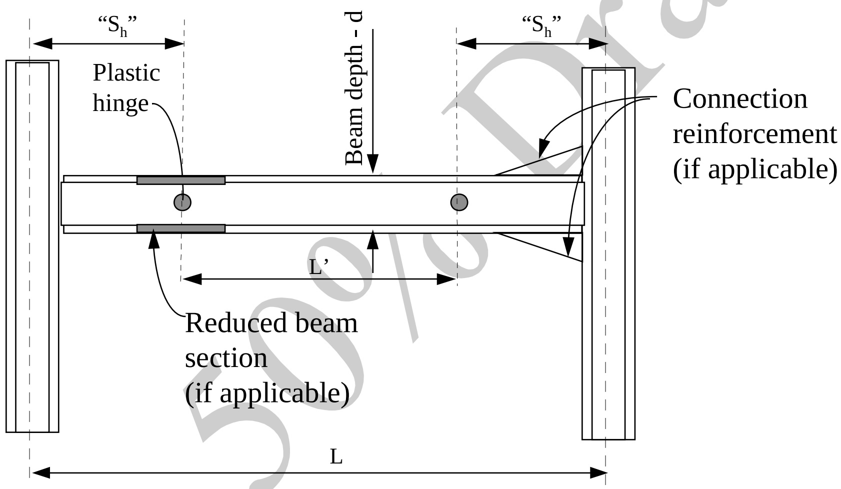


Figure 3-2 - Location of Plastic Hinge Formation

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. If significant gravity load is present, this can shift the locations of the plastic hinges, and in the extreme case, even 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. If gravity demands significantly exceed this level then plastic analysis of the frame should be performed to determine the appropriate hinge locations.

3.2.5 Determine Probable Plastic Moment at Hinges

For fully restrained connections designed to develop plastic hinging in the beam or girder, the probable plastic moment at the location of the plastic hinge should be determined as:

$$M_{pr}=1.1R_yZ_eF_y \qquad (3\text{-}1)$$

where:

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

R_y = A coefficient obtained from AISC Seismic Provisions (See Section 2.5.2).

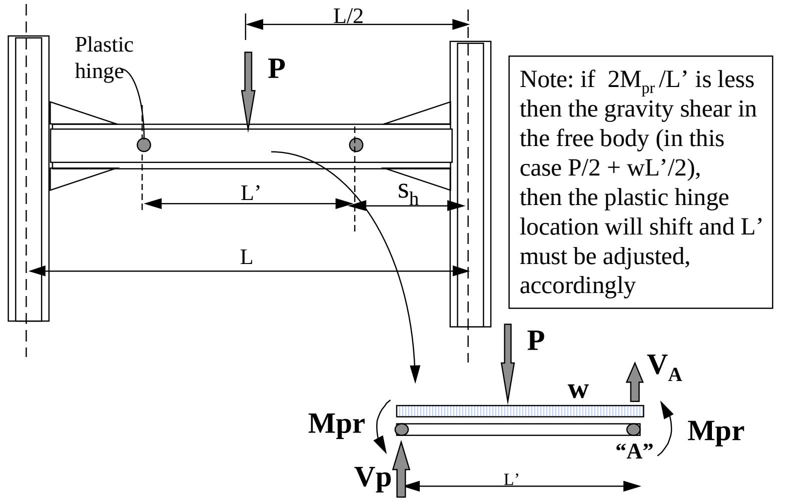
Z_e = The effective plastic modulus of the section (or connection) at the location of the plastic hinge

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

For connections which do not develop plastic hinges in the beam, the hinge strength should be calculated, or determined from tests, for the pertinent yield mechanism, considering the variation in material properties of the yielding elements. For pre-qualified connections, calculation methods to determine the yield strengths of the various active mechanisms are given in Section 3.4.

3.2.6 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 3-3 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.



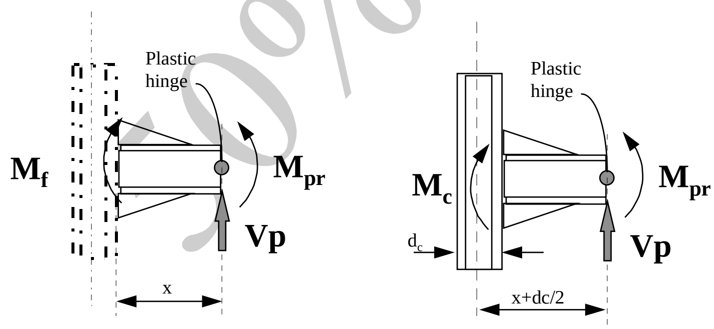
taking the sum of moments about "A" = 0

Vp ={Mpr + Mpr + P L'/2 + wL'²/2}/L'

Figure 3-3 - Sample Calculation of Shear at Plastic Hinge

3.2.7 Determine Strength Demands at Each Critical Section

In order to complete the design of the connection, 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 3-4 demonstrates this procedure for two critical sections, for the beam shown in Figure 3-3.



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

Critical Section at Column Face $\qquad$ Critical Section at Column Centerline

Figure 3-4 - 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.*

3.3 General Requirements

This section provides guidelines for connection design conditions which are considered to be general, that is, those conditions which, when they occur in a connection, 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. These guidelines also apply to pre-qualified connections, unless specifically noted otherwise in the individual connection pre-qualifications.

3.3.1 Beams

3.3.1.1 Beam Flange Stability

The *AISC Seismic Provisions* require that beam flange slenderness ratios $b_f/2t_f$ (b/t) be limited to a maximum of $52/\sqrt{F_y}$. This specific value is intended to allow for some plastic rotation of the beam to occur before the onset of local buckling of the flanges; it has little to do with the performance of a given moment frame connection. However, the actual value of the b/t of the beam involved in a specific connection can have a major effect on how the connection performs. Beams with b/t significantly lower than the maximum may perform better in some connection types and may exhibit worse performance in others. Beams and girders used in moment frames should comply with the limits provided by AISC, except as specifically modified by individual connection pre-qualifications or qualification tests. It is essential that the designer consider the effects of b/t when choosing the connection design or in selecting specific member sizes. Designers should also consider the effects of strength degradation associated with the local buckling on the overall frame performance, as determined from the force/deformation (hysteresis) curves for beams with similar proportions using the selected connection type.

Commentary: The value of b/t recommended by AISC dates to the early days of "Plastic Design". It is intended to be the ratio "below which ample plastic hinge rotations could be relied upon without reduction in the M_p value due to local buckling". While similar to the conditions of steel moment frame design, "plastic design" does not anticipate rotations of the magnitude expected in seismic design, nor does it consider repeated inelastic cycles of loading.

Some researchers and practicing engineers have expressed the opinion *that "local buckling is good", or, "is our friend" when it comes to maintaining the integrity of the joints of the moment connection. This opinion has sometimes been expressed because when beam flanges start to buckle, the amount of force they can carry begins to degrade, resulting in a reduction in demands on the connection. Thus, laboratory research has indicated that if a connection is able to resist the demands imposed on it to the point where local buckling initiates, it will generally be able to withstand very large inelastic connection assembly rotation demands. However, although local buckling of beam flanges "helps" a connection to meet large inelastic rotation demand criteria, it is not a desirable phenomena. The immediate strength degradation that accompanies local buckling of flanges increases both story drifts and P-Δ effects and therefore is not desirable. Also, local flange buckling results in very large local straining of the flanges and the early on-set of low-cycle fatigue induced tearing of the beam flanges, which ultimately limits the ability of the assembly to withstand inelastic rotation demands. Further, severely buckled beam flanges can be even more difficult to repair than fractured connections.*

3.3.1.2 Beam Depth Effects

The most obvious effect of beam depth on connection performance is that for a given drift angle, the flange deformations required for deeper beams are proportionally larger than for shallower beams, or, putting it another way, for the same rotation angle, a deeper beam will have larger strains in the flanges than a shallower beam. For this reason, the pre-qualified connections described later in this chapter include specification of beam depth limitations. For non pre-qualified connections, it is important to limit connection test results used for qualification to depths similar to that of the connection to be used. Section 3.7 of this chapter provides guidelines for interpolation and extrapolation of test results.

3.3.1.3 Beam Flange Thickness Effects

In addition to controlling the stability of the flange under compressive loading, as described above, beam flange thickness also affects the size of welds in welded connections. Thicker flanges, requiring larger welds, can result in larger heat affected zones and residual stresses. Beam flange thickness can also affect the ability to detect defects in welded joints using NDT techniques. For pre-qualified connections, limitations are given on beam weight (flange thickness) based on those used in the various tests used in developing the pre-qualified design. For non pre-qualified connections guidelines for interpolation and extrapolation of test results to other beam sizes are given in Section 3.7 of this chapter.

3.3.2 Welded Joints

3.3.2.1 Through-Thickness Strength

The strength of column material in the through-thickness direction, for typical beam flange to column flange welded connections, need not be explicitly checked in the design, except where the guidelines for the design of a specific pre-qualified connection are based on a certain value as a parameter for the design as indicated in the connection specific design procedure.

Commentary: FEMA 267A section 7.5.1 recommended in commentary that a limit on through-thickness strength of $0.9F_{yc}$ *be used for design. This value was selected somewhat arbitrarily based on computed (using simplified elastic models) through-thickness stresses experienced in successful tests of connections which shifted the hinge point away from the welded joint. Use of this value was accompanied by cautions and it was noted that further testing would be conducted under the SAC Phase II program.*

Such laboratory tests have been conducted by Dexter and Melendrez [Ref. }] and have shown that modern steels, with the conditions of constraint typically found in welded beam-flange-to-column-flange connections, do not exhibit failures of the column flange which can be attributed to insufficient "through-thickness" strength, or excessive through-thickness stress. The primary reason for the high strength exhibited in tests of these conditions is that the constraint developed in the connection elevates both the yield and ultimate tensile strength of the material in the through-thickness direction. In repeated tests, Dexter and Melendez were able to place more than 100 ksi of through thickness stress on A572 Grade 50 column material, without producing failure. Because of these elevated strengths, the studies indicate that the nominal strength of the column steel need not exceed that of the beam steel for purposes of through-thickness performance.

Type C3 (column flange divot) fractures of column flanges discovered after Northridge, that were initially interpreted by some engineers to be a result of through thickness failures, are now attributed to cracking which started in flaws at the weld root and spread into the column flange under the influence of the principle stress field and relative strength and toughness of the materials. This has been demonstrated both by the laboratory testing discussed above as well as confirmatory fracture-mechanics analytical studies. It is now generally accepted that these failures were not due to insufficient through-thickness strength.

Lamellar tearing which is often confused with through-thickness *failure occurs when large stresses, usually due to weld shrinkage, are applied to steels with longitudinal laminations caused by sulfur inclusions. Modern electric furnace steels typically have a sulfur content which is well below the threshold which can cause these problems. Further, the continuous casting process used by modern mills, allows structural shape to be produced with less cold working during the rolling process. This also results in less susceptibility to lamellar tearing. Older steels and steels produced by older processes may have sufficient sulfur content and cold working to exhibit lamellar tearing.*

The design guidelines for certain pre-qualified connections such as the Reduced Beam Section (RBS) are based on a limiting stress in the weld at the face of the column. The limits given are based on results obtained from successful tests and are to limit stresses in the welds and surrounding beam flange material, not the through-thickness stress of the column.

3.3.2.2 Base Material Notch-Toughness

The *AISC Seismic Provisions for Structural Steel Buildings* require that, when used as members in the Seismic Force Resisting System, rolled shapes with flanges 1-1/2 inches thick and thicker and sections made from plates 1-1/2 inches thick and thicker be checked for notch toughness. Such sections are required to have minimum Charpy V-Notch (CVN) toughness of 20 ft.-lbs. at 70 degrees F. Specifications should include this requirement.

Commentary: Research has not clearly demonstrated the need for a specific value of base metal toughness. However, it is judged that base metal toughness is important to prevention of brittle fracture of the base metal in the highly stressed areas of the connection. A number of connections tested in the SAC program have demonstrated base metal fractures at cope holes and at other discontinuities such as at the ends of cover plates (Whittaker, Ref.). In at least some of these tests, the fractures initiated in zones of low toughness. Tests have not been conducted to determine if higher base metal toughness would have reduced the incidence of such fractures.

The CVN value of 20 ft.-lbs. at 70 degrees F, was chosen because it is usually achieved by modern steels, and because steels meeting this criterion have been used in connections which have performed successfully.

Current studies (Ref.) have indicated that modern steels meet this requirement almost routinely even in the thicker shapes currently requiring testing. It has been suggested that the requirement for this *testing could be eliminated by a certification program administered by the mills, but such a program is not currently in existence. Until such time as a certification program is in place, or a statistically meaningful sampling from all major mills has been evaluated, it is recommended that the AISC requirement for testing be met. According to the AISC Commentary, thinner sections are judged not to require testing because they "are generally subjected to enough cross-sectional reduction during the rolling process that the resulting notch toughness will exceed that required." In other words, the toughness is required, but testing to verify it on a project basis is not judged to be necessary.*

3.3.2.3 Weld Wire Notch-Toughness

AISC Seismic Provisions require use of welding consumables with a rated Charpy V-Notch toughness of 20 ft.-lbs. at -20 degrees F, for Complete Joint Penetration (CJP) groove welds used in the Seismic Force Resisting System. The rating of the weld wire is as determined by AWS classification or manufacturer certification. Project specifications should be written to require the use of welding consumables meeting this requirement for the welds indicated.

The use of welding consumables with rated toughness for fillet welds or other weld configurations which may be used in web connections, continuity plate connections to the column web, or other parts of moment frame connections is not specifically addressed here. However, the principle should be followed that project specifications should require use of electrodes matching the properties of the welds specified in the individual connection pre-qualifications, or for project-specific qualification matching those used in the prototype tests used to qualify the connection.

Commentary: Principles of fracture mechanics demonstrate the importance of toughness to resist fracture propagation from flaws, cracks, and backing bars or other stress concentrations, which may be preexisting or inherent, or which may be caused by applied or residual stresses.

