State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes

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

DISCLAIMER

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

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

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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* and the 1997 *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and other Structures*. ATC is a nonprofit corporation founded to develop structural engineering resources and applications to mitigate the effects of natural and other hazards on the built environment. Since its inception in the early 1970s, ATC has developed the technical basis for the current model national seismic design codes for buildings; the *de facto* national standard for postearthquake safety evaluation of buildings; nationally applicable guidelines and procedures for the identification, evaluation, and rehabilitation of seismically hazardous buildings; and other widely used procedures and data to improve structural engineering practice. CUREe is a nonprofit organization formed to promote and conduct research and educational activities related to earthquake hazard mitigation. CUREe's eight institutional members are the California Institute of Technology, Stanford University, the University of California at Berkeley, the University of California at Davis, the University of California at Irvine, the University of California at Los Angeles, the University of California at San Diego, and the University of Southern California. These laboratory, library, computer and faculty resources are among the most extensive in the United States. The SAC Joint Venture allows these three organizations to combine their extensive and unique resources, augmented by subcontractor universities and organizations from across the nation, into an integrated team of practitioners and researchers, uniquely qualified to solve problems related to the seismic performance of steel moment-frame buildings.

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In Memory of Egor Popov, Professor Emeritus, University of California at Berkeley

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1. INTRODUCTION

1.1 Purpose

This report, FEMA-355E – *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, presents an overview of the development of the welded moment-resisting steel frame system as a preferred system for seismic resistance in the United States and the limited data upon which this reputation was based. This state of the art report was prepared in support of the development of a series of Recommended Design Criteria documents, prepared by the SAC Joint Venture on behalf of the Federal Emergency Management Agency (FEMA) and addressing the issue of the seismic performance of moment-resisting steel frame structures. These publications include:

- *FEMA-350 – Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*. This publication provides recommended criteria, supplemental to FEMA 302 – 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, for the design and construction of steel moment-frame buildings and provides alternative performance-based design criteria.
- *FEMA-351 – Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*. This publication provides recommended methods to evaluate the probable performance of existing steel moment-frame buildings in future earthquakes and to retrofit these buildings for improved performance.
- *FEMA-352 – Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*. This publication provides recommendations for performing postearthquake inspections to detect damage in steel moment-frame buildings following an earthquake, evaluating the damaged buildings to determine their safety in the postearthquake environment, and repairing damaged buildings.
- *FEMA-353 – Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*. This publication provides recommended specifications for the fabrication and erection of steel moment frames for seismic applications. The recommended design criteria contained in the other companion documents are based on the material and workmanship standards contained in this document, which also includes discussion of the basis for the quality control and quality assurance criteria contained in the recommended specifications.

Detailed derivations and explanations of the basis for these design and evaluation recommendations may be found in a series of State of the Art Report documents prepared by the SAC Joint Venture in parallel with these design criteria. These reports include:

- *FEMA-355A – State of the Art Report on Base Metals and Fracture.* This report summarizes current knowledge of the properties of structural steels commonly employed in building construction, and the production and service factors that affect these properties.

- *FEMA-355B – State of the Art Report on Welding and Inspection.* This report summarizes current knowledge of the properties of structural welding commonly employed in building construction, the effect of various welding parameters on these properties, and the effectiveness of various inspection methodologies in characterizing the quality of welded construction.
- *FEMA-355C – State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking*. This report summarizes an extensive series of analytical investigations into the demands induced in steel moment-frame buildings designed to various criteria, when subjected to a range of different ground motions. The behavior of frames constructed with fully restrained, partially restrained and fracture-vulnerable connections is explored for a series of ground motions, including motion anticipated at near-fault and soft-soil sites.
- *FEMA-355D – State of the Art Report on Connection Performance*. This report summarizes the current state of knowledge of the performance of different types of moment-resisting connections under large inelastic deformation demands. It includes information on fully restrained and partially restrained moment connections in welded and bolted configurations, based upon laboratory and analytical investigations.
- *FEMA-355E – State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*. This report summarizes investigations of the performance of steel moment-frame buildings in past earthquakes, including the 1995 Kobe, 1994 Northridge, 1992 Landers, 1992 Big Bear, 1989 Loma Prieta and 1971 San Fernando events.
- *FEMA-355F – State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings.* This report describes the results of investigations into the ability of various analytical techniques, commonly used in design, to predict the performance of steel moment-frame buildings subjected to earthquake ground motion. Also presented is the basis for performance-based evaluation procedures contained in the design criteria documents, FEMA-350, FEMA-351, and FEMA-352.

In addition to the recommended design criteria and the State of the Art Reports, a companion document has been prepared for building owners, local community officials and other non-technical audiences who need to understand this issue. *A Policy Guide to Steel Moment-Frame Construction* (*FEMA 354*), addresses the social, economic, and political issues related to the earthquake performance of steel moment-frame buildings. *FEMA 354* also includes discussion of the relative costs and benefits of implementing the recommended criteria.

