INTERIM GUIDELINES: Evaluation, Repair, Modification and Design of Steel Moment Frames

Report No. SAC-95-02

SAC Joint Venture

a partnership of:
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Applied Technology Council
California Universities for Research in Earthquake Engineering

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SAC Program to Reduce Earthquake Hazards in Steel Moment Resisting Frame Structures

Foreword and Disclaimer

The purpose of this document is to provide engineers and building officials with guidance on engineering procedures for evaluation, repair, modification and design of welded steel moment frame structures, to reduce the risks associated with earthquake-induced damage. The recommendations were developed by practicing engineers based on professional judgment and experience and a preliminary program of laboratory, field and analytical research. This preliminary research, known as the SAC Phase 1 program, commenced in November, 1994 and continued through the publication of these Interim Guidelines. Independent review and guidance was provided by an advisory panel comprised of experts from industry, practice and academia. Every reasonable effort has been made to assure the efficacy of the Interim Guidelines contained herein. However, users are cautioned that research into the behavior of these structures is continuing. The results of this research may invalidate or suggest the need for modification of recommendations contained herein. **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.** Such information must be used together with sound engineering judgment when applied to specific engineering projects. These Interim Guidelines have been developed by the SAC Joint Venture with funding provided by the Federal Emergency Management Agency, under contract number EMW-95-K-4672.

Acknowledgment

The SAC Joint Venture wishes to offer grateful acknowledgment to the Federal Emergency Management Agency (FEMA); FEMA's project officer, Mr. Michael Mahoney; and technical advisor, Dr. Robert D. Hanson. Following the discovery of severe damage to steel moment-resisting frame buildings in the Northridge Earthquake, this agency recognized the significance of this issue to the engineering community as well as the public at large, and acted rapidly to provide the necessary funding to allow these Interim Guidelines to be developed, published and distributed. Without the support of this agency, the important information and material presented herein could not have been made available.

SAC also wishes to recognize the American Institute of Steel Construction, the American Iron and Steel Institute, the American Welding Society, the California Office of Emergency Services, the Lincoln Electric Company, the Structural Shape Producers Council, and the many engineers, fabricators, inspectors and researchers who contributed services, materials, data and invaluable advice and assistance in the production of this document.

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OVERVIEW

The Northridge Earthquake of January 17, 1994, dramatically demonstrated that the prequalified, welded beam-to-column moment connection used for Special Moment Resisting Frames is much more susceptible to damage than was previously thought. The stability of moment frame structures in earthquakes is dependent on the capacity of the beam-column connection to remain intact and to resist tendencies to rotate, induced by the swaying of the building. These connections were believed to be ductile and capable of withstanding repeated cycles of large inelastic deformation. Although many affected connections were not damaged, a wide spectrum of unexpected brittle connection damage did occur, ranging from minor cracking observable only by detailed nondestructive testing (NDT) to completely severed columns. The most commonly observed damage occurred at the welds of girder bottom flanges to columns. Complete brittle fractures of the girder flange to column connections occurred in some cases. While no casualties or collapses occurred as a result of these connection failures, and some welded steel moment frame (WSMF) buildings were not damaged, the incidence of damage was sufficiently high in regions of strong motion to cause wide-spread concern by structural engineers and building officials.

No comprehensive tabulation is yet available to determine how many steel buildings were damaged in the Northridge Earthquake. More than 100 damaged buildings have been identified so far, including hospitals and other health care facilities, government, civic and private offices, cultural facilities, residential structures, and commercial and industrial buildings. The effect of these observations has been a loss of confidence in the procedures used in the past to design and construct welded connections in steel moment frames, and a concern that structures incorporating these connections may not be adequately safe.

It must be understood that the structural engineering community was surprised by the performance of these modern, code conforming structures. Prior to the discovery of this damage, many thought that WSMF structures were nearly invulnerable to earthquake damage. The unexpected brittle fracturing and attendant loss of connection strength resulted in serious degradation of the overall lateral-load-resisting capability of some affected buildings. Further, the ability of existing WSMF buildings to withstand earthquake-induced ground motion is now understood to be significantly less than that previously assumed. Research conducted to date has identified some, but probably not all, of the factors leading to this observed unsatisfactory behavior. At the same time, this research has indicated methods that can be used to improve the ability of these critical connections to more reliably withstand multiple, large, inelastic cycles. These include alterations in the basic design approach as well as improved practices for specification and control of materials and workmanship.

While the work is not yet complete, and future research is likely to provide both more reliable and more economical methods of improving the performance of these structures, the current investigations have led to many design and retrofit measures that can be used today to provide more reliable and consistent performance of these buildings than occurred in the Northridge

Earthquake. These are presented in these Interim Guidelines. They should not, however, be viewed as the only way of achieving these results, and the exercise of independent engineering judgment and alternative rational analytical approaches should be considered. It is anticipated that additional studies, planned by SAC and others, will lead to further improvements in our understanding of the problems, ability to predict probable earthquake performance and methods to design and construct more reliable structures.

There are many complex issues involved in the evaluation, repair, modification and design of WSMF buildings for reliable earthquake performance. These include considerations of metallurgy, welding, fracture mechanics, systems behavior, and basic issues related to fabrication and erection practice. Much remains to be learned in each of these areas. Engineers not familiar with the issues involved are cautioned to obtain qualified advice and third party review when contemplating design decisions that represent significant departures from these Interim Guidelines.