The efficacy of weld metal toughness in improving the performance of pre-Northridge type (unreinforced) connections has been demonstrated by Kaufmann and Fisher (Ref.) and by Goel and Stojadinovic (Ref.). Although the connections tested did not typically demonstrate acceptable performance (inelastic rotations in the range of 0.01 were achieved), the performance was definitely superior to that of similar connections welded with consumables not rated for notch toughness. It should be noted that the connections with improved performance also included other improvements, including removal of backing bars and runoff tabs, which help to reduce the presence of initial flaws.

The selection of the CVN value of 20 ft.-lbs. at -20 degrees F, was *made, because this is the most common rated value for readily available electrodes, and because, therefore, electrodes with this rating have been used in the majority of successfully tested connection specimens.*

3.3.2.4 Weld Wire Matching and Overmatching

The use of overmatched weld metals is recommended. Welding consumables specified for CJP groove welds of beam flanges and flange reinforcements should have yield and ultimate strengths at least slightly higher than the expected values of yield and ultimate strength of the beam or girder flanges being welded. Significant overmatching of the weld metal should not be required unless overmatching is specified in the connection pre-qualification or is used in the prototypes tested for project-specific qualification of the connection being used.

Commentary: Some studies (Deierlein, Ref.) have shown that just matching, or under-matching of weld metal is undesirable. The same studies have suggested overmatching may provide beneficial effects by protecting the weld from plasticity. Notwithstanding these studies, the majority of the successful connection tests have used weld metals with yield and tensile strengths in the range of 58 and 70 ksi respectively, which do not provide significant overmatching with beams of Grade 50 steel. Further study would be required to establish the magnitude of improvement due to a higher degree of overmatching and its cost benefit.

3.3.2.5 Weld Backing, Runoff Tabs, Reinforcing Fillet Welds

Project specifications should require that weld backing and runoff tabs be removed from CJP flange welds and from CJP welds of continuity plates to column flanges, unless otherwise noted in the connection pre-qualification or demonstrated as not required by project-specific qualification testing. When backing bars are removed, the weld root should be back-gouged and rewelded and a reinforcing fillet weld should be added. It is generally acceptable to leave backing in place at the beam top flange, and at continuity plate to flange welds, provided that the backing is attached to the column flange with a continuous fillet weld at the side of the backing that is not incorporated into the CJP weld root. Runoff tabs should be removed to within about 1/8 inch of the beam flange and then ground smooth (not flush). Care should be taken when grinding the ends of welds not to create any discontinuities (nicks, scratches, etc.), either in the beam or column flanges or in the end of the weld.

Commentary: As noted in the above discussion of weld notch toughness, backing bars attached in the usual way (tack weld to the column flange at the weld root side of the backing) have been shown to contribute to the onset of cracking at the root of the weld. It was originally hypothesized that the backing created an effective crack equal to the thickness of the backing. Finite element analyses by Deierlein (Ref.) have shown that this *is a relatively minor contributor to the fractures that occurred. Rather, the stress intensity factor is more related to the location of the tip of internal flaws, such as weld root defects, with respect to the non-uniform internal stress distribution. For this reason, flaws created by or obscured by backing bars on the bottom beam flanges (located at the extreme fiber of the beam) experience stress intensity factors that are about three times larger than for similar flaws at backing bars on the bottom of the top flange. Deierlein's studies have also shown that the use of continuous fillet weld reinforcing beneath the backing bar effectively reduces the stress intensity factor created by the backing bar gap by changing the condition from an edge to an interior crack.*

In addition, Paret (Ref.)has found that the root area above the backing is frequently not able to be properly inspected with the backing in place and that the removal of the backing and back gouging is the most effective way to assure that root defects are eliminated.

Another approach, currently under investigation by Ricles (Ref.) involves the use of beveled backing bars. In this approach, the backing bar is beveled so that the root of the weld is actually at the bottom of the backing bar, effectively producing a reinforced weld without backing bar removal. The backing is attached to the column flange with a continuous fillet weld on the under side.

Runoff tabs represent another source of discontinuity at the critical weld location. Additionally, the weld within the runoff tab length is likely to be of lower quality and more prone to flaws than the body of the weld. Flaws in the runoff tab area can create stress concentrations and crack starters and for this reason their removal is recommended. It is important that the process of removal of the runoff tabs not be, of itself, a cause of further stress concentrations, and therefore, it is important that specifications require the workmanship to result in smooth surfaces, free of defects.

3.3.2.6 Overlay Fillet Welds

Overlay fillet welds as a means of reinforcing connections in the construction of new buildings, other than as described in the preceding section, are recommended only when their effectiveness is established by project-specific qualification tests for the particular connection being used.

Commentary: Significant testing has been performed by Anderson (Ref.) on the use of built-up welds (overlay welds) as a means of repairing and reinforcing welded connections of smaller sized beams in existing buildings. Such overlay welds may prove to be beneficial and economical *for connections of smaller beam sizes in new buildings in areas of lower seismicity, or where other conditions of the design indicate that small amounts of inelastic rotation capacity are sufficient. Currently, tests of designs intended for new construction have not been performed.*

3.3.2.7 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.

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.

3.3.3 Other Design Issues for Welded Connections

3.3.3.1 Continuity Plates

Continuity Plates should be provided for all connections 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 (see Section 3.3.2.5). 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.

3.3.3.2 Panel Zone Strength

The *AISC Seismic Provisions* require that the panel zone be checked for strength at a force determined using the following Load Combinations:

$$1.2D + 0.5L + 0.2S + \Omega_O Q_E \tag{3-2}$$

$$0.9D - \Omega_O Q_E \tag{3-3}$$

However, the Provisions state that the shear need not exceed that determined from $0.8\ \Sigma R_y M_p$. This procedure should be used, except that the limit $0.8\ \Sigma R_y M_p$ should be changed to $1.0 \Sigma R_y M_p$ for one-sided connections. Special requirements for panel zone design are provided for certain specific pre-qualified connections, and should be applied for those connections types. For connections designed based on project-specific qualifications, the panel zone strength should match that of the tested connections.

Commentary: The methodology for the design of panel zones, as contained in the AISC Seismic Provisions, is considered to be suitable for use for most cases, except where the particular pre-qualified connection has been shown to be sensitive to panel zone distortion. In the latter case, specific guidelines are given in the section describing the pre-qualified connection. Some connection types actually rely on panel zone distortion to provide a significant part of their plastic rotation capacity, and, for those connections, care must be taken not to oversize or overstrengthen the panel zone.

The limit $0.8\ \Sigma R_y M_p$ in AISC was originally formulated considering that gravity loads on one side of a two-sided connection will inhibit formation of the full M_p on both sides of the column. This, of course, is not the case for one-sided connections. El-Tawil et al (Ref.) pointed out this concern, and their analyses have indicated that use of the 0.8 factor may lead to excessive panel zone distortions for one-sided connections.

It should be noted that in applying the load combinations of AISC, as cited above, the intent is that the equations be applied to the beams on the two column sides in such a way as to obtain the largest panel zone shear. For example, when the beams are of the same size and have the same *loading, the largest panel zone shear will occur when one combination is applied to the beam on one side and the other combination is applied to the opposite beam. When this is done, recognizing that for moment frames the factor Ω_0 is equal to 3, it is likely that for most economically designed frames the limit will either apply or be very close to the value calculated from the load combinations. Clearly, the first load combination will be critical for one-sided connections, and the limit will normally be significantly lower than the value calculated using the load combination. Thus, current practice provides for less conservative design for panel zones in one-sided connections than in two-sided.*

3.3.4 Bolted Joints

3.3.4.1 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).

3.3.4.2 Connection Upgrades

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.

3.4 Pre-qualified Welded FR Connections

This section provides pre-qualification data for alternative types of welded type FR MRSF connections, suitable for use in new construction. Table 3-2 indicates the various types of pre-qualified FR connections, and the structural systems (frame type) for which they are pre-qualified, for designs conducted in accordance with the guidelines of Chapter 2. The table also indicates limiting drift angles and capacity reduction factors for each of these connection types that may be used with the performance evaluation procedures of Chapter 4. Additional pre-qualification data on these various connection types is provided in the following sections.

Table 3-2 - Pre-qualified Welded FR Connections

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**
WURF	3.4.1	OMF	0.01	.9	.02	0.6
WCPF	3.4.2	SMF	0.03	.9	.06	0.75
WFP	3.4.3	SMF	0.03	.9	.08	0.8
RBS	3.4.4	SMF	0.03	.9	.10	0.85
WSH	3.4.5	SMF	0.03	.9	.08	0.85
WDH	3.4.6	SMF	0.04	.9	.10	0.9
SP	3.4.7	proprietary connection[1]				
SW	3.4.8	proprietary connection[1]				
1- Specific qualification data for proprietary connections is not provided by these Guidelines. Refer to the licenser for specific information on the applicability of these connection types to various framing systems.						

3.4.1 Welded Unreinforced Flange (WURF)

This section provides guidelines for design of unreinforced, welded flange connections. These connections utilize complete joint penetration (CJP) groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges directly to column flanges. In this type of connection, no reinforcement other than weld metal, is used to join the flanges. Web joints for these connections are CJP welded. This type of connection should be used only for OMF applications, when such systems are permitted by the building code, or when factored drift demands predicted by structural analysis, conducted in accordance with Chapter 4, can be shown to be lower than the product of $\phi_i\theta_i$, where θ_i and ϕ_I, respectively are the values indicated in Tables 3-2 and 3-3 for connection drift capacity and capacity reduction factor for the appropriate performance level. Figure 3-4 provides a typical detail for this connection type. These connections should be designed in accordance with the guidelines of this section.

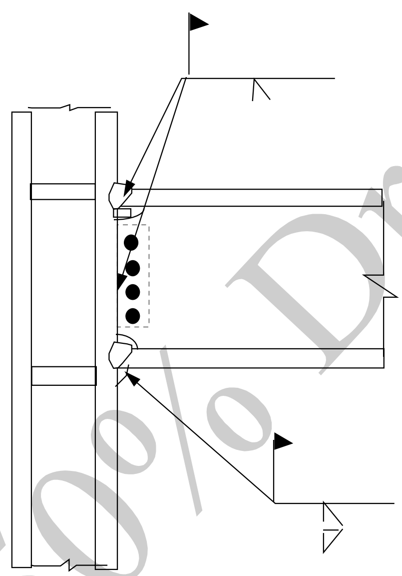


Figure 3-4 - Typical Detail - WUF Connection

Table 3-3 - 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, A992
Cover Plate Material	Minimum specified yield strength equal or larger than that specified

	for the beam material
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913, ASTM A-992
Panel zone to Beam strength ratio	*to be determined*
Column to Beam strength ratio	*to be determined*

3.4.1.1 Procedure for Sizing Shear Tabs

Shear tabs for this type of connection should be sized with minimum thickness, number of bolts, and welding to the column as required to resist erection loads. In addition to serving as an erection connection, the shear tab serves as a backing for the CJP web weld and therefore, it should be continuously fillet or groove welded on the side away from the CJP weld.

3.4.1.2 Procedure for Weld Sizing

The basic flange to flange weld for this connection is a CJP weld. The reinforcing fillet weld, added after removal of the backing, should be a minimum of 5/16 inch. Build-up of the top of the CJP weld is recommended, to provide a minimum of 5/16 inch of reinforcing.

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 have focused on the connection as used in pre-1994 practice, that is, 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 superior notch toughness ratings, relative to the E70T4 material. 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 metal is a major factor in the performance of this type of connection and that the tests demonstrate "the need to impose a fracture toughness requirement 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 program 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.

3.4.2 Welded Cover Plated Flanges (WCPF)

This section provides guidelines for design of welded cover plated flange connections. These connections utilize complete penetration groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges and top and bottom flange cover plates directly to column flanges. Web joints for these connections are welded. This type of connection should be used only when inelastic rotation demands can be shown to be lower than value indicated in Table 3-4. Figure 3-5 provides a typical detail for this connection type. These connections should be designed in accordance with the guidelines of this section.

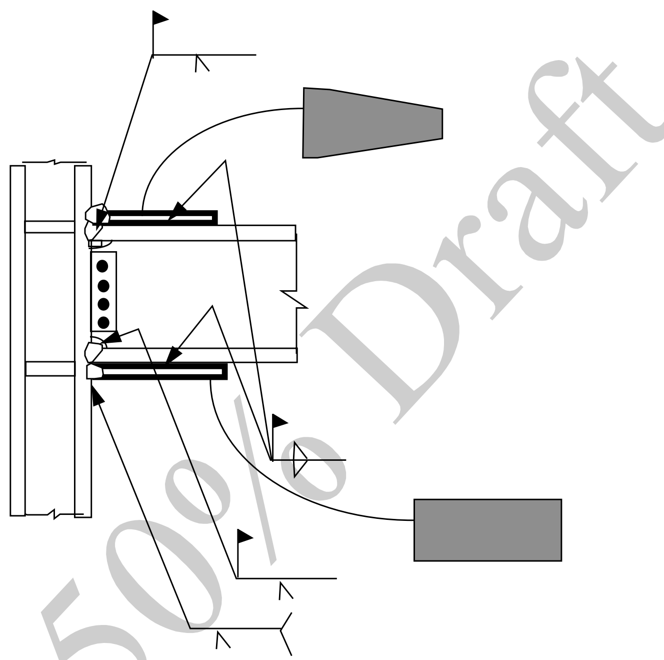


Figure 3-5 Typical Detail WCPF Connection

Table 3-4 Pre-qualification Data for WCPF Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Drift Angle Capacity	0.04 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor ϕ	0.75 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	$d_c/2 + l_p + d_b/4$
Maximum beam size	W36 x
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992

Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913, Grade 50 or 65, ASTM A-992
Panel zone to Beam strength ratio	*to be determined*
Column to Beam strength ratio	*to be determined*

3.4.2.1 Procedure for Sizing Cover Plates

Cover plates for this type of connection should have an area of about ¾ of that of the beam flange.