1.2 Background

For many years, the basic intent of the building code seismic provisions was to provide buildings with an ability to withstand intense ground shaking without collapse, but potentially with some significant structural damage. In order to accomplish this, one of the basic principles inherent in modern code provisions is to encourage the use of building configurations, structural systems, materials and details that are capable of ductile behavior. A structure is said to behave in a ductile manner if it is capable of withstanding large inelastic deformations without significant degradation in strength, and without the development of instability and collapse. The design forces specified by building codes for particular structural systems are related to the amount of ductility the system is deemed to possess. Generally, the building code allows structural systems with more ductility to be designed for lower forces than less ductile systems, as ductile systems are deemed capable of resisting demands that are significantly greater than their elastic strength limit. Starting in the 1960s, engineers began to regard welded steel moment-frame buildings as being among the most ductile systems contained in the building code. Many engineers believed that steel moment-frame buildings were essentially invulnerable to earthquake-induced structural damage and thought that should such damage occur, it would be limited to ductile yielding of members and connections. Earthquake-induced collapse was not believed possible. Partly as a result of this belief, many large industrial, commercial and institutional structures employing steel moment-frame systems were constructed, particularly in the western United States.

The Northridge earthquake of January 17, 1994 challenged this paradigm. Following that earthquake, a number of steel moment-frame buildings were found to have experienced brittle fractures of beam-to-column connections. The damaged buildings had heights ranging from one story to 26 stories, and a range of ages spanning from buildings as old as 30 years to structures being erected at the time of the earthquake. The damaged buildings were spread over a large geographical area, including sites that experienced only moderate levels of ground shaking. Although relatively few buildings were located on sites that experienced the strongest ground shaking, damage to buildings on these sites was, in many cases, quite extensive. Discovery of these unanticipated brittle fractures of framing connections, often with little associated architectural damage to the buildings, was alarming to all concerned. The discovery also caused some concern that similar, but undiscovered, damage may have occurred in other buildings affected by past earthquakes. Later investigations confirmed such damage in a limited number of buildings affected by the 1992 Landers, 1992 Big Bear and 1989 Loma Prieta earthquakes.

In general, steel moment-frame buildings damaged by the 1994 Northridge earthquake met the basic intent of the building codes. That is, they experienced limited structural damage, but did not collapse. However, the structures did not behave as anticipated and significant economic losses occurred as a result of the connection damage, in some cases, in buildings that had experienced ground shaking less severe than the design level. These losses included direct costs associated with the investigation and repair of this damage as well as indirect losses relating to the temporary, and in a few cases, long-term, loss of use of space within damaged buildings.

Steel moment-frame buildings are designed to resist earthquake ground shaking based on the assumption that they are capable of extensive yielding and plastic deformation, without loss of strength. The intended plastic deformation consists of plastic rotations developing within the beams, at their connections to the columns, and is theoretically capable of resulting in benign dissipation of the earthquake energy delivered to the building. Damage is expected to consist of moderate yielding and localized buckling of the steel elements, not brittle fractures. Based on this presumed behavior, building codes permit steel moment-frame buildings to be designed with a fraction of the strength that would be required to respond to design level earthquake ground shaking in an elastic manner.

Steel moment-frame buildings are anticipated to develop their ductility through the development of yielding in beam-column assemblies at the beam-column connections. This yielding may take the form of plastic hinging in the beams (or less desirably, in the columns), plastic shear deformation in the column panel zones, or through a combination of these mechanisms. It was believed that the typical connection employed in steel moment-frame construction, shown in Figure 1-1, was capable of developing large plastic rotations, on the order of 0.015 to 0.02 radians, without significant strength degradation.

Observation of damage sustained by buildings in the 1994 Northridge earthquake indicated that contrary to the intended behavior, in many cases brittle fractures initiated within the connections at very low levels of plastic demand, and in some cases, while the structures remained essentially elastic. Typically, but not always, fractures initiated at the complete joint penetration (CJP) weld between the beam bottom flange and column flange (Figure 1-2). Once initiated, these fractures progressed along a number of different paths, depending on the individual joint conditions.