The current judgment given in these Interim Guidelines is that the historic practices used for the design and construction of WSMF connections do not provide adequate levels of building reliability and safety and should not continue to be used in the construction of new buildings intended to resist earthquake ground shaking through inelastic behavior. The risk to public safety associated with the continued use of existing WSMF buildings is probably no greater than that associated with many other types of existing buildings with known seismic vulnerabilities, which are not currently the subject of mandatory seismic rehabilitation programs. The earthquake risk of WSMF buildings, in general, may be evaluated in accordance with the following general principles:

1. The historic practices and designs used for WSMF connections are no longer appropriate for design and construction of new steel buildings likely to experience large inelastic demands from earthquakes. Until research is completed, and better information becomes available, the procedures contained in these Interim Guidelines for the design of new buildings should be used in their place. The use of alternative systems, including bolted construction, braced construction, and moment-resisting frames incorporating partially restrained (PR) joints could also be considered, but are not directly addressed by these Interim Guidelines.

2. As a class, existing undamaged WSMF buildings appear to have a lower risk of collapse than many other types of buildings with known seismic vulnerabilities, the performance of which is currently implicitly accepted. Consequently, mandated or emergency programs to upgrade the performance of these buildings does not appear necessary to achieve levels of life safety protection currently tolerated by society. However, the risk of collapse is definitely greater than previously thought. Individual owners should be made aware of the increased level of seismic risk and encouraged to perform modifications to provide more reliable seismic performance, particularly in building housing many persons, or in critical occupancies.

3. Following strong earthquake-induced ground shaking, WSMF buildings incorporating the vulnerable welded moment-resisting connections should be subjected to rigorous evaluations to determine the extent and implications of any damage sustained. These Interim Guidelines may be used to determine which buildings should be evaluated, and for developing an appropriate program to perform such evaluations.

4. Structural repair and modification programs for damaged WSMF buildings should consider the seismic risk inherent in the building including the local seismicity, site geologic conditions, the building's individual construction characteristics, intended occupancy and the costs associated with alternative actions. The Interim Guidelines provided in this document for repair can restore a building's pre-earthquake seismic resistance, but not significantly improve its original levels of safety or reduce the inherent seismic risk. The Interim Guidelines provided in this document for structural modification (upgrading) can be used both to improve building safety and reduce seismic risk. Except in those cases where regulation sets minimum acceptable standards for repair, the ultimate responsibility for deciding whether a building should be modified for improved performance lies with the building owner. It is the structural engineer's responsibility to provide the owner with sufficient information upon which to base a decision. The following may be considered by engineers to provide such information:

    a) When a WSMF has experienced damage to only a few of its moment-resisting connections this damage should be repaired in an expeditious manner. Repair to the original configuration, with proper materials and workmanship, will essentially restore the structure's original earthquake-resisting capacity. However, it will not result in any significant improvement in the building's future performance. The fact that the building experienced only light damage should not be considered a demonstration that the building has a high degree of earthquake resistance and in future earthquakes either more or less damage may be experienced, depending on the particular characteristics of the event.

       Connections which have been damaged can be economically modified at the same time that repairs are made. However, in buildings where damage is limited, modification of the few damaged connections will not result in any significant improvement in the future earthquake performance of the building. Modification of connections throughout the structure, or provision of an alternative lateral force resisting system should be considered as a method of substantially improving probable building performance; however, this will entail a significant cost premium over the basic repair project.

    b) When a WSMF has experienced damage to a significant percentage of its moment-resisting connections (on the order of 25% in any direction of resistance), in addition to repair, consideration should be given to modifying the configuration of the individual damaged connections and possibly some or all of the undamaged connections to provide improved performance in the future. Modification of only some connections, and not others, may cause an increase in vulnerability, due to unbalanced concentrations of stiffness and strength. Therefore, such partial modifications should be made with due consideration of the effect on overall system behavior. Repair and/or modification should be completed expeditiously by structural engineers who are experienced in the design of WSMF buildings and understand the features which caused the observed damage.

c) When a WSMF building has had many seriously damaged connections (on the order of 50% in direction of resistance), owners should be informed that this damage may have highlighted basic deficiencies in the existing structural system, or a geologic feature which unusually amplifies site motion. In such cases the existing system should be both repaired and modified to provide an acceptably reliable structural system. Modifications may consist either of local reinforcement of individual connections and/or alteration of the structure's basic lateral-force-resisting system. Such modifications could include addition of braced frames, shear walls, energy dissipation devices, base isolation and similar measures.

These principles are for regular buildings that have good characteristics of design, materials, and construction workmanship. Buildings with clear and apparent seismic deficiencies pose substantial life safety hazards regardless of the type of structural system employed, or material type. Such deficiencies include incomplete load paths, incompatible structural systems, irregular configurations such as soft or weak stories or torsional irregularity, and improper construction practices. Any such deficiencies found in a WSMF should be corrected.

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1. INTRODUCTION

These Interim Guidelines apply to welded steel moment frame (WSMF) structures subject to large inelastic demands from earthquakes. They provide recommended methods for: determining which buildings should be subjected to detailed post-earthquake evaluations; developing a program for post-earthquake visual and non-destructive inspections of buildings suspected to have damage; evaluating the effect of discovered damage on residual building safety; identifying appropriate strategies for continued occupancy, structural repair and/or modification of damaged buildings; and designing and constructing new buildings. These recommendations are based on an initial, Phase 1, program of research that included collection and analysis of data on buildings damaged by the Northridge Earthquake; detailed structural analyses of damaged and undamaged buildings; review of past literature on relevant research; and laboratory testing of large-scale connection assemblies. They were developed by a group of researchers and practicing engineers, with assistance and consultation from experts in metallurgy, fracture mechanics, welding, design, structural steel production, fabrication erection and inspection.