3.4.2.2 Procedure for Sizing Shear Tabs

Shear tabs for this type of connection should be sized with minimum thickness, bolts and welds to the columns, as required to resist erection loads. The shear tab serves as a backing for the CJP web weld and therefore, it should be continuously fillet or groove welded on the side away from the CJP weld.

3.4.2.3 Procedure for Weld Sizing / Weld Configuration

<<<<< to be developed >>>>>>

3.4.2.4 Continuity Plate Sizing

Continuity Plates should be sized using the guidelines of Section 3.3.3.1, considering the beam flange thickness to be equal to the thickness of the combined flange and cover plate.

3.4.3 Welded Flange Plates (WFP)

3.4.4 Reduced Beam Section (RBS, or Dog Bone)

This section provides guidelines for design of type FR reduced beam section connections. These connections utilize circular radius cuts in both top and bottom flanges to reduce the flange area over a length of the beam near the ends of the beam span. Welds are complete penetration groove welds, with improvements over those used prior to the Northridge earthquake, to join beam or girder flanges directly to column flanges. In this type of connection, no reinforcement other than weld metal, is used to join the flanges. Web joints for these connections are welded. This type of connection should be used only when drift angle demands can be shown to be lower than the value indicated in Table 3-6. Figure 3-7 provides a typical detail for this connection type.

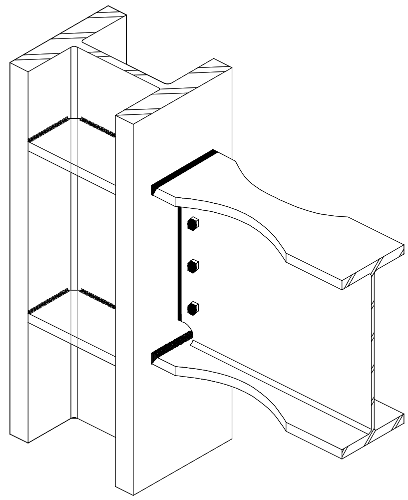


Figure 3-6 Typical RBS Connection (Ref Englehardt)

Table 3-5 Pre-qualification Data for RBS Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Drift Angle Capacity	0.04 radian - collapse prevention 0.015 radian - incipient damage
Capacity Reduction Factor ϕ	0.85 - collapse prevention 0.9 - incipient damage
Hinge location distance s_h	Center of Col. to Center of Reduced Sect.
Maximum beam size	W36 x 194 (Largest Tested)
Beam Steel Grades	ASTM A-572, Gr. 50, ASTM A-992
Maximum column size	unlimited
Column Steel Grades	ASTM A-572, Gr. 50, ASTM A-913, Grade 50 or 65, ASTM A-992
Panel zone to Beam strength ratio	*to be determined*
Column to Beam strength ratio	*to be determined*

3.4.4.1 Procedure for Sizing Section Reduction

Figure 3-7 shows the geometry of a radius cut RBS, and Fig. 3-8 shows the entire moment frame beam. The designer should select the dimensions *a* and *b* according to the following guidelines:

$$a \cong (0.5 \text{ to } 0.75)\, b_f \tag{3-4}$$

$$b \cong (0.65 \text{ to } 0.85)\, d \tag{3-5}$$

where b_f and d are the beam flange width and depth respectively.

The remaining dimension that must be chosen when sizing the RBS is c, the depth of the cut. The value of c will control the maximum moment developed within the RBS, and therefore will control the maximum moment and shears generated at the face of the column. The final dimensions should be chosen so that the maximum moment at the face of the column is in the range of about 85 to 100 percent of the beam's expected plastic moment. The value of c should be chosen to be less than or equal to $0.25b_f$.

The radius of the cut R can be related to dimensions b and c based on the geometry of a circular arc, using the equation in Fig. 3.8. The amount of flange material which is removed at the minimum section of the RBS is sometimes referred to the *percent flange removal* which is computed as $(2c/b_f) \times 100$, where b_f is the unreduced flange width of the beam.

Once dimensions have been selected based on the above guidelines, calculations using standard methods of strength of materials and the general guidelines given in sections 3.2 and 3.3 above can be used to verify that the required condition for maximum moment at the column face is met.

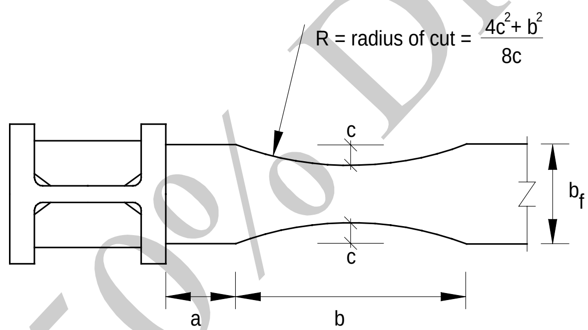


Fig. 3.7 Geometry of Radius Cut RBS (Ref. Englehardt)

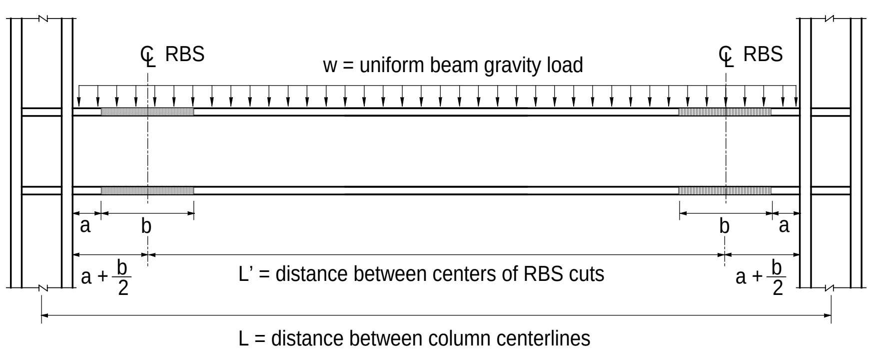


Fig. 3.8 Typical Moment Frame Beam with RBS Connections (Ref. Englehardt)

Commentary: The above recommendations are based on the experimental data and the design approach recommended in Refs. (Englhardt M.D., Winneberger, T., et al, 1996, 1997). Many of the design steps parallel recommendations provided in Ref. (Gross, J.L., Englehardt, M.D. et al).

The guidelines presented here are only for radius cut RBS connections. Other shapes of cuts including constant cut and tapered cut have been successfully tested. In some cases, fractures have occurred at the changes of section of those configurations, indicating that stress concentrations have likely been induced by the relatively abrupt section change and also pointing up the difficulty of avoiding fabrication caused concentrations at those critical locations. Testing has verified the higher degree of reliability of the radius cut RBS as compared to the other configurations.

The overall goal in sizing the RBS cut is to limit the maximum beam moment that can develop at the face of the column to values in the range of about 85 to 100 percent of the beam's expected plastic moment capacity. This approach, in effect, limits the maximum stress at the beam flange groove welds to values on the order of the actual yield stress of the beam. Experiments (Ref. Englehardt and Winneberger, 1997) have shown that connections detailed in accordance with the recommendations provided below are capable of safely resisting this level of moment. As a point of comparison, tests on all-welded moment connections without RBS cutouts often show maximum moments at the face of the column of about 125 percent of M_p or greater. Consequently, the addition of the RBS cutouts in the beam results in a substantial reduction in moment at the face of the column.

The guidelines for selection of RBS dimensions follow recommendations of FEMA 267and FEMA 267A, with several exceptions. Most significant of these exceptions is that FEMA 267A places a limit on the maximum stress permitted at the face of the column equal to ninety percent of the minimum specified yield stress of the column. For the normal case of an A572 Gr. 50 column, this results in a limit of 45 ksi. This limit was established to address concerns regarding the potential for through-thickness failures in column flanges. As noted above, the design procedure goal is to limit the maximum stress at the face of the column to a value on the order of the actual yield stress of the beam, which will typically exceed this 45 ksi limit. The actual yield stress of either an A36 or A572 Gr. 50 beam is anticipated to be in the vicinity of approximately 55 ksi. This exception to the requirements of FEMA 267A has been adopted for several reasons. First, specimens designed according to the procedures described have performed well in laboratory tests. Second, satisfying the 45 ksi stress limit would result in excessively large flange cutouts in many cases, or would require supplemental flange *reinforcement such as cover plates or ribs. Further, as described in section 3.3.1 above, research conducted under the SAC Phase 2 program suggests that the potential for through-thickness failures is considerably less than previously thought, and it is believed that the current limit of 45 ksi can be safely increased.*

In past tests, the dimensions "a" and "b" have generally been chosen based on the judgment of the researchers. In general, these dimensions should be kept as small as possible in order to minimize the growth of moment from the plastic hinge located in the RBS back to the face of the column. The dimension, "a," should be large enough, however, to permit stress in the reduced section of the beam to spread uniformly across the flange width before reaching the column face. Similarly, the dimension "b" should be large enough to avoid excessive inelastic strains within the RBS. Thus, the dimensions "a" and "b" should be chosen considering these differing requirements. Examination of RBS test data indicates that successful connection performance has been obtained for a wide range of values for "a" and "b."

3.4.4.2 Procedure for Sizing Shear Tabs

Shear tabs for this type of connection should be sized with minimum thickness, bolts and welds to the columns, as required to resist erection loads. The shear tab serves as a backing for the CJP web weld and therefore, it should be continuously fillet or groove welded on the side away from the CJP weld.

If it is desired to use a welded shear tab in lieu of a direct CJP weld of the web, the shear tab and welding should be designed to provide strength and stiffness equaling that of the fully welded web.

Commentary: It is recommended that the connection of the beam web to the column be welded. While a welded web connection is more costly than the more conventional bolted web connection, it is believed that the welded web improves the reliability of the connection. The welded web provides for more effective force transfer through the web connection, thereby reducing stress levels at the beam flanges and beam flange groove welds.

As an alternative to a CJP groove weld, the beam web connection can also be made using a heavy welded shear tab. The shear tab is typically welded to the column using either fillet welds or a CJP groove weld. The shear tab, in turn, is then welded to the beam web with fillet welds.

3.4.4.3 Procedure for Flange Weld Sizing

The basic flange to flange weld for this connection is a CJP weld. The reinforcing fillet weld, added after removal of the backing, should be a minimum of 5/16 inch. Flaring of the top of the CJP weld to provide a 5/16 inch fillet is recommended.

3.4.4.4 Fabrication Requirements

The RBS Cut is normally made by thermal cutting. The finished cut should be smooth to the touch, avoiding nicks, gouges, and other discontinuities. After the cut is made, the surface should be ground smooth. All corners should be rounded to minimize notch effects and in addition, cut edges should be ground in the direction of the flange length to have a surface roughness values less than or equal to 1,000, as defined in ANSI/ASME B46.1.

Commentary: Grinding parallel to the flange avoids grind marks perpendicular to the direction of stress, which can act as stress risers.

3.4.4.5 Supplemental Lateral Bracing at RBS

Supplemental lateral bracing should be placed at, or near to, the end of the RBS away from the column. Welding or bolting to provide bracing at the center of the RBS should not be done.

Commentary: FEMA 267A recommends that a lateral brace be provided at the RBS. This recommendation addresses the concern that a beam with RBS cuts may be prone to earlier or more severe instability than a beam without RBS cuts. The concern here is primarily with lateral torsional buckling rather than with local beam flange instability, since it is apparent that the RBS cut reduces the local b/t of the flange. Lateral torsional buckling in the RBS connection may be worse than was envisioned for the old pre Northridge connection for two reasons: 1) because of the narrower flange resulting from the RBS and 2) because the hinge is moved away from the location of the natural brace provided by the column. By providing bracing at the end of the RBS away from the column and using the natural bracing provided by the column, the RBS is effectively braced at both ends. It is felt that welding or bolting which might be required to connect a brace at the center of the RBS may create a local condition which will be harmful to the performance of the desired plastic hinge, so such bracing is not recommended. Rather, it is preferred that bracing and other attachment to the RBS be made outside the zone of anticipated plasticity.

There is not universal agreement that the lateral bracing described is necessary. The following is quoted from Englehardt (Ref.____).

"Virtually all moment connections that dissipate energy by yielding of the beam are subject to varying degrees of beam instability at large levels of inelastic rotation. This is true both for reinforced connections (cover plates, ribs, haunches, etc.) and for RBS connections. This instability generally involves a combination of flange buckling, web buckling and lateral torsional buckling and typically results in a deterioration in the flexural strength of the beam with increasing inelastic rotations. In the experience of the writer, the degree of instability and associated strength deterioration for RBS connections tested in the laboratory have been no more severe, and perhaps somewhat less severe than for many types of reinforced connections. This is demonstrated by the connection test results shown in Fig. 8.