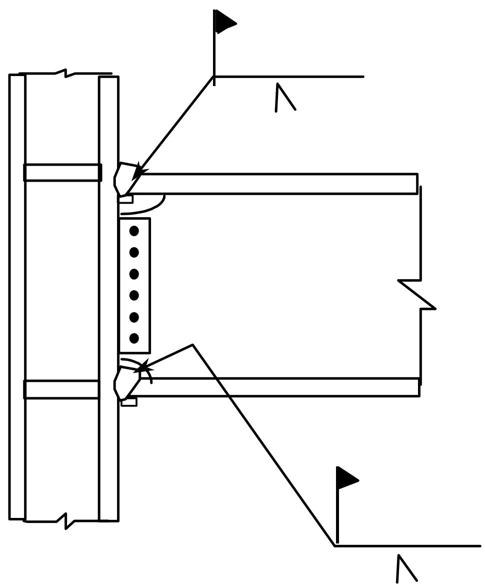


Figure 1-1 Typical Welded Moment-Resisting Connection Prior to 1994

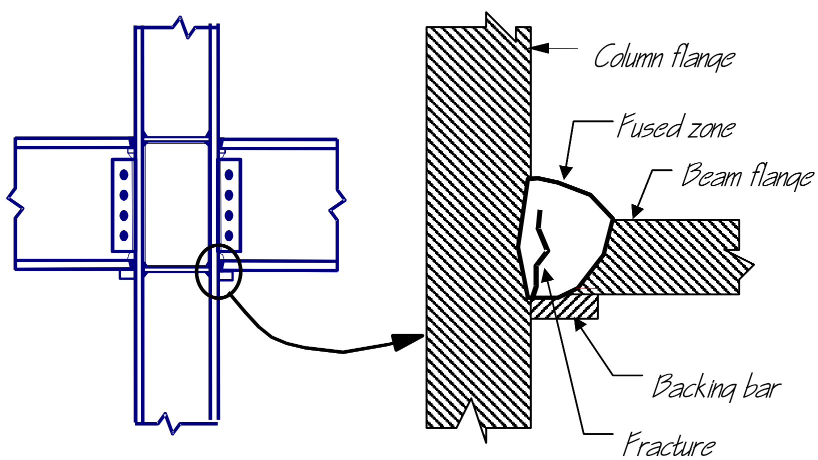


Figure 1-2 Common Zone of Fracture Initiation in Beam-Column Connection

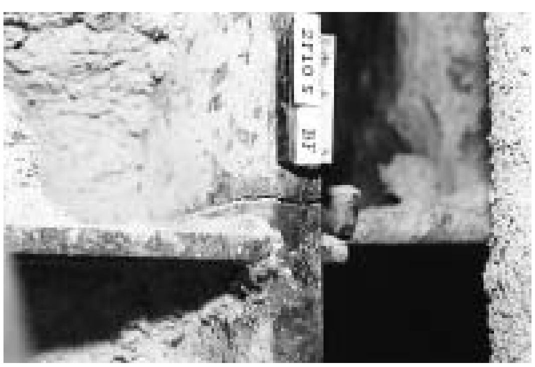
In some cases, the fractures progressed completely through the thickness of the weld, and when fire protective finishes were removed, the fractures were evident as a crack through exposed faces of the weld, or the metal just behind the weld (Figure 1-3a). Other fracture patterns also developed. In some cases, the fracture developed into a crack of the column flange material behind the CJP weld (Figure 1-3b). In these cases, a portion of the column flange remained bonded to the beam flange, but pulled free from the remainder of the column. This fracture pattern has sometimes been termed a “divot” or “nugget” failure.

A number of fractures progressed completely through the column flange, along a near-horizontal plane that aligns approximately with the beam lower flange (Figure 1-4a). In some cases, these fractures extended into the column web and progressed across the panel zone (Figure 1-4b). Investigators have reported some instances where columns fractured entirely across the section.

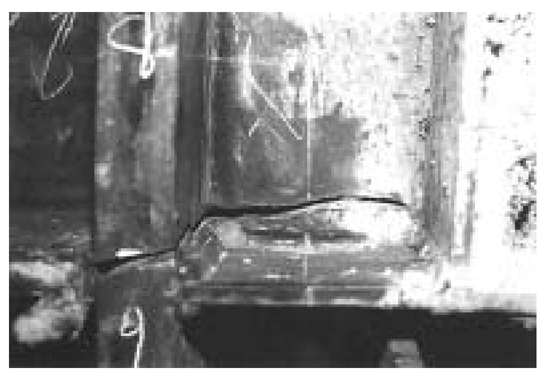
a. Fracture at Fused Zone

b. Column Flange "Divot" Fracture

Figure 1-3 Fractures of Beam-to-Column Joints



a. Fractures through Column Flange



a. Fracture Progresses into Column Web

Figure 1-4 Column Fractures

Once such fractures have occurred, the beam-column connection has experienced a significant loss of flexural rigidity and strength to resist those loads that tend to open the crack. Residual flexural strength and rigidity must be developed through a couple consisting of forces transmitted through the remaining top flange connection and the web bolts. However, in providing this residual strength and stiffness, the bolted web connections can themselves be subject to failures. These include fracturing of the welds of the shear plate to the column, fracturing of supplemental welds to the beam web, or fracturing through the weak section of shear plate aligning with the bolt holes (Figure 1-5).