A significant body of valuable information is presented in these Interim Guidelines, which can be used today to provide improved reliability in welded steel moment frame structures. However, much additional research remains to be performed. The parameters controlling the performance of welded moment resisting connections are not yet fully understood, nor has consensus been obtained on all recommendations contained herein. Engineers engaged in the design of WSMF structures are advised to be watchful for new developments in the future.

Although portions of this document are written in code-like language, it is not a building code, nor is it intended to be used as such. Rather, it is intended to provide engineers and building officials with information on what is known at the present time with regard to these structures, and to provide a series of recommendations that can be used on an interim basis to assist in practice. The use of these Interim Guidelines is not intended to serve as a substitute for the application of informed engineering judgment, nor should they be used to prevent the application of such judgment in particular engineering applications.

1.1 Purpose

These Interim Guidelines have been prepared by the SAC Joint Venture to provide practicing engineers and building officials with:

- understanding of the types of damage buildings incorporating fully restrained (FR) welded steel moment frame (WSMF) connections may experience in strong earthquakes, and the potential implications of such damage;
- a methodology for post-earthquake inspection of existing WSMF buildings, to determine if significant structural damage has occurred;

- an approach for characterizing the relative severity of damage to a WSMF and to determine appropriate occupancy and repair strategies;
- methods of repair for fractured, yielded and buckled elements in WSMF buildings and structures;
- design approaches for modifications to existing WSMF buildings and structures with FR connections to improve performance in future earthquakes; and
- design approaches for connections in new WSMF buildings and structures for improved performance in future strong earthquakes.

Earlier publications by the SAC Joint Venture on this topic include a series of three Design Advisories and the proceedings of an International Workshop (SAC-1994-1). The International Workshop, held in October, 1994 was attended by more than 100 invited researchers, practicing engineers, representatives of industry, and government agencies, and provided an initial focus to the investigations of fractures sustained by welded steel moment-resisting buildings in the Northridge Earthquake. Design Advisory No. 1 (SAC-1994-2) and Design Advisory No. 2 (SAC-1994-3) contained collections of papers and topical reports prepared by practicing engineers, building officials, industry groups and researchers, suggesting factors which contributed to the observed damage, methods of repairing damage and designing new structures to avoid such damage in the future. Design Advisory No. 3 (SAC-1995) categorized the information presented in the previous advisories into a series of discrete engineering issues and presented the consensus opinions of a panel of practicing engineers, researchers and industry representatives with regard to appropriate response to these issues. Dissenting opinions and commentary were also provided as were specific recommendations for directed research required to provide resolution to a number of these issues.

These Interim Guidelines provide specific engineering recommendations based on the results of an initial limited program of research. This research included evaluation of the characteristics of ground motion experienced throughout the Los Angeles area during the Northridge Earthquake, projection of potential ground motions resulting from future earthquakes in this region, analytical investigation of both damaged and undamaged structures affected by the Northridge Earthquake for their response to a range of ground motions, laboratory testing of representative beam-column connections in undamaged, damaged, repaired, and reinforced states, parametric studies on the effects of strain rate and toughness on connection performance, surveys of engineers and building owners to collect data on the extent of damage sustained in the Northridge Earthquake, and statistical evaluation of the data collected and engineering analysis of all of the above.

1.2 Scope

These Interim Guidelines are applicable to steel moment-resisting frame structures incorporating fully restrained connections in which the girder flanges are welded to the columns and which are subject to significant inelastic demands from strong earthquake ground shaking. Recommendations are provided with regard to:

- Designation of buildings to be inspected following an earthquake producing strong ground motion;
- Scope of inspection for buildings so designated;
- Appropriate types of repairs for damaged buildings;
- Methods to modify buildings to reduce the probability of connection fracture damage in future earthquake events;
- Design of new Special Moment Resisting Frame (SMRF) buildings for seismic resistance;
- Design of new Ordinary Moment Resisting Frame (OMRF) buildings located in *Uniform Building Code* (UBC) Seismic Zones 3 and 4 {*National Earthquake Hazards Reduction Program* (NEHRP) Map Areas 6 and 7}; and
- Quality Assurance and Control in the repair, modification and construction of WSMF buildings.

Commentary: The design recommendations contained in these Interim Guidelines are generally applicable to SMRF structures designed for earthquake resistance and to those OMRF structures located within UBC Seismic Zones 3 and 4 {NEHRP Map Areas 6 and 7}. The recommendations should be considered for the design of any welded steel moment frame structure that is desired to have a high degree of reliability for resisting earthquake induced forces. In particular, they should be considered for buildings occupied by a large number of people. Chapter 7 provides further guidelines on this applicability.

1.3 Background

Following the January 17, 1994 Northridge, California Earthquake, more than 100 steel buildings with welded moment-resisting frames 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 are spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few such buildings were located on sites that experienced the strongest ground shaking, damage to these buildings was quite severe. Discovery of these extensive connection fractures, often with little associated architectural damage to the buildings, has been alarming. The discovery has also caused some concern that similar, but undiscovered damage may have occurred in other buildings affected by past earthquakes. Indeed, there are isolated reports of such damage. In particular, a publicly owned building at Big Bear Lake is known to have been damaged by the Landers-Big Bear, California sequence of earthquakes, and at least one building, under construction in Oakland, California at the time of the 1989 Loma Prieta Earthquake, was reported to have experienced such damage.