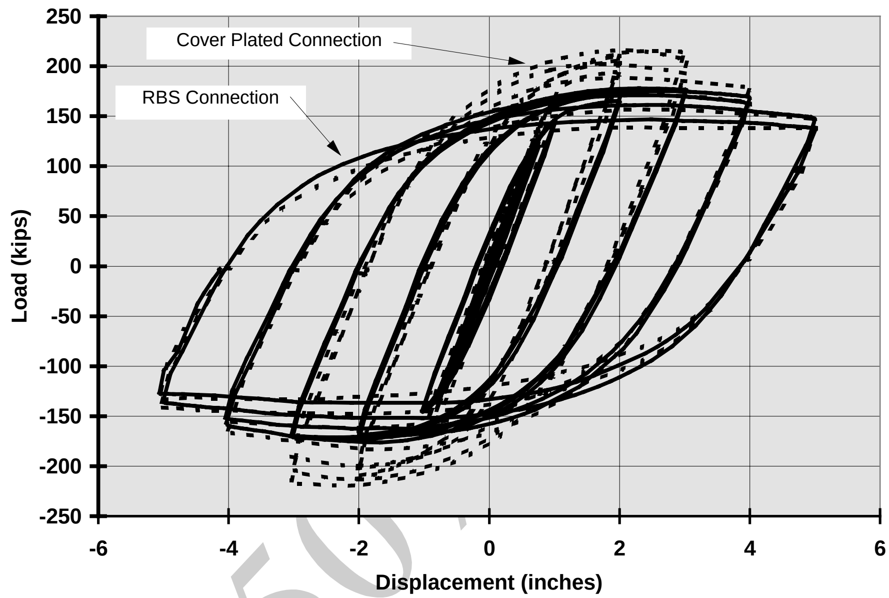


Fig.3-8 - Comparison of Test Results for Cover Plated and RBS Connections

This figure shows a plot of beam tip load versus beam tip displacement for two different test specimens (Refs. 2 and 15). These two specimens were virtually identical, except for the connection detail. Both specimens were constructed with the same member sizes (W36x150 beam and W14x426 column) and heats of steel, and tested in the same test setup with identical member lengths, identical member end support conditions, and identical lateral bracing. Both specimens were subject to the same loading history. The only difference was that one specimen was constructed with a cover plated connection and the other with an RBS connection. Both specimens were provided with a single beam lateral support near the point *of load application.*

As can be seen from Fig. 3-8, the peak strength of the RBS connection is less than that of the cover plated connection. This, of course, is expected and is in fact the advantage of the RBS in that it reduces the moment generated at the connection and the moment delivered to the column. After reaching their peak strength, both connections exhibited some strength deterioration due to combined flange, web and lateral torsional buckling in the beam. Note however that the rate of deterioration is less for the RBS specimen. In fact, at large inelastic deformations, the RBS exhibits the same strength as the cover plated connection. This comparison demonstrates the observation made above, i.e., RBS connections exhibit no more strength deterioration, and perhaps somewhat less deterioration than reinforced connections.

The test data summarized in Appendix A indicates that many RBS connection tests have been conducted without an additional lateral brace at the RBS. There is no instance where an investigator reported unusually severe or unacceptable strength deterioration due to the absence of a lateral support.

Based on the available experimental data, in the judgment of the writer, no additional lateral support is required at the RBS. Of course, the designer should still adhere to the normal code provisions for beam lateral support and for beam flange and web slenderness limits."

The above opinion notwithstanding, it is worth noting that in some early testing of RBS connections, conducted for the AISC at Smith-Emery laboratories, testing of the connection assembly was halted at approximately 0.04 radians total deformation due to the development of large lateral torsional twisting of the beam section and concern that the laboratory apparatus would be damaged. Based on this, these guidelines recommend that lateral bracing for compression flanges be provided.

3.4.5 Welded Single Haunch (WSH)

<<<<< to be developed >>>>>>

3.4.6 Welded Double Haunch (WDH)

<<<<< to be developed >>>>>>

3.4.7 Side Plate (SP)

This proprietary connection is shown schematically in Figure 3-9. Beam shear and flexural stresses are transferred to a pair of vertical side plates that sandwich both the beam and column in the connection area. The plates than transfer these stresses to the column, primarily through shear behavior. The plates are designed with adequate strength to force plastic behavior of the connection assembly into the beam span, adjacent to the edge of the side plates. Specific qualification and design information for this connection type may be obtained from the licenser.

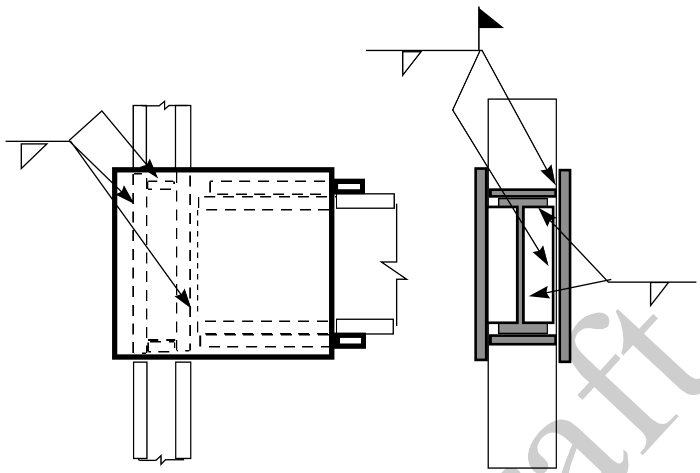


Figure 3-9 Proprietary Side Plate Connection

3.4.8 Slotted Web (SW)

This proprietary connection is shown schematically in Figure 3-10. It is similar in configuration to the WURF connection of Section 3.4.1 except that horizontal slots are cut into the ends of the beam web at the k region of the shape. The slots reduce stress concentrations in the beam flange to column flange joint and also promote buckling of the beam flanges to limit load delivered to these welded joints. Specific qualification and design information for this connection type may be obtained from the licenser.

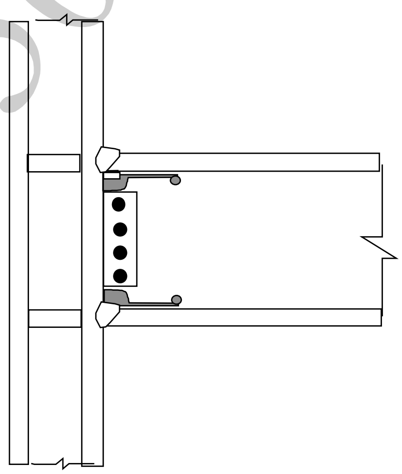


Figure 3-10 Proprietary Side Plate Connection

3.5 Pre-qualified Bolted FR Connections

This section provides guidelines for four specific types of bolted, FR, MRSF connections suitable for different member sizes and with varying inelastic rotation capabilities, as indicated in Table 3-6. Depending on the rotation capacity required for the moment frame type being used and the member sizes needed, the designer may select a suitable connection from the table.

Table 3-6 - Pre-qualified Bolted FR Connections

Connection Type	Criteria Section	Frame Type	Incipient Damage		Collapse Prevention	
			Drift Angle	Reliability Factor Φ_I	Drift Angle	Reliability Factor Φ_c
BEP $d_b \leq 24"$	3.5.1	OMF	0.02	0.9	.06	0.6
BEP $d_b \leq 18"$	3.5.1	IMF	0.03	0.9	.07	0.6
WFPBB $d_b \leq 24"$	3.5.2	OMF	0.02	0.9	.06	0.75
WFPBB $d_b \leq 18"$	3.5.2	IMF	0.03	0.9	.06	0.75
BB	3.5.3	Proprietary connection[1]				

1- Specific qualification data for proprietary connections is not provided by these Guidelines. Refer to the licenser for specific information on the applicability of these connection types to various framing systems.

3.5.1 Bolted End Plate (BEP)

<<<<< to be developed >>>>>>

3.5.2 Welded Flange Plates with Bolted Beam (WFPBB)

This section provides guidelines for design of connections utilizing plates welded to column flanges and bolted to beam flanges. The flange plates are welded to the column flange using CJP welds following the recommendations given in sections 3.3.2.1 through 3.3.2.5. The flange plates are bolted to beam flanges following the recommendations of sections 3.3.3.1 and 3.3.3.2. A detail of this type of connection is shown in figure 3-11. The figure also shows various dimensions and nomenclature which is used in the design procedure which follows. Table 3-7 presents the range of pre-qualification for this connection type.

3.5.2.1 General Design Procedure

The behavior of this type connection can be controlled by a number of different modes including flexural yielding of the beam section, flexural yielding of the cover plates, net-section tensile failure of the beam flange or cover plates, shear failure of the

Table 3-7 Pre-qualification Data for WFPBB Connections

Applicable systems	Mode 1: OMF, IMF, SMF Mode 2: OMF, IMF
Pre-qualified Drift Angle Capacity	Mode 1: 0.04 radian - collapse prevention $\phi = 0.75$ 0.015 radian - incipient damage $\phi = 0.9$ Mode 2: 0.03 radian - collapse prevention $\phi = 0.75$ 0.015 radian - incipient damage $\phi = 0.9$
Hinge location distance s_h	Mode 1: - $S_1+S_3+d_b/2$ Mode 2: - $S_1/2$
Maximum beam size	W24 x 94
Beam Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Plate Material	A36, A572, Gr. 50, A992
Maximum column size	unlimited
Column Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Panel zone to Beam strength ratio	*to be developed*
Column to Beam strength ratio	*to be developed*

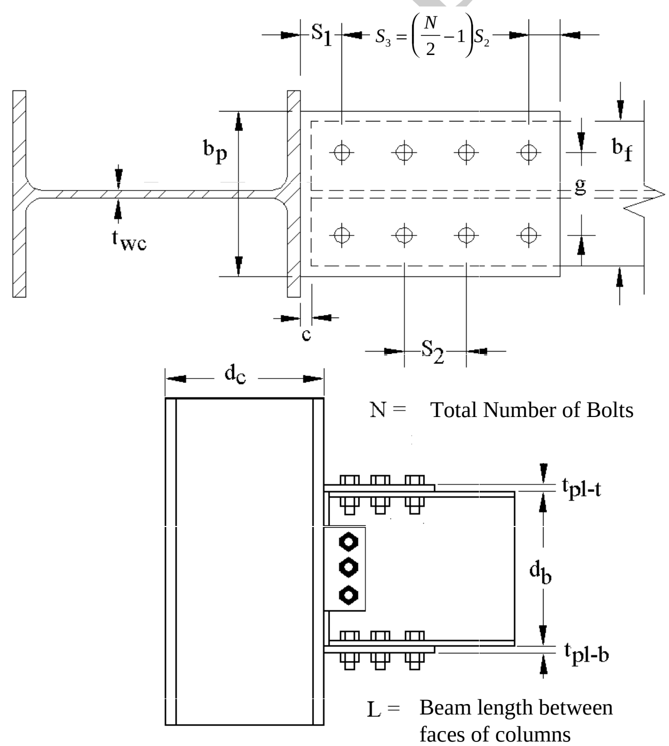


Figure 3-11 Welded Flange Plate Bolted Beam Connection

bolted connections, etc., depending on the relative proportions of these various components. Some of these modes are quite brittle, while others have the ability to display significant ductility. Two behavioral modes are permitted under the pre-qualification for this connection type:

- Mode 1 - Flexural yielding of beam, within span, adjacent to cover plates
- Mode 2 - Flexural yielding of cover plates, adjacent to the column

Regardless of the controlling behavioral mode selected for the connection, it is necessary for the design to satisfy the following relationships:

- The moment at the face of the column, as controlled by yielding in the desired behavioral mode, $M_{yd,}$ must be less than the corresponding moment at the face of the column for yielding of any of the other failure modes, M_{yi}.
- The upper bound moment at the face of the column, as controlled by yielding in the desired behavioral mode, but considering potential over-strength and strain hardening must be less than the moment at the face of the column corresponding with failure of any of the other behavioral modes.

The procedure for calculating the moment at the face of the column, corresponding with yield and failure of each of these behavioral modes is indicated in Sections 3.5.2.2 through 3.5.2.7. The parameters used in these equations are defined in Figure 3-9.

3.5.2.2 Bolt Shear

The moment at the column face, when bolt shear failure occurs at the center line of the bolt group, should be calculated using the following equation:

$$\text{Failure: } M_{fail} = NA_b\left(.62F_{u-bolt}\right)\left(d_b + .5\left(t_{pl-top} + t_{pl-bot}\right)\right)\frac{L}{L - \left(S_1 + .5\left(\frac{N}{2} - 1\right)S_2\right)} \qquad (3\text{-}6)$$

where A_b is the cross sectional area of one bolt. It is assumed that all bolts are of the same diameter.

The moment at the face of the column, when yielding elongation of the bolt holes occurs shall be calculated using the following equation:

$$\text{Yield: } M_{yield} = T_n(d_b + .5(t_{pl-t} + t_{pl-b})\frac{L}{L - (S_1 + .5(\frac{N}{2} - 1)S_2)} \qquad (3\text{-}7)$$

where T_n is the nominal tensile or compressive force in the flange developed when the bolts develop their strength, in accordance with the AISC Load and Resistance Factor design provisions. In addition, T_n shall be calculated using the procedures of the AISC LRFD provisions (Ref.) for calculating block shear capacity See Fig. 3-12 for failure types to be checked.