Despite the obvious local strength impairment resulting from these fractures, many damaged buildings did not display overt signs of structural damage, such as permanent drifts or damage to architectural elements, making reliable postearthquake damage evaluations difficult. In order to determine reliably if a building has sustained connection damage it is necessary to remove architectural finishes and fireproofing, and perform detailed inspections of the connections. Even if no damage is found, this is a costly process. Repair of damaged connections is even more costly. At least one steel moment-frame building sustained so much damage that it was deemed more practical to demolish the building than to repair it.

Figure 1-5 Vertical Fracture through Beam Shear Plate Connection

Initially, the steel construction industry took the lead in investigating the causes of this unanticipated damage and in developing design recommendations. The American Institute of Steel Construction (AISC) convened a special task committee in March, 1994 to collect and disseminate available information on the extent of the problem (AISC, 1994a). In addition, together with a private party engaged in the construction of a major steel building at the time of the earthquake, AISC participated in sponsoring a limited series of tests of alternative connection details at the University of Texas at Austin (AISC, 1994b). The American Welding Society (AWS) also convened a special task group to investigate the extent to which the damage was related to welding practice, and to determine if changes to the welding code were appropriate (AWS, 1995).

In September 1994, the SAC Joint Venture, AISC, the American Iron and Steel Institute and National Institute of Standards and Technology jointly convened an international workshop (SAC, 1994) in Los Angeles to coordinate the efforts of the various participants and to lay the foundation for systematic investigation and resolution of the problem. Following this workshop, FEMA entered into a cooperative agreement with the SAC Joint Venture to perform problem-focused studies of the seismic performance of steel moment-frame buildings and to develop recommendations for professional practice (Phase I of SAC Steel Project). Specifically, these recommendations were intended to address the following: the inspection of earthquake-affected buildings to determine if they had sustained significant damage; the repair of damaged buildings; the upgrade of existing buildings to improve their probable future performance; and the design of new structures to provide reliable seismic performance.

During the first half of 1995, an intensive program of research was conducted to explore more definitively the pertinent issues. This research included literature surveys, data collection on affected structures, statistical evaluation of the collected data, analytical studies of damaged and undamaged buildings, and laboratory testing of a series of full-scale beam-column assemblies representing typical pre-Northridge design and construction practice as well as various repair, upgrade, and alternative design details. The findings of these tasks formed the basis for the development of *FEMA-267 – Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures*, which was published in August, 1995. *FEMA-267* provided the first definitive, albeit interim, recommendations for practice, following the discovery of connection damage in the 1994 Northridge earthquake.

In September 1995, the SAC Joint Venture entered into a contractual agreement with FEMA to conduct Phase II of the SAC Steel Project. Under Phase II, SAC continued its extensive problem-focused study of the performance of moment-resisting steel frames and connections of various configurations, with the ultimate goal of developing reliable seismic design criteria for steel construction. This work has included: extensive analyses of buildings; detailed finite element and fracture mechanics investigations of various connections to identify the effects of connection configuration, material strength, and toughness and weld joint quality on connection behavior; as well as more than 120 full-scale tests of connection assemblies. As a result of these studies, and independent research conducted by others, it is now known that the typical moment-resisting connection detail employed in steel moment-frame construction prior to the 1994 Northridge earthquake, and depicted in Figure 1-1, had a number of features that rendered it inherently susceptible to brittle fracture. These included the following:

- The most severe stresses in the connection assembly occurred where the beam joins to the column. Unfortunately, this is also the weakest location in the assembly. At this location, bending moments and shear forces in the beam must be transferred to the column through the combined action of the welded joints between the beam flanges and column flanges and the shear tab. The combined section properties of these elements, for example the cross sectional area and section modulus, were typically less than those of the connected beam. As a result, stresses were locally intensified at this location.
- The joint between the bottom beam flange and the column flange was typically made as a downhand field weld, often by a welder sitting on top of the beam top flange, in a so-called “wildcat” position. To make the weld from this position, each pass was interrupted at the beam web, with either a start or stop of the weld at this location. Further, the welder often completed all passes on one side of the beam web rather than alternating from one side to the

other as required. This welding technique often resulted in poor quality welding at this critical location, with slag inclusions, lack of fusion, and other defects. These defects can serve as crack initiators, when the connection is subjected to severe stress and strain demands.