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

The basic intent of the earthquake resistive design provisions contained in the building codes is to protect the public safety, however, there is also an intent to control damage. The developers of the building code provisions have explicitly set forth three specific performance goals for buildings designed and constructed to the code provisions (SEAOC - 1990). These are to provide buildings with the capacity to

- resist minor earthquake ground motion without damage;
- resist moderate earthquake ground motion without structural damage but possibly some nonstructural damage; and
- resist major levels of earthquake ground motion, having an intensity equal to the strongest either experienced or forecast for the building site, without collapse, but possibly with some structural as well as nonstructural damage.

In general, WSMF buildings in the Northridge Earthquake met the basic intent of the building codes, to protect life safety. However, many of these buildings experienced significant damage that could be viewed as failing to meet the intended performance goals with respect to damage control. Further, some members of the engineering profession (SEAOC - 1995b) and government agencies (Seismic Safety Commission - 1995) have stated that even these performance goals, are inadequate for society's current needs.

WSMF buildings are designed to resist earthquake ground shaking, based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation consists of plastic rotations developing within the beams, at their connections to the columns, and is theoretically capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, not brittle fractures. Based on this presumed behavior, building codes require a minimum lateral design strength for WSMF structures that is approximately 1/8 that which would be required for the structure to remain fully elastic. Supplemental provisions within the building code, intended to control the amount of interstory drift sustained by these flexible frame buildings, typically result in structures which are substantially stronger than this minimum requirement and in zones of moderate seismicity, substantial overstrength may be present to accommodate wind and gravity load design conditions. In zones of high seismicity, most such structures designed to minimum code criteria will not start to exhibit plastic behavior until ground motions are experienced that are 1/3 to 1/2 the severity anticipated as a design basis. This design approach has been developed based on historical precedent, the observation of steel building performance in past earthquakes, and limited research that has included laboratory testing of beam-column models, albeit with mixed results, and non-linear analytical studies.

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 indicates just one of these potential fracture growth patterns. Investigators initially identified a number of factors which may have contributed to the initiation of fractures at the weld root including: notch effects created by the backing bar which was commonly left in place following joint completion; sub-standard welding that included excessive porosity and slag inclusions as well as incomplete fusion; and potentially, pre-earthquake fractures resulting from initial shrinkage of the highly restrained weld during cool-down. Such problems could be minimized in future construction, with the application of appropriate welding procedures and more careful exercise of quality control during the construction process. However, it is now known that these were not the only causes of the fractures which occurred.

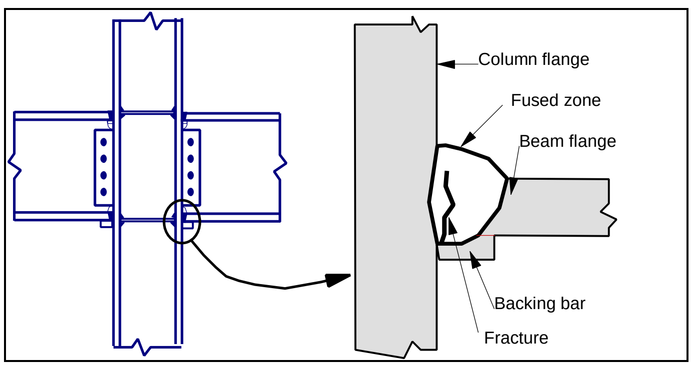


Figure 1-1 - Common Zone of Fracture Initiation in Beam -Column Connection

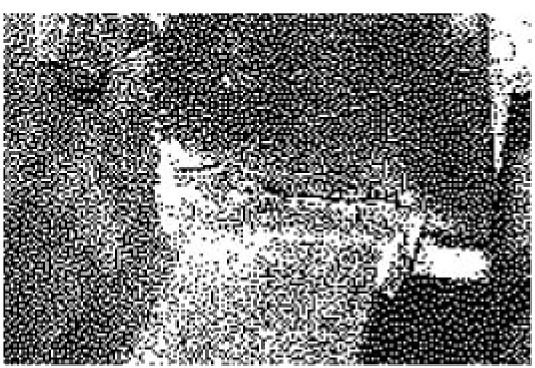
Current production processes for structural steel shapes result in inconsistent strength and deformation capacities for the material in the through-thickness direction. Non-metallic inclusions in the material, together with anisotropic properties introduced by the rolling process can lead to lamellar weakness in the material. Further, the distribution of stress across the girder flange, at the connection to the column is not uniform. Even in connections stiffened by continuity plates across the panel zone, significantly higher stresses tend to occur at the center of the flange, where the column web produces a local stiffness concentration. Large secondary stresses are also induced into the girder flange to column flange joint by kinking of the column flanges resulting from shear deformation of the column panel zone.

The dynamic loading experienced by the moment-resisting connections in earthquakes is characterized by high strain tension-compression cycling. Bridge engineers have long recognized that the dynamic loading associated with bridges necessitates different connection details in order to provide improved fatigue resistance, as compared to traditional building design that is subject to “static” loading due to gravity and wind loads. While the nature of the dynamic loads resulting from earthquakes is somewhat different than the high cycle dynamic loads for which fatigue-prone structures are designed, similar detailing may be desirable for buildings subject to seismic loading.