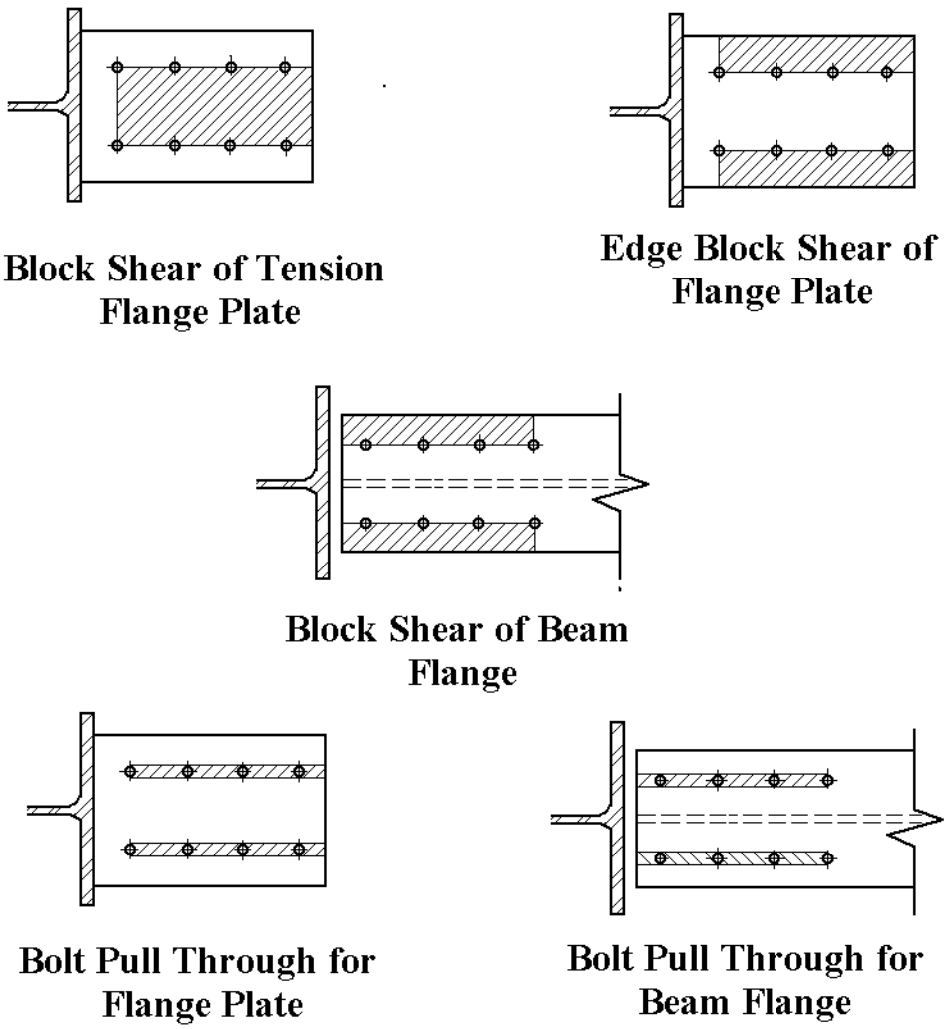


Figure 3-12 Block Shear and Pull Through Failures

3.5.2.3 Flange Plate

The moment at the column face corresponding to flange plate fracture at the net section in the row of bolts nearest to the column face, shall be calculated using the following equation:

Failure: $$M_{fail} = .85F_{u-pl}(b_p - 2(\phi_{bolt} + .125)t_{pl}(d_b + t_{pl})\frac{L}{L - S_1} \quad (3\text{-}8)$$

The moment at the column face corresponding to flange plate yielding between the column face and the first row of bolts shall be calculated using the following equation:

$$\text{Yield:}\quad M_{yield} = 1.1 R_y F_{y-pl} t_{pl} b_p (d_b + .5(t_{pl-t} + t_{pl-b}) \frac{L}{L - \frac{S_1}{2}} \qquad (3\text{-}9)$$

3.5.2.4 Beam Flange

The moment at the column face associated with fracture of the beam at the net section at the last row of bolts away from the column face should be calculated using the following equation:

$$\text{Failure:}\quad M_{fail} = \left\{ F_{u-bm} \left(b_f - 2(\phi_{bolt} + .125) t_{pl} \left(d_b - t_f \right) + .25 F_{y-bm} \left(A_{bm} - 2 b_f t_f \right) \left(d_b - 2_{tf} \right) \frac{L}{L - \left(S_1 + \left(\frac{N}{2} - 1 \right) S_2 \right)} \right) \right\} \qquad (3\text{-}10)$$

The moment at the column face associated with yielding of the beam at the net section at the last row of bolts away from the column face should be calculated using the following equation:

$$\text{Yield:}\quad M_{yield} = 1.1 Z R_y F_{ybm} \frac{L}{L - (S_1 + (\frac{N}{2} - 1) S_2} \qquad (3\text{-}11)$$

3.5.2.5 Groove Weld

The moment at the column face associated with fracture of the CJP groove weld shall be calculated using the following equation:

$$\text{Failure:}\qquad M_{fail} = F_{u-weld} t_e (d_b \frac{t_{pl-t} + t_{pl-b}}{2}) \qquad (3\text{-}12)$$

3.5.2.6 Panel Zone

The moment at the face of the column associated with yielding of the column panel zone shall be calculated using the following equation:

$$\text{Yield:}\qquad Myield = .5225 d_b (1.1 R_y F_{y-col}) d_c t_{w-c} \qquad (3\text{-}13)$$

Commentary: The methodology for design of this type of connection still requires a significant amount of development. The information provided above primarily is interpreted from the 50% SOA Report on Connection Performance. Additional research is now underway by Schneider which is *expected to answer many of the remaining questions. In the interim, this commentary includes a summary of important points for consideration.*

The draft SOA provides the basic design equation as $M_{fail} < M_{yield}$, suggesting that any yielding mode may be selected for the control of the connection design and that all other elements of the connection must be designed to allow them to safely develop the yield strength of the selected mode. This is fine as far as it goes, but it is well recognized that the moment at first yield and that at the required plastic rotation are not the same thing. It is also recognized that each of the yield mechanisms will not have the same relationship of yield strength to failure strength, so that typically in these connections, inelastic behavior will be controlled by combinations of yield mechanisms in several behavior modes, each one occurring successively as strain hardening occurs in earlier ones, until such time as failure occurs.

1. *What is the relationship of yield overstrength to F_u overstrength for a given member? Should we use F_{ye} with F_u (nominal), or is this too conservative? Research from the materials and fracture team indicates that there is no direct relationship between yield and ultimate overstrength, although the ASTM specifications do control, for a given piece of material the ratio of yield to tensile strength.*

2. *How does strain hardening come into the picture? It is probable that if M_{yield} is defined as including yield overstrength and a strain hardening factor, while Fu is nominal, it will not be practical to design these connections. But, will it be conservative enough to do otherwise? In reality, depending on the design, the first yield mechanism to occur may partially strain harden and then another yield mechanism will reach its first yield, and so on. Somewhere in this process, the lowest strength, M_{fail} will be achieved. The problem is how to decide on an appropriate rotation angle the connection will reach through this complex process.*

3. *In the equation provided for calculation of M_{yield} controlled by beam flange yielding, it is assumed that the plastic hinge will occur at the row of bolts farthest from the column face. The table indicates that the Plastic Rotation Limit is "same as the beam." Is it true that one could get a rotation of say .04 radians (approximate capability of the beam itself) at the location of the bolt holes? It seems unlikely to me. Maybe this would be possible if the conservative approach in 1 above were used.*

4. *The equation for "Panel Zone Yielding" considers only the column*

web. In this case it may be appropriate to use the conservative (higher) strength for this mechanism (e.g. the Krawinkler Formula) since the M_{yield} is on the left side of the inequality. As it is in the table, an innocent designer would decide to use panel zone yielding as his mechanism to get .025 radians of plastic rotation while assuming that the M_{yield} formula given would preclude reaching the lowest calculated M_{fail}. Again, the problem is that the formula is expressing first yield for the panel zone, so we are left with the need to know at what point in its strain hardening and gradual utilization of the flange strength other yield mechanisms or failure mechanisms occur, in order to know how much plastic rotation the connection can derive from the column panel zone.

5. *It is not clear that we can make any use of "block shear yielding", it just adds to the complexity expressed by the questions above. This is not to suggest that it should not be discussed in the SOA report.*

Considering all of the above, it seems like the only reasonable approach to making a predictable design for these connections is to pick one, or at most two, yield mechanisms that are predictable, determine their expected strength at the required rotation (or at the peak if it occurs before the required rotation is achieved), and conservatively assure that these are lower than the lowest M_{fail}. This is the approach that has taken in this draft of the Guidelines. For example, if we use an A36 flange plate sized to yield after the panel zone first yield, but before it reaches its peak capacity, and calculate its strain hardened strength at the required rotation (less the rotation provided by the panel zone) and keep the moment at the column face at this strength less than the lowest M_{fail}, the connection may be successful. This of course, brings up another problem: the effect of a given panel zone rotation on the capacity of the flange plate welds needs to be defined.

3.5.3 Bolted Bracket (BB)

This connection type is shown schematically in Figure 3-13. Beam shear and flexural stresses are transferred to the column through a pair of heavy, bolted brackets, located at the top and bottom beam flanges. The concept of using bolted brackets to rigidly connect beams to columns is within the public domain, however, generic pre-qualification data have not been developed for this connection type. One licenser has developed patented steel castings of the bolted brackets, for which specific design qualification data has been prepared. Specific qualification and design information for this connection type may be obtained from the licenser.

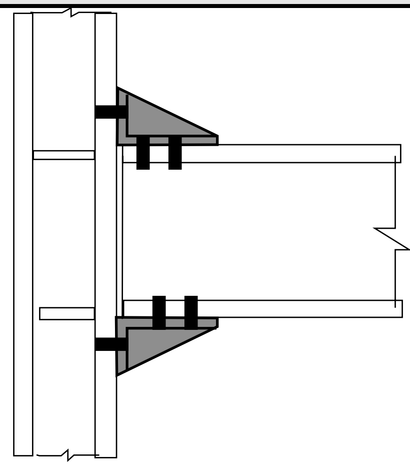


Figure 3-13 Bolted Bracket Connection

3.6 Pre-qualified PR Connections

This section provides guidelines for pre-qualified Partially Restrained (PR) MRSF connections suitable for different member sizes and with varying inelastic rotation capabilities, as indicated in Table 3-8. Depending on the rotation capacity required for the moment frame type being used and the member sizes needed, the designer may select a suitable connection from the types described in the following paragraphs. If a connection type other than one of the pre-qualified connections is to be used, it should be qualified by tests as described later in this section.

Table 3-8 - Pre-qualified Bolted PR Connections

Connection Type	Criteria Section	Frame Type	Incipient Damage		Collapse Prevention	
			Drift Angle - Incipient Damage	Reliability Factor Φ_I	Drift Angle - Incipient Damage	Reliability Factor Φ_C
DST $d_b \leq 30$”	3.6.1	IMF			.08	0.75
DST $d_b \leq 24$”	3.6.1	SMF			.10	0.75
STC $d_b \leq 30$”	3.6.2				.	
STC $d_b \leq 24$”	3.6.2					
SLC $d_b \leq 30$”	3.6.3				.	
SLC $d_b \leq 24$”	3.6.3	OMF			.10	0.75
SC $d_b \leq 24$”	3.6.4	*			.10	0.85
SC $d_b \leq 18$”	3.6.4	*			.12	0.9

3.6.1 Double Split Tee Connections (DST)

This section provides guidelines for design of type PR connections employing bolted split tee connectors between the beam and column flanges. This type of connection should be used only when design parameters are within the limitations indicated in Table 3-9. Figure 3-14 provides a typical detail for this connection type. The design procedure of this section should apply.

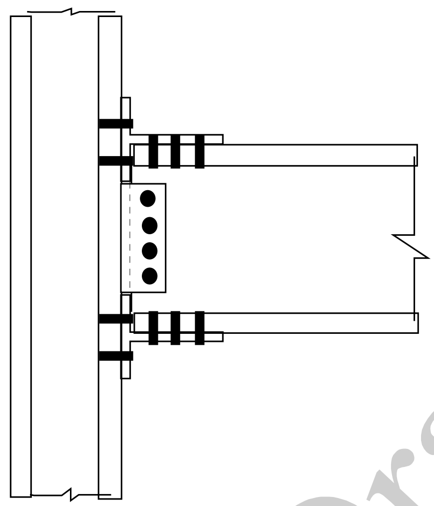


Figure 3-14- Typical Double Split Tee Connection

Table 3-9 - Pre-qualification Data for DST Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Inelastic Rotation Demand	0.10 radian - collapse prevention $\phi = 0.75$ 0.03 radian - incipient damage $\phi = 0.9$
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, A992
Maximum column size	unlimited
Column Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Connection Stiffness	$10EI/L_b$
Connection Strength (fraction of Beam M_p)	30%

3.6.1.1 Procedure for Sizing Tees

<<<<< to be developed >>>>>>

3.6.1.2 Procedure for Sizing Bolts to Column Flange

<<<<< to be developed >>>>>>

3.6.1.3 Procedure for Sizing Bolts to Beam Flange

<<<<< to be developed >>>>>>

3.6.2 Single Tee Composite (STC) Connection

This section provides guidelines for design of type PR connections employing bolted split tee connectors at the beam bottom flange to column flange joint and utilizing a reinforced composite floor slab to transfer beam top flange forces.. This type of connection should be used only when design parameters are within the limitations indicated in Table 3-10. Figure 3-15 provides a typical detail for this connection type.

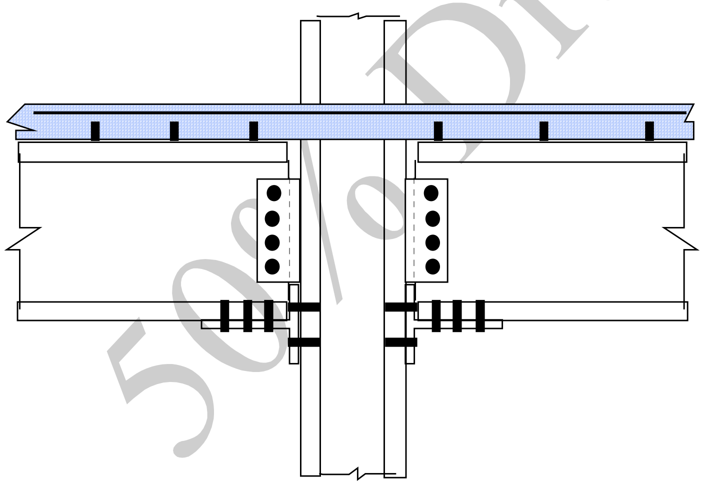


Figure 3-15 - Typical Single Tee Composite Connection

Table 3-11 - Pre-qualification Data for DST Connections

Applicable systems	OMF, IMF, SMF
Pre-qualified Inelastic Rotation Demand	0.10 radian - collapse prevention $\phi = 0.75$ 0.03 radian - incipient damage $\phi = 0.9$
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, A992
Maximum column size	unlimited
Column Material	A36, A572, Gr. 50, A913 Grade 50 or 65, A992
Connection Stiffness	$10EI/L_b$
Connection Strength (fraction of Beam M_p)	30%

3.6.2.1 Procedure for Sizing Tees

<<<<< to be developed >>>>>>

3.6.2.2 Procedure for Sizing Bolts to Column Flange

<<<<< to be developed >>>>>>

3.6.2.3 Procedure for Sizing Bolts to Beam Flange

<<<<< to be developed >>>>>>

3.6.2.4 Procedure for Sizing Shear Studs

<<<<< to be developed >>>>>>

3.6.2.5 Procedure for Sizing Slab Reinforcement

<<<<< to be developed >>>>>>

3.6.3 Single Angle Composite (SLC)

<<<<< to be developed >>>>>>

3.6.4 Shear Tab Composite (SC)

<<<<< to be developed >>>>>>

3.7 Non-pre-qualified Connections

This section provides guidelines for design and project-specific qualification of connections for which there is no current pre-qualification 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 prototype 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 alone.