- The basic configuration of the connection made it difficult to detect hidden defects at the root of the welded beam-flange-to-column-flange joints. The backing bar, which was typically left in place following weld completion, restricts visual observation of the weld root. Therefore, the primary method of detecting defects in these joints was through the use of ultrasonic testing (UT). However, the geometry of the connection also made it very difficult for UT to detect flaws reliably at the bottom beam flange weld root, particularly at the center of the joint, at the beam web. As a result, many of these welded joints had undetected significant defects that can serve as crack initiators.
- Although typical design models for this connection assume that nearly all beam flexural stresses are transmitted by the flanges and all beam shear forces by the web, in reality, due to boundary conditions imposed by column deformations, the beam flanges at the connection carry a significant amount of the beam shear. This results in significant flexural stresses on the beam flange at the face of the column, and also induces large secondary stresses in the welded joint. Some of the earliest investigations of these stress concentration effects in the welded joint were conducted by Richard, et al. (1995). The stress concentrations resulting from this effect resulted in severe strength demands at the root of the complete joint penetration welds between the beam flanges and column flanges, a region that often includes significant discontinuities and slag inclusions, which are ready crack initiators.
- Weld access holes were needed to complete both the top and bottom flange welds. Depending on their geometry, severe strain concentrations can occur in the beam flange at the toe of these weld access holes. These strain concentrations can result in low-cycle fatigue and the initiation of ductile tearing of the beam flanges after only a few cycles of moderate plastic deformation. Under large plastic flexural demands, these ductile tears can quickly become unstable and propagate across the beam flange.
- The center of the beam-flange-to-column-flange joint is restrained from movement, particularly in connections of heavy sections with thick beam flanges. This condition of restraint inhibits the development of yielding at this location, resulting in locally high stresses on the welded joint, which exacerbates the tendency to initiate fractures at defects in the welded joints.
- Design practice in the period from 1985 to 1994 encouraged connections with relatively weak panel zones. In connections with excessively weak panel zones, inelastic behavior of the assembly is dominated by shear deformation of the panel zone. This panel zone shear deformation results in a local kinking of the column flanges adjacent to the beam-flange-to-column-flange joint, and further increases the stress and strain demands in this sensitive region.

In addition to the above, additional conditions contributed significantly to the vulnerability of connections constructed prior to 1994.

- In the mid-1960s, the construction industry moved to the use of the semi-automatic, self-shielded, flux-cored arc welding process (FCAW-SS) for making the joints of these connections. The specific welding consumables that building erectors most commonly used under this process inherently produced welds with very low notch toughness. The weld quality and notch toughness of this material could be further compromised by excessive deposition rates, which unfortunately were commonly employed by welders. As a result, brittle fractures could initiate in welds with large defects, at stresses approximating the yield strength of the beam steel, precluding the development of ductile behavior.
- Early steel moment frames tended to be highly redundant and nearly every beam-column joint was constructed to behave as part of the lateral-force-resisting system. As a result, member sizes in these early frames were small and much of the early acceptance testing of this typical detail were conducted with specimens constructed of small framing members. As the cost of construction labor increased, the industry found that it was more economical to construct steel moment-frame buildings by moment-connecting a relatively small percentage of the beams and columns and by using larger members for these few moment-connected elements. The amount of strain demand placed on the connection elements of a steel moment frame is related to the span-to-depth ratio of the member. Therefore, as member sizes increased, strain demands on the welded connections also increased, making the connections more susceptible to brittle behavior.
- In the 1980s, many steel mills adopted modern production processes, including the use of scrap-based production. Steels produced by these more modern processes tended to include micro-alloying elements that increased the yield strength of the materials so that despite the common specification of A36 material for beams, many beams actually had yield strengths that approximated or exceeded that required for grade 50 material. As a result of this increase in base metal yield strength, the weld metal in the beam-flange-to-column-flange joints became under-matched, potentially contributing to its vulnerability.

At this time, it is clear that in order to obtain reliable ductile behavior of steel moment-frame construction, a number of changes to past practices in design, materials, fabrication, erection and quality assurance are necessary. The recommendations contained in this document, and the companion publications, are based on an extensive program of research into materials, welding technology, inspection methods, frame system behavior, and laboratory and analytical investigations of different connection details.

1.3 Overview

Dynamic. Engineers know this word describes building behavior in earthquakes. Ground motions impart energy to the elements of a structure, which interact in complex ways. Yielding components redirect forces, resulting in a cascade of change that pushes the structure into a different state. When the elements work together and complement each other, a building can survive an earthquake with little damage. When they do not, the results can be devastating.

“Dynamic” is an appropriate descriptor for the behavior of the engineering and construction communities as well. Powerful forces drive change in one sector, imparting energy and impetus that soon affects the whole industry. Interests are varied, and their interactions complex.

Sometimes they work together, and the result is positive change. Sometimes they compete and the result can be movement in the wrong direction or perpetuation of the status quo.