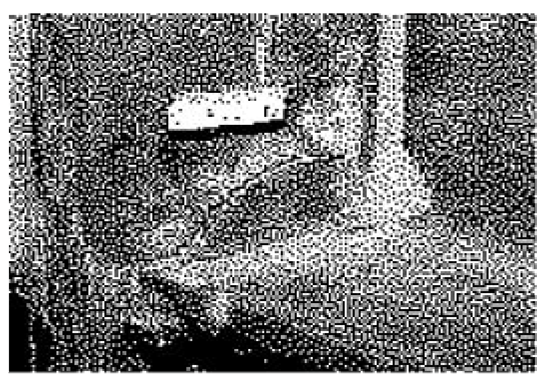
In design and construction practice for welded steel bridges, mechanical and metallurgical notches should be avoided because they may be the initiators of fatigue cracking. As fatigue cracks grow under repetitive loading, a critical crack size may be reached whereupon the material toughness (which is a function of temperature) may be unable to resist the onset of brittle (unstable) crack growth. The beam-to-column connections in WSMF buildings are comparable to category C or D bridge details that have a reduced allowable stress range as opposed to category B details for which special metallurgical, inspection and testing requirements are applied. The rapid rate of loading imposed by seismic events, and the complete inelastic range of tension-compression-tension loading applied to these connections is much more severe than typical bridge loading applications. The mechanical and metallurgical notches or stress risers created by the beam-column weld joints are a logical point for fracture problems to initiate. This, coupled with the tri-axial restraint provided by the beam web and the column flange, is a recipe for brittle fracture.

During the Northridge Earthquake, once fractures initiated in beam-column joints, they progressed in a number of different ways. In some cases, the fractures initiated but did not grow, and could not be detected by visual observation. In other cases, the fractures progressed completely through the thickness of the weld, and if fireproofing was removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-2a). Other fracture patterns also developed. In some cases, the fracture developed into a through-thickness failure of the column flange material behind the CJP weld (Figure 1-2b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

A number of fractures progressed completely through the column flange, along a near horizontal plane that aligns approximately with the beam lower flange (Figure 1-3a). 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.

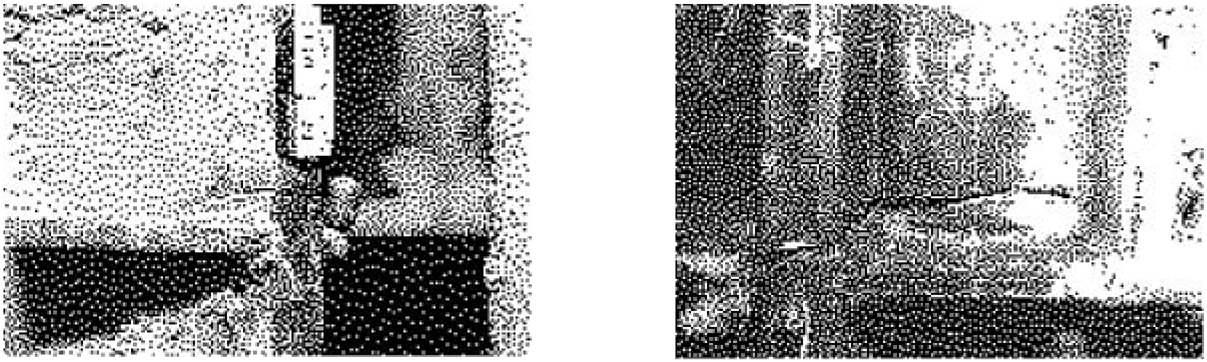


a. Fracture at Fused Zone



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 these fractures have occurred, the beam - column connection has experienced a significant loss of flexural rigidity and capacity. 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. Initial research suggests that residual stiffness is approximately 20% of that of the undamaged connection and that residual strength varies from 10% to 40% of the undamaged capacity, when loading results in tensile stress normal to the fracture plane. When loading produces compression across the fracture plane, much of the original strength and stiffness remain. However, in providing this residual strength and stiffness, the beam shear connections can themselves be subject to failures, consisting of fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-4).

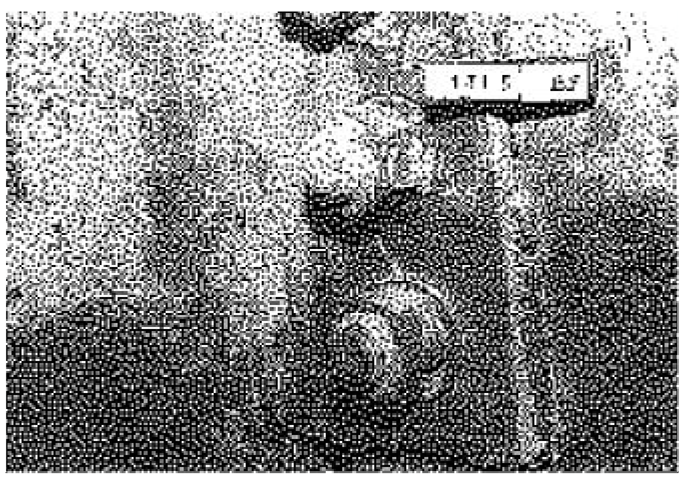


Figure 1-4 - Vertical Fracture through Beam Shear Plate Connection

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 extreme damage to architectural elements. Until news of the discovery of connection fractures in some buildings began to spread through the engineering community, it was relatively common for engineers to perform cursory post-earthquake evaluations of WSMF buildings and declare that they were undamaged. In order to reliably determine if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing and perform nondestructive examination including visual inspection and ultrasonic testing. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. A few WSMF buildings have sustained so much connection damage that it has been deemed more practical to demolish the structures rather than to repair them.