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.

3.7.1 Testing Procedure

The testing program should follow the general requirements of AISC Appendix S, except that testing should be continued until connection assembly failure occurs. The program should include tests of multiple specimens and should be used to predict the median value of the inter-story drift angle capacity for both the incipient damage and collapse prevention states, together with corresponding resistance factors. The inter-story drift angle capacity, θ, shall be defined as indicated in Figure 3-16. The capacity shall be taken as indicated in Table 3-12.

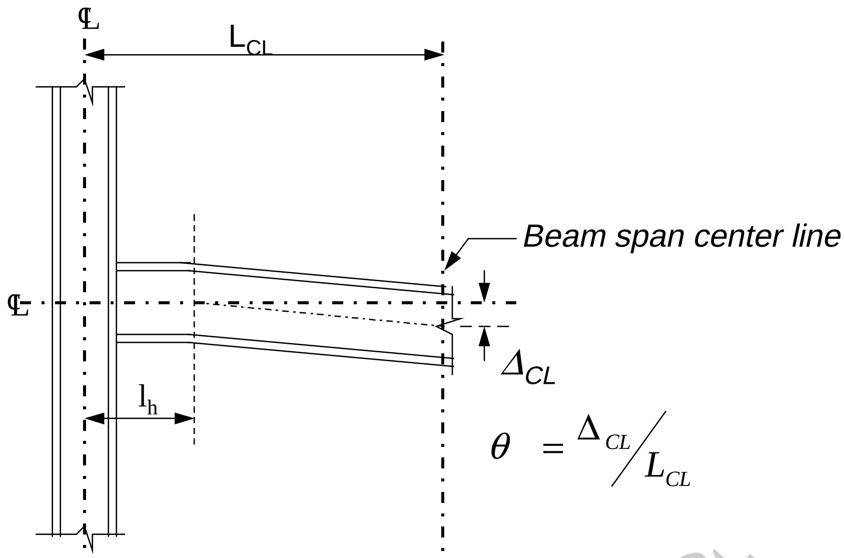


Figure 3-16 - Angular Rotation of Test Assembly

Table 3-12 Inter-story Drift Capacity

Performance Level	Inter-story Drift Capacity
Incipient Damage	Taken at that value of θ, per Figure 3-13, at which peak load resistance occurs.
Collapse Prevention	Taken at that value of θ, per Figure 3-13, at which connection damage is so severe that continued ability to remain stable under gravity loading is uncertain.

Commentary: The AISC Seismic Provisions (Ref.) have been adopted by reference into the 1997 NEHRP Recommended Provisions and are also being considered for adoption for use with the 1997 UBC. The AISC Seismic Provisions include, and require the use of, Appendix S – Qualifying Cyclic Tests of Beam-to-Column and Link-to-Column Connections for qualification of connections which are not pre-qualified. Appendix S includes a complete commentary on the requirements, thus no additional commentary is provided here.

3.7.2 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.

3.7.3 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 angle 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:

$$\phi = e^{-\frac{k\sigma_{\ln(t)}^{2}}{2b}\left(1+\frac{1}{N-1}\right)} \tag{3-15}$$

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.

4. Performance Evaluation

4.1 Scope

This Chapter provides guidelines for performance evaluation of new MRSF structure designs. It includes definition of performance objectives, discussions of expected performance for different design levels, and discussions of performance evaluation procedures and reliability. The guidelines of this chapter may be used in two ways. First, they may be used in an iterative process to design for performance objectives other than those that serve as defaults in the building code. Alternatively, the guidelines of this chapter may be used to evaluate on a building specific basis, the level of confidence with which a given design will actually be able to achieve specific performance objectives. The performance evaluation guidelines contained in this chapter are not intended to permit the design and construction of structures that do not, as a minimum, meet the basic strength and stiffness requirements of the building code.

Commentary: The building code provisions do not include any specific requirement to undertake a performance evaluation as part of the design process, other than to ensure that the structure is capable of providing the minimum strength to resist lateral forces specified for the applicable Seismic Design Category and to provide sufficient stiffness to respond to design earthquake ground motion specifications within the permissible drift limits for the applicable Seismic Use Group. The strength and stiffness evaluation requirements contained in the code are intended to provide a high degree of confidence that SUG-I structures will not experience collapse under Maximum Considered Earthquake ground motions. As indicated in Figure 2-1, commentary to the NEHRP Provisions implies that structures that are adequately designed and constructed in accordance with the Provisions, should be capable of providing other levels of performance for more likely (and less severe) ground motions. The Figure similarly indicates that for structures designed for the requirements of other Seismic Use Groups, other superior performance is expected to be achieved.

Although the NEHRP Commentary implies such performance for structures designed in accordance with the provisions, no effort has ever been made, other than observation of the actual behavior of buildings complying with the code, to determine the adequacy or reliability of the Provisions in actually meeting the implied performance capability.

The performance evaluation procedures of this Chapter are recommended to be followed as part of the design of any structure for which it is desired to obtain seismic performance with known reliability, *or for which it is desired to obtain performance other than that indicated in Figure 2-1 for the various Seismic Use Groups.*

Performance is evaluated in these guidelines based on two basic types of parameters. The first of these is the inter-story drift induced in the structure by the design ground motion. Inter-story drift of MRSF structures is closely related to rotation (elastic and inelastic) demands on individual beam-column connections, and therefore, is closely related to performance. Conventional analytical evaluations of MRSF structures inherently assume that the structures remain integral and stable throughout the response. This assumption is generally valid, unless P-Δ instability, or column failure occurs, either through tensile failure of splices or local buckling failure occurs. The potential for P-Δ instability is directly considered by these guidelines in the establishment of acceptance criteria for inter-story drift. The potential for column splice or buckling failures must be evaluated separately.

It is the intent of these guidelines that structures designed and constructed in accordance with the guidelines would provide a high degree of confidence that the desired performance objectives will be met. Specifically, it is intended that there be less than a 5% chance(95% confidence level) that structures that comply with the performance evaluations contained in these guidelines would experience more severe damage than indicated within the selected hazard return period. The guidelines of this chapter permit this confidence to be evaluated on a building specific basis, and also permit design for alternative performance and alternative levels of confidence.

It is not intended that the performance evaluation guidelines contained in this chapter be used as a means of designing structures that do not conform to the applicable building code requirements. The performance acceptance criteria contained in these guidelines presume that the basic strength of the structure, the compactness of sections and other features of the building design conform to the basic code requirements.

4.2 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 an acceptable probability of exceeding that performance level within a 50 year period. Buildings may be evaluated and designed to meet multiple performance objectives, such as a non-collapse (termed near-collapse in FEMA-303 and collapse prevention in FEMA-273)) performance with a probability of exceedance of 2% in 50 years, and a non-damage condition (termed immediate occupancy in FEMA-273 and FEMA-303) for other probabilities of exceedance, for example 50% in 50 years.

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.

In addition to the specification of a performance level, the specification of performance in FEMA-273 guidelines require 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 - 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 given probability, 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 with regard to attainment of the desired performance to be established. Neither the FEMA-273 Guidelines or the FEMA-302 NEHRP Provisions are able to establish a confidence level for the attainment of specified performance. In general, the design provisions contained in these Guidelines 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 of Chapters 2 and 3 of this document, would experience damage exceeding the desired level more often than specified (e.g. Collapse Prevention at a 10% probability of exceedance in 50 years, rather than a 2% probability of such exceedance).*

This chapter of the guidelines provides procedure for designing for performance other than the default performance objectives upon which Chapter 2 and 3 are based and also to establish building-specific confidence levels with regard to attainment of specified performance.

4.2.1 Hazard Specification

4.2.1.1 General

Earthquake hazards include direct ground fault rupture, ground shaking, liquefaction, lateral spreading, and landsliding. 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 shall also be considered in structural performance evaluation.

4.2.1.2 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 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, 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.

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.

The 1997 NEHRP Provisions consider two levels of ground motion for design - an MCE level and a DE level. However, except for base isolated structures, only one of these levels, the DE, is actually used in the design process. DE ground shaking parameters are obtained by reference to design maps, which define shaking parameters for the MCE level. These MCE shaking parameters are then adjusted for site response effects and reduced by a factor of 2/3 to attain DE level parameters. The 2/3 reduction is based on the presumption that structures designed and *constructed in accordance with the Provisions have an inherent margin of 1.5 against collapse. That is, it is anticipated that such structures could experience at least 150% of the design ground motion without collapse. In essence, therefore, the 1997 NEHRP Provisions are intended to provide for Collapse Prevention performance for MCE earthquake demands. Reference to the DE level was left in the Provisions, to provide a link back to earlier code approaches in which design was intended to provide for Life Safety performance for a design level event.*

4.2.1.3 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 subject to these hazards, the effects of the projected ground displacements shall be evaluated using a mathematical model of the structure. The severity of these hazards used in performance evaluation shall be compatible with that used in specification of ground shaking hazards.

Commentary: Most sites are not at significant risk from 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.

4.2.2 Performance Levels

Building performance is a combination of the performance of both structural and nonstructural components. Table 4-1 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 some variation of the extent of damage among buildings achieving the same Performance Level must be expected.

Independent performance definitions are provided for structural and nonstructural components. Structural performance levels are identified in these Guidelines by both a name and numerical designator in Section 4.2.2.1. Nonstructural performance levels are identified by a name and alphabetic designator in Section 4.2.2.2.

Table 4-1 - Building Performance Levels

	Building Performance Levels	
	Collapse Prevention Level (5-E)	Immediate Occupancy Level (1-B)
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, these are treated independently, with separate structural and nonstructural performance levels defined. Each building performance level comprises the individual structural and nonstructural performance levels selected by the design team.

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. Definitions of nonstructural performance levels, as contained in FEMA-273 are reproduced here, only for reference. FEMA-273 provides a more complete set of *recommendations with regard to evaluating the performance of nonstructural components.*

4.2.2.1 Structural Performance Levels

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

Structural Performance Levels are the Incipient Damage Level (S-1), and the Collapse Prevent Level (S-5). Table 4-2 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 (inter-story drift ratios and component capacities) 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 4-2 - Structural Performance Levels

		Structural Performance Levels	
Elements	Type	Collapse Prevention S-5	Incipient Damage S-1
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	3%-6% depending on structural system	1% - 1-1/2% transient negligible permanent

4.2.2.1.1 Incipient Damage Performance Level (S-1)

Structural Performance Level S-1, 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.

4.2.2.1.2 Collapse Prevent Performance Level (S-5)

Structural Performance Level S-5, 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 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. 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.

4.2.2.2 Nonstructural Performance Levels

Nonstructural Performance Levels are defined in these Guidelines for reference only. No specific guidelines are provided for attaining these performance levels. Refer to *FEMA-273* for more detailed information on the performance design of nonstructural components and systems.

4.2.2.2.1 Operational Performance Level (N-A)

Nonstructural Performance Level A, Operational, means the post-earthquake damage state of the building in which the nonstructural components are able to support the building's intended function. Under this level, most nonstructural systems required for normal use of the building including lighting, plumbing, HVAC, computer systems are functional although minor cleanup and repair of some items may be required. This performance level requires considerations beyond those that are normally within the sole province of the structural engineer. In addition to assuring that nonstructural components are properly mounted and braced within the structure, in order to achieve this performance, it is often necessary to provide emergency standby utilities. It may also be necessary to performance rigorous qualification testing of the ability of key electrical and mechanical equipment items to function during or after strong shaking.

4.2.2.2.2 Immediate Occupancy Level (N-B)

Nonstructural Performance Level B, Immediate Occupancy, means the post-earthquake damage state in which only limited nonstructural damage has occurred. Basic access and life safety systems, including doors, stairways, elevators, emergency lighting, fire alarms, and suppression systems, remain operable, provided that power is available. There could be minor window breakage and slight damage to some components. Presuming that the building is structurally safe, it is expected that occupants could safely remain in the building, although normal use may be impaired and some cleanup and inspection may be required. In general, components of mechanical and electrical systems in the building are structurally secured and should be able to function if necessary utility service is available. However, some components may experience misalignments or internal damage and be inoperable. Power, water, natural gas, communications lines, and other utilities required for normal building use may not be available. The risk of life-threatening injury due to nonstructural damage is very low.

4.2.2.2.3 Life Safety Level (N-C)

Nonstructural Performance Level C, Life Safety, is the post-earthquake damage state in which potentially significant and costly damage has occurred to nonstructural components but they have not become dislodged and fallen, threatening life safety either within or outside the building. Egress routes within the building are not extensively blocked, but may be impaired by lightweight debris. HVAC, plumbing, and fire suppression systems may have been damaged, resulting in local flooding as well as loss of function. While injuries may occur during the earthquake from the failure of nonstructural components, it is expected that overall, the risk of life-threatening injury is very low. Restoration of the nonstructural components may take extensive effort.

4.2.2.2.4 Hazards Reduced Level (N-D)

Nonstructural Performance Range D, Hazards Reduced, represents a post-earthquake damage state range in which extensive damage has occurred to nonstructural components, but large or heavy items that pose a falling hazard to a number of people such as parapets, cladding panels, heavy plaster ceilings, or storage racks are prevented from falling. While isolated serious injury could occur from falling debris, failures that could injure large numbers of persons, either inside or outside the structure, should be avoided. Exits, fire suppression systems, and similar life-safety issues are not addressed in this performance level.