The history of welded steel moment frame (WSMF) buildings offers a case study of the dynamic nature of the engineering community. There is an interwoven relationship of research, regulation, practice, and, significantly, of nature. A careful study of this structural system and its past earthquake performance traces a path that started in the 1850s, when steel became a mass produced material, and led to unexpected results in the Northridge earthquake.

This report explores some of the forces that cut this path. It also illuminates several events that shaped its direction. It is clear that the two most important of these forces have been “need” and “nature.” Our need to build and improve our homes and workplaces led to the explosion of the metropolis and demanded buildings mass produced in manners unimaginable in previous centuries. The first “skyscrapers” rose over a hundred feet at the turn of the century. The tallest buildings in any city had been churches; they became offices and apartments. New materials and structural systems were needed. Steel alone could meet the need and so became the material of choice for tall buildings.

As steel construction flourished, the steel industry sponsored research and contributed to code development efforts. This sponsorship over the past forty years has allowed researchers to investigate the anticipated seismic performance of WSMF connections. Their overall expectations of good dynamic behavior were enthusiastic. But in many cases, researchers noted that critical conditions at the beam-column interface could lead to premature brittle failures. In retrospect, these concerns deserved more attention than they now appear to have received. All who endorsed steel moment frames for seismic resistance—engineers, inspectors, building officials, contractors, material suppliers, and the researchers themselves—share responsibility for these oversights.

Nature has been more obvious in its impact. No other impetus, whether political, economic or scientific, has had the ability to move the engineering and construction industries forward like an earthquake. Steel construction has probably been the greatest beneficiary of earthquakes’ effects on buildings. After nearly every major event this century the steel building has been hailed as an excellent performer and has been compared with examples of disastrous performance of concrete and masonry structures. This perceived performance led engineers and code writers to encourage the use of steel frames in seismic regions.

Although steel has generally outperformed other structural materials, the WSMF as a seismic force-resisting system developed a glowing reputation that was perhaps undeserved. There has not been conclusive evidence to substantiate the “excellent” performance of modern WSMFs. In fact, there is a decided lack of evidence about the performance of WSMFs prior to 1994. The term “steel frame” was used in numerous postearthquake reports to encompass all manner of steel construction, not just WSMFs. There were only a handful of well-documented examples of WSMF performance before Northridge. This is in part because the number of true WSMFs shaken by earlier earthquakes was small. It is also due to our own tendency to focus on damage that might be more obvious in concrete and masonry buildings and not on the more subtle behavior of steel frames.

Table 1-1 summarizes some of the important milestones in the development of WSMF construction, considering practice, regulation, research, and nature. Each perspective is described in more detail in a separate chapter of this report.

1.4 Approach

In order to understand the performance of any structural system, it is essential to know where that system came from. McGuire (1988) put it nicely: “As in all long-established branches of technology, there is found in current practice a residual influence of decisions made and directions taken long ago, before the underlying sciences were well understood.” Therefore, before reviewing the record of WSMFs in earthquakes, this report presents a brief history of the system. It offers both a review of past practice and an historical context in which to understand current thinking. The report describes what the state of the art in steel frame design has been over the past half century and how that state evolved finally to produce the current SAC Guidelines. Specifically, the report reviews the following:

- The development of WSMF connections, following the transition from iron to steel framing, and tracing the use of riveted, bolted, and ultimately welded connections.
- Research from the past thirty years on the performance and design of WSMF connections.
- Milestones in the development of building code provisions related to WSMF construction.
- The performance of steel moment frame connections in past earthquakes, including Northridge. Summaries of the available raw data from Northridge are included in Appendices A and B. Related data from the 1995 Kobe (Japan) and 1999 Ji-Ji (Taiwan) earthquakes is provided in Appendix C.

There are two ways to present and assess the “past performance” of a given structural system. One is to compare its performance with other systems. The other is to compare its performance with the intentions and expectations of its own proponents. For the most part, this report takes the latter approach, which the authors consider to be more useful, more reliable, and more enlightening.

1.5 Limitations

This report focuses narrowly on topics that may be useful to the users of the SAC Guidelines. The broader topic of steel construction and even of steel moment frame construction has not been addressed in detail, nor is it the intent of this report to make a detailed comparison of steel construction with other structural systems.

Information for this report has been gathered from previously published material and from unpublished test reports and postearthquake data. No original research was performed. The data gathered for this report and contained in the appendices is largely the original data. No effort has been made to synthesize this information into recommendations for design or construction. Researchers are encouraged to review the original data and reports produced by SAC and others, many of which are cited in this report and listed in the References.