Immediately following the Northridge Earthquake, a series of tests of beam-column subassemblies were performed at the University of Texas at Austin, under funding provided by the AISC as well as private sources. The test specimens used heavy W14 column sections and deep (W36) beam sections commonly employed in some California construction. Initial specimens were fabricated using the standard prequalified connection specified by the *Uniform Building Code* (*UBC*). Section 2211.7.1.2 of *UBC-94* {*NEHRP-91* Section 10.10.2.3} specified this prequalified connection as follows:

> **"2211.7.1.2 Connection strength**. The girder top column connection may be considered to be adequate to develop the flexural strength of the girder if it conforms to the following:
>
> 1. the flanges have full penetration butt welds to the columns.
>
> 2. the girder web to column connection shall be capable of resisting the girder shear determined for the combination of gravity loads and the seismic shear forces which result from compliance with Section 2211.7.2.1. This connection strength need not exceed that required to develop gravity loads plus $3(R_w/8)$ times the girder shear resulting from the prescribed seismic forces.
>
> Where the flexural strength of the girder flanges is greater than 70 percent of the flexural strength of the entire section, (i.e. $bt_f/(d-t_f)F_y>0.7Z_xF_y$) the web connection may be made by means of welding or high-strength bolting.
>
> For girders not meeting the criteria in the paragraph above, the girder web-to-column connection shall be made by means of welding the web directly or through shear tabs to the column. That welding shall have a strength capable of developing at least 20 percent of the flexural strength of the girder web. The girder shear shall be resisted by means of additional welds or friction-type slip-critical high strength bolts or both.
>
> and:
>
> **2211.7.2.1 Strength**. The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint..."

In order to investigate the effects that backing bars and weld tabs had on connection performance, these were removed from the specimens prior to testing. Despite these precautions, the test specimens failed at very low levels of plastic loading. Following these tests at the University of Texas at Austin, reviews of literature on historic tests of these connection types indicated a significant failure rate in past tests as well, although these had often been ascribed to poor quality in the specimen fabrication. It was concluded that the prequalified connection, specified by the building code, was fundamentally flawed and should not be used for new construction in the future.

In retrospect, this conclusion may have been premature. When the first test specimens for that series were fabricated, the welder failed to follow the intended welding procedures. Further, no special precautions were taken to assure that the materials incorporated in the work had specified toughness. Some engineers, with knowledge of fracture mechanics, have suggested that if materials with adequate toughness are used, and welding procedures are carefully specified and followed, adequate reliability can be obtained from the traditional connection details. Others believe that the conditions of high tri-axial restraint present in the beam flange to column flange joint (Blodgett - 1995) would prevent ductile behavior of these joints regardless of the procedure used to make the welds. Further they point to the important influence of the relative yield and tensile strengths of beam and column materials, and other variables, that can affect connection behavior. To date, there has not been sufficient research conducted to resolve this issue.

In reaction to the University of Texas tests as well as the widespread damage discovered following the Northridge Earthquake, and the urging of the California Seismic Safety Commission, in September, 1994 the International Conference of Building Officials (ICBO) adopted an emergency code change to the 1994 edition of the *Uniform Building Code* (*UBC*-94) {1994 *NEHRP Recommended Provisions Section 5.2}*. This code change, jointly developed by the Structural Engineers Association of California, AISI and ICBO staff, deleted the prequalified connection and substituted the following in its place:

> "**2211.7.1.2 Connection Strength**. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotation and develop the strength criteria in Section 2211.7.1.1 considering the effect of steel overstrength and strain hardening."
>
> "**2211.7.1.1 Required strength**. The girder-to-column connection shall be adequate to develop the lesser of the following:
>
> 1. The strength of the girder in flexure.
>
> 2. The moment corresponding to development of the panel zone shear strength as determined from formula 11-1."

Unfortunately, neither the required "inelastic rotation", or calculation and test procedures are well defined by these code provisions. Design Advisory No. 3 (SAC-1995) included an Interim Recommendation (SEAOC-1995) that attempted to clarify the intent of this code change, and the preferred methods of design in the interim period until additional research could be performed and reliable acceptance criteria for designs re-established. The State of California similarly published a joint Interpretation of Regulations (DSA-OSHPD - 1994) indicating the interpretation of the current code requirements which would be enforced by the state for construction under its control. This applied only to the construction of schools and hospitals in the State of California. The intent of these Interim

Guidelines is to supplement these previously published documents and to provide updated recommendations based on the results of the limited directed research performed to date.

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

The SAC Joint Venture developed a two phase program to solve the problem posed by the discovery of fractured steel moment connections following the Northridge Earthquake. Phase 1 of this program was intended to provide guidelines for the immediate post-Northridge problems of identifying damage in affected buildings and repairing this damage. In addition, Phase 1 included dissemination of the available design information to the professional community. It included convocation of a series of workshops and symposiums to define the problem; development and publication of a series of Design Advisories (SAC-1994-1, SAC-1994-2, SAC-1995); limited statistical data collection, analytical evaluation of buildings and laboratory research; and the preparation of these Interim Guidelines. Phase 2 will consist 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 WSMF connections for new buildings, as well as reliable retrofitting concepts for existing undamaged WSMF structures.

The SAC Joint Venture's unique capability to combine the efforts of researchers, industry representatives, code writers and practicing structural engineers is being applied to all major tasks.

In addition, a Technical Oversight Committee and Technical Advisory Board with nationwide membership from the engineering, research and steel construction communities has been established to oversee the input of information, quality of technical investigations, and development of recommendations, and to assist in disseminating the information obtained.