4.3 Evaluation Approach

Performance evaluation of MRSF structures can be initiated only after a preliminary design, conforming to the building code provisions, has been prepared. The basic approach 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 the design must be revised and the performance evaluation process repeated. 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.

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:

$$\lambda = \beta e^{\left(\frac{k}{2b}\Sigma \sigma_i^{\,2}\right)} \qquad (4\text{-}1)$$

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 inter-story 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 3 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:

$$\gamma_{con} = \frac{\phi \Delta_C}{\lambda \Delta_D} \qquad (4\text{-}2)$$

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.

4.4 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 2.7.

4.4.1 Alternative Procedures

Four alternative analytical procedures are available for use in performance evaluation of MRSF structures. The basic analytical procedures are described in detail in *FEMA-273*. This section provides supplementary guidelines on the applicability of the *FEMA-273* procedures and also provides supplemental modeling recommendations. The four procedures are:

- Linear static procedure - an equivalent lateral force technique, similar, but not identical to that contained in many model building code provisions
- Linear dynamic procedure - an elastic, modal response spectrum analysis or an elastic time history analysis
- Nonlinear static procedure - a simplified nonlinear analysis procedure in which the forces and deformations induced by a monotonically increased pattern of lateral loading is evaluated using a series of incremental elastic analyses of

structural models that are sequentially degraded to represent the effects of structural non-linearity.

- Nonlinear dynamic procedure - a nonlinear dynamic analysis procedure in which the response of a structure to a ground motion 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. Table ( ) contains hysteretic models for different types of beam-column connections and other modeling parameters for nonlinear dynamic analyses (distributed plasticity models, point hinge models, bilinear nondegrading and degrading, cyclic stress-strain models, post-fracture models, panel zone, etc.).

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 design ground motion. Once the values of these response parameters are predicted, the structure is evaluated for adequacy using the basic equation:

$$\gamma_{con} = \frac{\phi \Delta_C}{\lambda \Delta_D} \qquad (4\text{-}3)$$

where:

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

Δ_D = *the predicted inter-story drift demand*

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

Δ_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, and several alternative modeling approaches, depending on the performance levels, to account for these uncertainties.

4.4.2 Procedure Selection

Table 4-3 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 4-3 Analysis Procedure Selection Criteria

Performance Level	Analysis Procedure: Linear Static	Linear Dynamic	Nonlinear Static	Nonlinear Dynamic
Incipient Damage S-1	Permitted for regular structures, per the NEHRP Provisions $\lambda = 1.3$	Permitted for structures of any configuration $\lambda = 1.0$	Permitted for structures of any configuration $\lambda = 1.2$	Permitted for structures of any configuration $\lambda = 1.0$
Collapse Prevention S-5	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 $\lambda = 1.0$

4.4.3 Linear Static Procedure (LSP)

4.4.3.1 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 4.5. 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 4.5.

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 static lateral forces whose sum is equal to the pseudo lateral load. The magnitude of the pseudo lateral load has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. 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.*

4.4.3.2 Modeling and Analysis Considerations.

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:

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

where

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

C_t =0.035 for moment-resisting frame systems of steel

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

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

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \tag{4-5}$$

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.

4.4.3.3 Determination of Actions and Deformations

4.4.3.3.1 Pseudo Lateral Load

A pseudo lateral load, given by equation 4-6, shall be independently calculated for each of two orthogonal directions of building response, and applied to a mathematical model of the building structure.

$$V = C_1 C_2 C_3 S_a W \qquad (4\text{-}6)$$

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 4.4

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 stiffness degradation and strength deterioration on maximum displacement response. Values of C_2 for different framing systems and Performance Levels are listed in Table 4-4. 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. This effect is in addition to the consideration of static P-Δ effects as defined in Section 2.7. C_3 shall be calculated as $1 + 5\,(\theta\text{-}0.1)/T$. 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.

$$\theta = \frac{P\Delta}{VH} \qquad (4\text{-}7)$$

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 4-4 - Correlation 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	2.0		
T ≥ 1.0 Sec	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 (4-6) may be significantly larger than the actual strength of the structure to resist this force. The acceptance criteria in Section 4.5 are developed to take this aspect into account.

4.4.3.3.2 Vertical Distribution of Seismic Forces

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

$$F_x = C_{vx} V \tag{4-8}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{4-9}$$

where

k = 1.0 for T ≤ 0.5 second

= 2.0 for $T \geq 2.5$ seconds

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

C_{vx} = Vertical distribution factor

V = Pseudo lateral load from Equation (4-6)

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

4.4.3.3.3 Horizontal Distribution of Seismic Forces

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

4.4.3.3.4 Floor Diaphragms

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

4.4.3.3.5 Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral loads in accordance with Equations 4-6, and 4-8 and stiffnesses obtained from Chapter 2. 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 4-2.

4.4.3.3.6 Determination of Column Demands

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

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \tag{4-10}$$

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 4-5

Table 4-5 Value of Load Factors λ_c for Columns - Linear Static Procedure

Column Located In	$\overline{M/M_p}$ [1] ≤ 1	$1 < \overline{M/M_p} \leq 2$	$2 < \overline{M/M_p}$
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.

4.4.4 Linear Dynamic Procedure (LDP)

4.4.4.1 Basis of the Procedure

Linear dynamic procedure analysis of MRSF structures shall 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 4-4.

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), inertial 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 Response Spectrum analysis (RSA) or Response-History Analysis (RHA). Modal spectral analysis is carried out using unreduced, linearly-elastic response spectra scaled to the appropriate hazard level. As with the LSP, it is expected that the

LDP will produce estimates of displacements and inter-story drifts that are approximately correct, but will produce estimates of 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 4.5. Calculated displacements are factored by the applicable load factor, λ, obtained from Table 4-3 and compared with factored acceptable values, per Section 4.5. 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 4.5.

Commentary: Under the LDP, either a response spectrum or response history analysis may be performed. Of these two approaches, response spectrum analysis is both easier to perform and provides more reliable results, and therefore, is the preferred approach under these guidelines. The results of response history analysis are highly dependent on the peculiarities of the individual ground motion records used in the analysis. Every record ;results in spectra with large peaks and valleys. Depending on the periods of the structure being analyzed, the structure may end up either at a peak or a valley in a given record, and therefore, may give very different predictions of structural response for multiple records representing similar ground shaking events. For this reason, these guidelines require that when response history analysis is used, a number of analyses, using different records be performed and that response quantities used for design be based on statistics obtained from these multiple analyses. The response spectrum analysis approach avoids these complexities and when appropriate smoothed design spectra are utilized, provides valid design response quantities with less effort.

4.4.4.2 Modeling and Analysis Considerations

4.4.4.2.1 General

The LDP shall conform to the criteria of this section. The analysis shall be based on appropriate characterization of the ground motion. The modeling and analysis considerations set forth in Section 4.4.3.2 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.

4.4.4.2.2 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

4.4.4.2.3 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 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.

4.4.4.2.4 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.

4.4.4.3 Determination of Actions and Deformations

4.4.4.3.1 Factored Inter-story Drift Demand

Factored inter-story drift demand shall be obtained by multiplying 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 4.4.3.2 and by the applicable λ obtained from Table 4-3.

4.4.4.3.2 Determination of Column Demands

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

$$P_c' = \frac{\lambda_c P}{C_1 C_2 C_3} \qquad (4\text{-}10)$$

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 4-6

Table 4-6 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	$\frac{1.25}{\overline{\overline{M/M_p}}}$	$\frac{1.5}{\overline{\overline{M/M_p}}}$
10 stories below the top 3 stories	1.0	$\frac{1.15}{\overline{\overline{M/M_p}}}$	$\frac{1.25}{\overline{\overline{M/M_p}}}$

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.

4.4.5 Nonlinear Static Procedure (NSP)

4.4.5.1 Basis of the Procedure

Under the Nonlinear Static Procedure (NSP), a model directly incorporating the inelastic material and geometric response 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 or displacements until either a target displacement is exceeded or mathematical instability occurs. The target displacement is intended to approximate the total maximum displacement likely to be experienced by the actual structure, 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 4.4.5.3. 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 4.5. Calculated inter-story drifts and column and column splice forces are factored, and compared directly with factored acceptable values for the applicable performance level.

> *Commentary: The nonlinear static analysis approach provides valid results only if response is dominated by the first mode behavior of the structure. This is the basic reason that these guidelines recommend this approach be used only for structures with relatively short periods. What constitutes a building with a "short period" is very much dependent on the spectral characteristics of ground shaking anticipated at the site. The small magnitude events, that dominate the hazard at many central and eastern U.S. sites, tend to have most of their energy at very short periods, particularly on firm soil and rock sites. For sites subject to such shaking, nonlinear static analyses may be valid only for very short and rigid structures.*

4.4.5.2 Modeling and Analysis Considerations

4.4.5.2.1 General

In the context of these *Guidelines*, the NSP involves the monotonic application of lateral forces or displacements to a nonlinear mathematical model of a building until the displacement of the control node in the mathematical model exceeds a target displacement. For buildings that are not symmetric about a plane perpendicular to the applied lateral loads, the lateral loads must be applied in both the positive and negative directions, and the maximum forces and deformations used for design.

The relation between base shear force and lateral displacement of the control node shall be established for control node displacements ranging between zero and 150% of the target displacement, δ_t, given by Equation 4-12. 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 shall be applied to appropriate elements and components of the mathematical model during the NSP.

The analysis model shall 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.

4.4.5.2.2 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.

4.4.5.2.3 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 4-9, 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.

4.4.5.2.4 Period Determination

The effective fundamental period T_e in the direction under consideration shall 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 shall be calculated as:

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (4\text{-}11)$$

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 4-1 for further information.

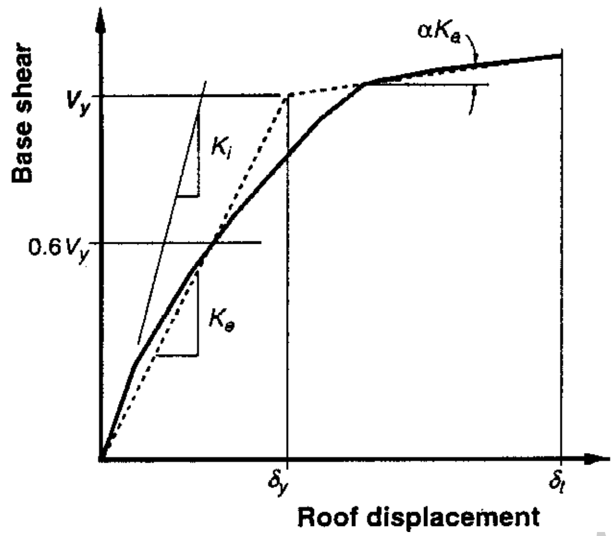


Figure 4-1 Calculation of Effective Stiffness, K_e

4.4.5.2.5 Analysis of Three-Dimensional Models

Static lateral forces shall 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.

4.4.5.2.6 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.

4.4.5.3 Determination of Actions and Deformations

4.4.5.3.1 Target Displacement

The target displacement δ_t for a building with a rigid diaphragm at each floor level shall 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:

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

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 4-7

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 4.4.3.3. 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 4.4.3.3.

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 Equation 4-14. Values for C_3 need not exceed the values set forth in Section 4.4.3.3.

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:

$$R = \frac{S_a}{V_y / W} \cdot \frac{1}{C_0} \qquad (4\text{-}13)$$

Table 4-7 **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 4-1)

W = Total dead load and anticipated live load, as calculated in Section 4.4.3.3.

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

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{3/2}}{T_e} \qquad (4\text{-}14)$$

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 4-1)

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 4-12. 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 4-12 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 4-12, 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 2.7 to account for system torsion.

4.4.5.3.2 Floor Diaphragms

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

4.4.5.3.3 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 4-3.

4.4.5.3.4 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces at the target displacement by the applicable load factor λ from Table 4-3.

4.4.6 Nonlinear Dynamic Procedure (NDP)

4.4.6.1 Basis of the Procedure

Under the Nonlinear Dynamic Procedure (NDP), inertial 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 for 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 records. Calculated response can be highly sensitive to characteristics of individual ground motions; therefore, it is necessary to carry out the analysis with more than one ground motion record. Because the numerical 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.

Results of the NDP are to be checked using the applicable acceptance criteria of Section 4.5. Calculated displacements and internal forces are factored, and compared directly with factored acceptable values for the applicable performance level.

4.4.6.2 Modeling and Analysis Assumptions

4.4.6.2.1 General

The NDP shall conform to the criteria of this section. The analysis shall be based on characterization of the seismic hazard in the form of ground motion records, compatible with a spectrum that has been scaled to the appropriate level of hazard. The modeling and analysis considerations set forth in Section 4.4.5.2 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.

4.4.6.2.2 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.

4.4.6.2.3 Response-History Method

Response-History Analysis shall be performed using pairs of horizontal ground motion histories.

Multidirectional excitation effects should 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.

4.4.6.3 Determination of Actions and Deformations

4.4.6.3.1 Modification of Demands

The effects of torsion should be considered according to Section 2.7.

4.4.6.3.2 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 4-3.

4.4.6.3.3 Factored Column and Column Splice Demands

Factored demands on columns and column splices shall be obtained by multiplying the calculated column forces at the target displacement by the applicable load factor λ from Table 4-3.

4.5 Acceptance Criteria

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

$$\gamma_{con} = \frac{\phi C}{\lambda D} \qquad (4\text{-}15)$$

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

for each of the performance parameters indicated in Table 4-8. 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 4-9, or through direct calculation of confidence level through the procedures of Section 4.6. 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 4-8 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 4.5.1
Column Axial Load	The adequacy of each column to withstand the calculated maximum compressive load for that column shall be evaluated. Refer to Section 4.5.2
Column Splice Tension	The adequacy of column splices to withstand calculated maximum tensile demands for the column shall be evaluated. Refer to Section 4.5.3

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.

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.