Table 1-1 Milestones in U.S. Steel Frame Research, Regulation, Practice, and Performance

Year	Practice	Research	Regulation
1850s	Bessemer process allows mass production of steel.		
1895			First steel material specifications.
1900s	Steel "skyscrapers" reach over 100 feet. Iron phased out in favor of structural steel.		
1906	**San Francisco earthquake** No WSMF structures, but buildings with steel frames perform well.		
1906			San Francisco adopts 30 psf wind and seismic load provisions.
1920s	Welding popular for mechanical equipment.		
1925	**Santa Barbara earthquake** No WSMF structures, but buildings with steel frames perform well compared to those of other materials.		
1927			First seismic provisions written into the *UBC*.
1933	**Long Beach earthquake** No WSMF structures, but buildings with steel frames perform well compared to those of other materials.		
1933			California regulates design of state buildings.
1937			Base shear as functions of soil and height.
1920s–1950s	Masonry infill phased out in seismic regions.		
1948, 1950s	High strength bolts replace rivets.		*UBC* building K factor introduced. Base shear a function of period.

Table 1-1 Milestones in U.S. Steel Frame Research, Regulation, Practice, and Performance (continued)

Early 1960s	Welding in building construction becomes popular.		1959: SEAOC publishes first Blue Book. K factor a function of structural material and system. Steel frame, or its equivalent, required for tall buildings.
1964	**Prince William Sound earthquake** Considerable damage to all material types, including steel. Only a few true WSMF structures, however, and performance is inconclusive.		
Late 1960s	FCAW popular for high production work. Introduction of E70T-4 flux core wire.	AISI sponsored research on moment frames. Potential ductility demonstrated but weld fractures noted.	1968: Blue Book defines K factor for ductile WSMFs and defines properties for ductile steel and concrete systems.
1971	**San Fernando earthquake** Considerable damage to concrete and masonry. Steel perceived to perform well. Only a few true WSMF structures. Brittle fractures repaired in two buildings under construction. Completed buildings not thoroughly inspected.		
1972		Research focuses on panel zone design.	
1975			Panel zone and continuity plate requirements in Blue Book. Web welds recommended.
1980s	Increased WSMF construction. Detailing requirements and computer-aided design lead to use of larger sections and less redundant frames.		
1985	**Mexico City earthquake** Considerable damage to all material types. Some poorly configured steel braced frames perform poorly. Only a few WSMFs. Weld damage is noted but overshadowed by other steel issues.		
1985-1988		W18 and similar sized beams tested. Poor ductility observed. Weld fractures not raised as a major concern.	*UBC* requires supplemental web welds and strong column-weak beam, relaxes panel zone requirements, defines “prequalified” WSMF connection. R_w replaces K factor.

Table 1-1 Milestones in U.S. Steel Frame Research, Regulation, Practice, and Performance (continued)

1989	**Loma Prieta earthquake**		
	Highlights poor performance of URMs and non-ductile concrete. Steel buildings, including WSMFs, perceived to perform well. At least five buildings with connection damage are discovered, most upon inspection after 1994 Northridge earthquake.		
Early 1990s	1992: AISC seismic specifications published.	Research into repair of weld cracks: initial specimens fail at low plastic rotation. Tests of supplemental web welds on large sections: nearly 80% fail at bottom flange welds.	
1994	**Northridge earthquake**		
	Unexpected fractures in some WSMF connections, but none cause death or serious injury. Expected poor performance of outdated systems is realized.		
1994	Engineers await approved details for repair, strengthening, and new construction.	Northridge data collection and testing of alternative WSMF details begin.	ICBO enacts emergency code change requiring cyclic testing of moment frame joint designs.

1.6 Summary

This report reviews the past performance of welded steel moment frames as seismic force-resisting systems. The historical context in which this system developed is described as three interrelated streams of activity: research, regulation, and practice. Each stream at times lagged and at times led the others. All three responded to the redirecting forces of actual earthquakes.

The 1994 Northridge earthquake was a benchmark event for welded steel moment frames (WSMFs). This report offers a post-Northridge view of their use in the United States and their evolution from earlier steel frame systems. From a post-Northridge perspective, some compelling lessons include:

- The WSMF is a young structural system. Its essential components were not in place until about 1970, and it evolved substantially over the next two decades. In Los Angeles County, more structural steel was erected in the 1980s than in the previous two decades combined.
- A quarter century of WSMF connection tests consistently demonstrated the potential for outstanding seismic performance. But a broad view now reveals a pattern of fractures and inconsistent acceptance criteria. Testing could not keep pace with practice, missing key developments in member sizing, material properties, and erection procedures.
- Building codes and standards embraced the connection tests and adopted their results broadly, standardizing the connection detail even for untested sizes and unprecedented frame configurations. Design codes also failed to address reliability issues raised by the tests.

- The modern WSMF had never been tested in meaningful numbers until the Northridge earthquake. It missed the destructive San Fernando earthquake in 1971, and, perhaps because other structural systems fared worse, it was largely unscrutinized after Loma Prieta in 1989.
- Early estimates of WSMF damage in the Northridge earthquake were high. In the end, no WSMFs collapsed, only a handful lost significant lateral capacity, and half of all inspected WSMF buildings had no connection damage at all.