1.5 Sponsors

Funding for the Phase 1 SAC Steel Program was provided by the California Office of Emergency Services and the Federal Emergency Management Agency. 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 as well as the American Iron and Steel Institute, the American Institute of Steel Construction, the Structural Shape Producers Council, the American Welding Society and the Lincoln Electric Company for the contribution of technical advice and assistance as well as material directly used in the research program. Acknowledgment is also made of the many engineers, fabricators, inspectors and researchers who contributed services and data for use in the development of these Guidelines.

1.6 Summary of Phase 1 Research

These Interim Guidelines are based on the material presented in Design Advisory No. 3 (SAC-1995), professional judgment and experience, a review of past relevant research, concurrent research being performed under grants provided by the National Science Foundation and supplemental information obtained in the SAC Phase 1 research program. This research included:

- Collection of data on buildings damaged by the Northridge Earthquake. This consisted of the collection of detailed information on the configuration and detailing of WSMF buildings damaged by the Northridge Earthquake, together with data on the distribution, type and severity of damage within each structure. This work was conducted as an extension of an earlier survey, performed under funding from the National Institute of Standards and Technology (Youssef, et. al. - 1994). Data on a total of 89 buildings is available from these combined studies (Bonowitz & Youssef - 1995)
- A telephone survey was conducted on a random sample of 200 steel framed buildings located within the zone which experienced estimated ground motion with a peak horizontal acceleration of 0.2g or greater during the Northridge Earthquake. The intent of this survey was to determine the geographic distribution of inspected, damaged and repaired structures in order to correlate damage with ground motion parameters and other factors. (Michael Durkin & Associates - 1995)
- A series of interviews were conducted with engineers, inspectors, building officials and others engaged in the investigation and repair of a number of damaged WSMF buildings. The purpose of these interviews was to collect data on pertinent interpretations or trends noted by engineers and others engaged in this work. (Gates & Morden - 1995)

- Maps of ground motion parameters(peak ground acceleration and pseudo spectral velocity at various periods) were developed for the San Fernando Valley and surrounding areas affected by strong ground motion in the Northridge Earthquake, based on fault rupture and ground motion propagation modeling techniques. Time histories of ground motion were developed for various discrete sites, using these same modeling techniques. These estimated ground motions were developed for use in comparing geographic distributions of damage with ground motion parameters, and as a basis for performing structural analyses of selected buildings. (Sommerville- 1995)

- A fracture model element was developed for use with the DRAIN-2D, non-linear analysis software, to permit analytical simulation of the effect of beam-column connection fractures on overall structural behavior. (Campbell - 1995)

- A series of linear and non-linear structural analyses were performed on eight WSMF buildings which were damaged by the Northridge Earthquake and on two WSMF buildings adjacent to two of these structures, which were not damaged. The purpose of these analyses was to explore the ability of analytical methods to predict the presence of damage within buildings as well as to predict specific locations within buildings where damage is likely to have occurred. In addition, these analyses were intended to indicate threshold demand levels at which damage is likely to have occurred, to provide information on the total demands developed in structures during response to various earthquake ground motions, and to explore the potential for earthquake induced collapse. (Krawinkler, et. al. 1995), (Engelhardt, et. al. - 1995a), (Hart, et. al. - 1995), (Kariotis & Eimani - 1995), (Anderson & Fillippou - 1995), (Naeim, et. al. - 1995), (Uang, et. al. - 1995), (Paret & Sasaki - 1995)

- A series of parametric analytical investigations were performed to assess the influence of various ground motions and structural characteristics on seismic response of WSMF buildings. These included investigations involving hypothesized fractures of beam-column connections for various real and idealized frame structures subject to various intense ground motion records. The consequences of these ground motions were assessed as was the sensitivity of response to vertical ground motion and to various analytical modeling assumptions. (Iwan - 1995), (Hall - 1995), (Hart et. al. - 1995b), (Englehardt, et. al. 1995b), (Krawinkler, et. al. - 1995)

- Four damaged beam-column connections were removed from a WSMF building which was affected by the Northridge Earthquake and subsequently demolished. These specimens were moved to a laboratory and subjected to testing to determine their residual strength and stiffness, for use in making assessments as to the consequences of fracture damage to overall building stability. Following this testing, the specimens were repaired and re-tested, to judge the effectiveness of the repair techniques. In addition, detailed building analyses were performed. (Anderson - 1995)

- A total of 12 large scale beam-column assemblages were fabricated using typical pre-Northridge detailing practice and following correct welding procedures. These were cycled inelastically, using a testing protocol similar to that indicated in ATC-24 (Applied

Technology Council - 1988) and experienced failure at low levels of plastic demand. Following initial testing and failure, the specimens were repaired using specifications followed by engineers in the Los Angeles area, or repaired and reinforced using details proposed by Los Angeles area engineers. The purpose of these tests was to explore whether initial structural capacity could be re-established in damaged structures by common repair techniques, and to determine the efficacy of proposed structural reinforcement techniques. (Popov et. al. - 1995), (Bertero and Whitaker- 1995), (Uang - 1995b), (Engelhardt - 1995c)