Table 4-9 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, 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 uncertainty. 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 4-3, the load factors associated with nonlinear analysis approaches are generally lower than those associated with the linear approaches. Similarly, as reflected in Chapter 3, 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 4.6 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 3 presents procedures for determining resistance factors, based on connection behavior. The more detailed procedures of Section 4.6 may be used, when warranted, to reduce the uncertainty inherent in performance prediction and potentially obtain more optimistic estimates of probable performance.

4.5.1 Inter-story 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 4-10, based on global response, or the product of the resistance factor ϕ and capacity C, obtained from Chapter 3 for the beam-column connections incorporated in the structure. In lieu of the values contained in Table 4-10, the more detailed procedures of Section 4.6.1 may be used to determine inter-story drift capacity as limited by global building response.

4.5.2 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.

4.5.3 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:

$$x=a+b \tag{4-16}$$

Table 4-10 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

4.6 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 4.6.1
- Development of a suite of ground motion accelerograms in accordance with Section 4.6.2
- Performance of a suite of dynamic pushover analyses in accordance with Section 4.6.3
- Calculation of factored drift capacity in accordance with Section 4.6.4
- Calculation of confidence index, γ_{con}, and inherent confidence in building performance, in accordance with Section 4.6.5

4.6.1 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 4-11.

Table 4-11 Approximate Hazard Parameter, *k*

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

4.6.2 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 4.6.1. The accelerograms shall be scaled to achieve spectral compatibility in accordance with the guidelines of FEMA-273.

4.6.3 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 4.6.2, 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 inter-story drift obtained from the analysis shall be created. This plot is termed a dynamic pushover plot.

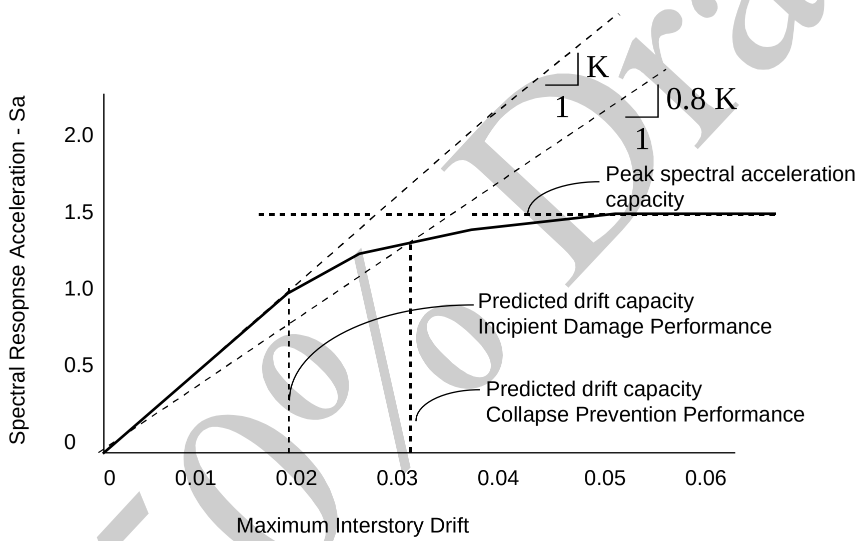


Figure 4-2 Dynamic Pushover Curve

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 4-2. The inter-story drift capacity for collapse prevention performance shall not be taken as greater than 0.1.

4.6.4 Determination of Factored Inter-story Drift Capacity

The inter-story drift capacities δ_i, determined from each of these 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, σ_{ln_δ} of the natural logarithms of the inter-story drift capacities. A resistance factor, ϕ, shall be determined from the equation:

$$\phi = e^{-k\sigma_{\ln\delta}^2 / 2b} \tag{4-17}$$

where: k = the slope of the hazard curve, determined in accordance with Section 4.6.1

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 analyses

Factored inter-story drift demand for global response shall be taken as the product of ϕ determined in accordance with equation 4-17 and the median inter-story drift capacity determined from the dynamic pushover analyses.

4.6.5 Determination of Confidence Level

A performance confidence index, γ_{con}, shall be determined in accordance with Section 4.5, 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}:

$$K_x = \frac{\ln(\gamma_{con})}{b\sigma_{UT}} + \frac{k\sigma_{UT}}{2} \tag{4-18}$$

where:
k = the slope of the hazard curve, determined in accordance with Section 4.6.1
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 4-12.

Table 4-12 Uncertainty Measures for Different Analytical Procedures

Analytical Procedure	σ_{UT}

Linear Static Procedure	0.6
Linear Dynamic Procedure	0.7
Nonlinear Static Procedure	0.8
Nonlinear Dynamic Procedure	0.9

The level of confidence with regard to the target performance shall be determined by interpolation from, Table 4-13.

Table 4-13 - Values of Kx for Various Levels of Confidence

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

5. MATERIALS AND FRACTURE-RESISTANT DESIGN

5.1 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 knowledgeable 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.

5.2 Parent Materials

5.2.1 Steels

Designers should specify materials that are readily available for building construction and that will provide suitable ductility and weldability for seismic applications. Structural steels that may be used in the lateral-force-resisting systems for structures designed for seismic resistance without special qualification include those contained in Table 5-1. Refer to the applicable ASTM reference standard for detailed information.

Table 5-1 - 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 that 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.*

5.2.2 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 greater use of recycled steel in the production process. 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.

5.2.3 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 5-3 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 5-4 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 5-3 - 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 the A992 steel material 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	

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.

5.2.4 Toughness Properties

For critical connections, non-redundant components and unusual or difficult geometries involving Group 3 (with flanges 11/2 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 5-1 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 5-1.

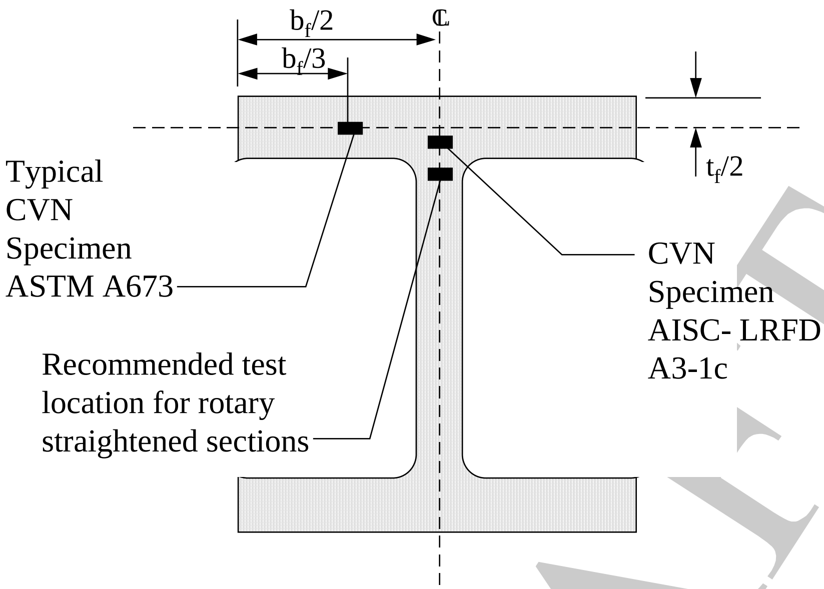


Figure 5-1 - Standard Locations for Charpy V-Notch Specimen Extraction, Longitudinal Only

Commentary: 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 minimum weld metal toughness values of 20 ft-lb. at various 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 value of Charpy toughness obtained from the three specimens must exceed the specified minimum. One of the tested values 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, whichever is greater. 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. Recent tests at Lehigh (Ref.) on new continuous cast shapes and one old shape recovered from a 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.

5.2.5 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 though out repair detailing and weld procedure specifications (WPS) should be prepared in advance.

5.3 Welding

5.3.1 Welding Process

Applicable welding processes for structural steel 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 or service conditions, some restrictions on weld processes or parameters may be appropriate and should be stated in the contract documents.

5.3.2 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.

5.3.3 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 CJP tension applications where transvere loading or applied 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°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.

5.3.4 Preheat and Interpass Temperatures

The preheat temperatures and conditions given in AWS D1.1, Chapter 3 should be strictly observed with special attention given to Section 3.2, for the thickness of metal to be welded. For repair welding of earthquake damage, the *AASHTO/AWS D1.5 Bridge Welding Code*, Chapter 12 preheat requirements for fracture-critical 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 cooling 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.

ANSI/AASHTO/AWS D1.5 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 5-6 - ANSI/AASHTO/AWS D1.5 Preheat Requirements for Fracture Critical Repairs

Steel	*Thickness, in.*	*Minimum Preheat/Interpass Temp., °F*

A36/A572	*to 1-1/2*	*325*
A36/A572	*>1-1/2*	*375*

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.

5.3.5 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.

5.3.6 Controlled Cooling

Most of the weldment cooling occurs 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 advantage.

5.3.7 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 5.29 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."

5.4 Bolting

Structural bolts employed in connections of MRSFs should conform to one of the standard types indicated in Table 5-7 and to the applicable requirements of the ASTM specifications.

Table 5-7 - 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	

5.5 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.*

5.5.1 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.

5.5.2 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.

5.5.3 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.

5.5.4 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.

5.5.5 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 5-2. 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.

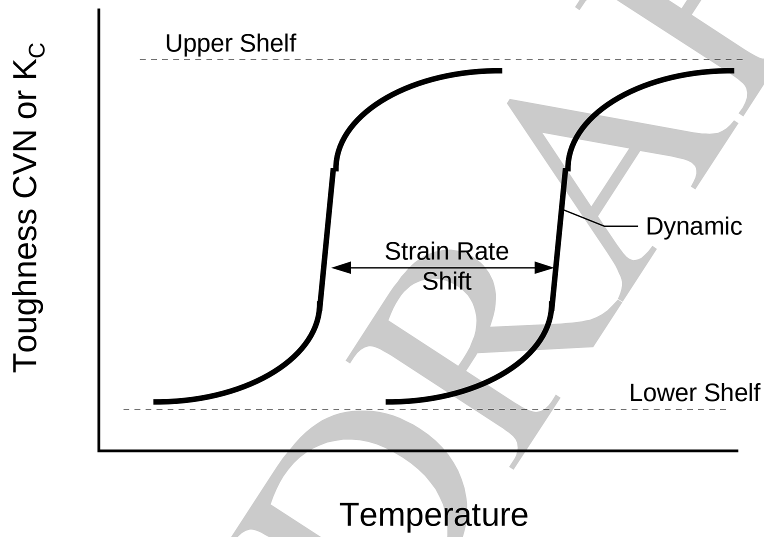


Figure 5-2 - Schematic Relationship Between Notch Toughness, Temperature and Strain Rate

5.5.6 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:

$$K_{ID} = \sqrt{5E(CVN)} / 1000$$

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.

5.5.7 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.

5.5.8 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.

5.6 Connections Conducive to Brittle Fracture

5.6.1 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.

5.6.2 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.

6. STRUCTURAL SPECIFICATIONS

6.1 Scope

This section provides guidelines for development of those divisions of construction specifications related to the fabrication and erection of structural steel for MRSF 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.

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. These specifications are for guidance only. Only the paragraphs which are emboldened are specifically recommended these guidelines. The language or similar language defining the same concepts of the emboldened sections should be incorporated in each firm's standard specification.

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.

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. **A992 - Standard Specification for Steel for Structural Shapes for Use in Building Framing**

    Commentary: ASTM A913 Grades 50 and 65 are now accepted for seismic use as columns 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**

   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.

3. 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, 1998 Edition.
2. D1.4 - Structural Welding Code - Reinforcing Steel, 1998 Edition.

D. ICBO - International Conference of Building Officials

1. UBC - Uniform Building Code, 1997 Edition.

E. SPC - Society for Protective Coatings, "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 Sections: 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.

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

C. 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.

Commentary: The Interim Guidelines, FEMA-267 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, pre-tensioned 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 manufacturer's 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.**

Commentary: Section 5.2..2 commentary notes "The analysis and reporting of an expanded set of elements should be possible, and could be beneficial in the preparation of welding procedure specifications (WPS's) by the welding engineer if critical welding parameters are required."

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

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

4. **Heavy Shapes: Charpy V-Notch**

5. *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 personnel 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. Welders who have not used the welding process for a period of six or more months shall be requalified.

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

4. 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."

Commentary: The Interim Guidelines, section 9.1.1 recommends that such a conference be held "to plan and discuss the project and fabrication procedures."

D. 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 inspector's 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.

Commentary: The Interim Guidelines 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 rejected.

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: ASTM A992

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. For steel produced by the continuous casting process, samples may be taken at random.

Commentary: The above is recommended in section 8.1.4 of the Interim Guidelines.

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 or B, bolts with ASTM A563 hex nuts.

I. High Strength Bolts:

1. ASTM A325, type 1 or type 3; unless indicated otherwise.

2. ASTM A490 where designated on Drawings.

3. Nuts and washers conforming to RCSC.

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

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

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

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

4. Nuts: ASTM A563 or A194, heavy hex.

K. Anchor Rods (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.

Commentary: The Interim Guidelines 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 45ºF 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.

Commentary: Although not specifically evaluated by testing conducted under the SAC project, 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.

D. Assembly with High Strength Bolts

1. Construct connections in accordance with RCSC, using provisions for pre-tensioned joints, unless snug-tight bolts are indicated on Drawings.

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

E. 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.

F. Welded Construction

1. Examine fit-up of joint for conformance with welding procedure specification.

2. Weld in accordance with AISC Specification andAWS 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, if required, 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. Heavy Shapes - Complete penetration groove weld 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.

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 approved procedures.

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

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

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

I. Headed Studs: Automatically weld in accordance with AWS D1.1, Section 7, 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 galvanized faying surfaces of slip-critical high strength bolted connections to achieve Class C surface in accordance with RCSC.

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 RCSC.

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

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) Observe preheat and interpass temperatures, and weld pass sequence.

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 "K-Area" as described in section 8.1.6 of the IG Advisory.*

9. Ultrasonically inspect base metal thicker than 1-1/2 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 rods.

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 except as permitted by RCSC specifications.

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