- Four large scale beam-column subassemblies were fabricated using selected details recommended in these guidelines for new construction and subjected to cyclic testing to failure.
- A series of acoustic emission recordings were made on the large scale structural assemblages tested in the laboratory to assist in interpretation of the fracture sequence and to explore the ability of acoustic instrumentation techniques to identify damage in WSMF buildings affected by strong ground motion. (Thewalt - 1995), (Engelhardt, et. al. - 1995d)
- A series of ambient vibration tests were performed on damaged buildings in order to determine the ability of low level vibration testing to be used as a method of detecting damage in WSMF buildings affected by strong ground motion, and to calibrate analytical models. (Beck - 1995)
- Specimens from damaged connections in buildings affected by the Northridge Earthquake were removed from the buildings and subjected to metallurgic and fractographic analyses to determine the fracture mechanisms and effect of metallurgy on fracture behavior. (ATLSS - 1995a)
- A series of moderate-scale "T" specimens were fabricated to simulate the connection of a beam bottom flange to a column flange in a major axis WSMF connection. These tests were performed to explore the ability to economically use moderate scale models to explore the behavior of large scale beam-column assemblages and also to perform parametric experimental studies on the effects of strain-rate on specimen behavior and the effects of weld metal notch-toughness and weld procedure on connection behavior. (ATLSS - 1995b)

Additional information was collected from various other sources, including research performed under funding provided by the American Institute of Steel Construction, the National Science Foundation, and the National Institute of Standards and Technology, as well as testing performed as part of privately sponsored research (Allen, et. al. -1995, Jokerst - 1995) and lessons learned in the inspection, evaluation and repair of buildings which has taken place to date.

1.7 Intent

These Interim Guidelines are primarily intended for two different groups of potential users:

a) Engineers engaged in evaluation, repair, and upgrade of existing WSMF buildings and in the design of new WSMF buildings incorporating either Special Moment-Resisting Frames or Ordinary Moment-Resisting Frames utilizing welded beam-column connections. The recommendations for new construction are applicable to all WSMF construction expected to resist earthquake demands through plastic behavior.

b) Regulators and building departments responsible for control of the evaluation, repair, and occupancy of WSMF buildings that have been subjected to strong ground motion and for regulation of the design, construction, and inspection of new WSMF buildings.

The fundamental goal of the information presented in these Interim Guidelines is to help identify and reduce the risks associated with earthquake-induced fractures in WSMF buildings through provision of timely information on how to inspect existing buildings for damage, repair damage if found, upgrade existing buildings and design new buildings. The information presented here primarily addresses the issue of beam-to-column connection integrity under the severe plastic 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.

1.8 Limitations

The information presented in these Interim Guidelines is based on limited research conducted since the Northridge Earthquake, review of past research and the considerable experience and judgment of the professionals engaged by SAC to prepare and review this document. Additional research on such topics as the effect of floor slabs on frame behavior, the effect of weld metal and base metal toughness, the efficacy of various beam-column connection details and the validity of current standard testing protocols for prediction of earthquake performance of structures are planned as part of the Phase 2 program and will likely provide important information not available at the time these Guidelines were formulated. Therefore, some recommendations cited herein may change as a result of forthcoming research results.

Although the information presented is limited almost exclusively to technical engineering issues, it is well recognized that acceptable solutions to the steel WSMF problems must eventually address the non-technical concerns of building officials, owners, tenants, contractors, lenders, insurers, and legislators. It is hoped that by limiting the scope of this document to technical matters, this material can provide an objective basis for further discussion and debate.

The information presented here is based on consideration of the typical building and WSMF frame configurations found in buildings today. Non-building structures (e.g. bridges, towers, or open frameworks) are not specifically addressed; however, to the extent that construction of these structures is similar to that for buildings, the information presented may be applicable. Beams and columns are assumed to be constructed of hot-rolled or built-up wide flange sections with beams framing into the column flange, although some recommendations should also apply to box columns and beams framing to column webs.

The recommendations presented herein represent the group consensus of the committee of Guideline Writers employed by SAC following independent review by a technical advisory panel, Project Oversight Committee and Technical Advisory Board. They may not reflect the individual opinions of any single participant. They do not necessarily represent the opinions of the SAC Joint Venture, the Joint Venture partners, or the sponsoring agencies. Users are cautioned that available information on the nature of the WSMF problem is in a rapid stage of development and any information presented herein must be used with caution and sound engineering judgment.

1.9 Use of the Guidelines

It is anticipated that the users of these Interim Guidelines will generally desire information in one or more of the following specific areas:

1. a general understanding of the performance of WSMF buildings in the Northridge Earthquake and the probable performance of such buildings in future earthquakes;

2. inspection, evaluation and repair of buildings which have been affected by the Northridge Earthquake or other earthquakes;

3. seismic upgrade of existing WSMF buildings to provide more reliable performance in future earthquakes; and

4. design of new WSMF buildings to provide more reliable performance in future earthquakes.

In order to provide information useful to all such users, this document has been made quite broad. Table 1-1 provides a quick reference to the contents of this document.

Table 1-1 - Quick Reference Guide

User Need	Section	Contents
General Information	Chapter 1	Introductory material
	Chapter 2	Abbreviations, Notation & Terminology
	Chapter 3	Damage Classification, Safety Issues, Economic Loss Data
Post-Earthquake Inspection, Evaluation, and Repair	Chapters 1-3	Background Information
	Chapters 4 and 5	Inspection
	Chapter 6	Repair
	Chapter 8	Metallurgy and Welding
	Chapter 9, 10, 11	Inspection & Quality Control
New Building Design	Chapters 1-3	Background Information
	Chapter 7	Design Criteria
	Chapter 8	Metallurgy & Welding
	Chapter 9, 10, 11	Inspection & Quality Control