Nevertheless, the Northridge earthquake exposed a faulty detail that performed in the field much as it had in the lab for twenty years. Most of the connections survived. Too many failed. In many ways, this report merely updates a 1991 study of the past performance of steel structures in earthquakes (Yanev et al., 1991). That report captured the pre-Northridge thinking of the entire design and construction community—both the right and wrong of it—in a single short paragraph:

> When failures of steel structures occur, connection failures are the most common cause. No advantage can be derived from the strength and ductility of a steel member if its connections fail prematurely. However, use of industry-standard details generally provides acceptable performance.

2. DESIGN AND CONSTRUCTION OF WSMFs IN SEISMIC AREAS

The history of steel as a modern building construction material traces its evolution in large part to its ease of manufacture and erection. The Bessemer process and the open hearth furnace, developed in the mid 19th century, allowed steel to be mass produced. Fifty years later it was already an essential material for “high-rise” buildings. By proportioning the alloying elements, especially the carbon content, manufacturers could control the strength and ductility of steel to a degree not possible with iron.

This chapter summarizes some milestones in steel frame design and construction. Much of the information presented is taken from FEMA-274 (1997). An excellent brief history of steel frame construction is given by McGuire (1988). The information presented here is not exhaustive, but is intended to give the reader insight into how steel became such a widely used and hailed material. With respect to the use of steel moment frames in seismic areas, this brief history offers a number of lessons in hindsight:

- The growth of cities and the need for denser development required practical structural systems for tall buildings. Steel fit the bill. Good performance of steel buildings relative to masonry ones in earthquakes made steel even more attractive.
- The rigidity provided by large riveted connections in steel frames and the development of curtain wall systems after World War II made the use of brittle infill masonry as a lateral force-resisting element unnecessary.
- The introduction of bolted and later welded connections allowed engineers to distinguish moment and non moment-resisting connections. This allowed the use of fewer but larger sections in discrete plane frames, offering an economic advantage over the more traditional distributed system, but focusing seismic demands on fewer members and connections. The ability to optimize structural systems with computer technology heightened this effect.
- Increased production of steel frame buildings called for faster and more economical welding processes. When higher deposition rates became available, engineers, fabricators, and inspectors adopted them without much regard for weld toughness and appropriate quality control.

Table 2-1 summarizes some of the milestones and developments discussed in this section.

2.1 Cast Iron and Wrought Iron Construction

Cast iron has been used in construction for over two thousand years and was popular in the United States up until about 1900. Cast iron is stronger than wood or masonry and can support taller buildings with relatively slender structural columns. However, cast iron is brittle and unable to resist large bending or tensile demands. Therefore, it was not an ideal material for moment frame structures, nor was it a reliable choice for resisting dynamic loads.

Table 2-1 Milestones in the Development of Structural Steel Buildings

Date	Milestone
1850s	Bessemer process allows mass production of steel
1895	First specification for structural steel
1900s	Iron phased out as structural steel becomes easy to manufacture.
1906, 1925	Performance of steel in earthquakes increases its popularity.
1920s	Welding becomes popular for mechanical equipment manufacture.
1920s – 1950s	Steel frames with masonry infill phased out in seismic regions as steel moment frames are constructed with built-up joint sections and as curtain wall systems are introduced. Moment connections "partially restrained."
1950s	High strength bolts replace rivets in moment frame joints. Connections become more compact. Slip critical connections create "fully restrained" joints.
Early 1960s	Welding in building construction becomes popular. Moment connections become more efficient as beam flanges can be directly welded to columns without cover plates or clips. Webs are typically bolted to shear tabs welded to the columns.
Late 1960s-Early 1970s	Semiautomatic Flux Core Arc Welding (FCAW) becomes popular for high speed, high production work, replacing the slower Shielded Metal Arc Welding (SMAW) in WSMF construction. Lincoln Electric introduces the NS-3M (E70T-4) flux cored wire.
1980s	Steel use increases as building boom in western U.S. demands more efficient systems for large structures. Codes requiring costly doubler and continuity plates encourage the use of "jumbo" column sections with thick webs and flanges.
Mid 1980s	Development of PC based computer-aided design software allows engineers to "tune" the design of WSMFs. Space frames are eliminated and replaced by plane frames, often only one or two bays wide, with larger members.
1994	Northridge earthquake highlights defects in WSMF performance at welded beam-to-column joints. WSMF design in high seismic regions slows as damage is assessed and research is begun.

Wrought iron, used primarily in the U.S. after the 1850s, has a greater ductility than cast iron and so could be used more reliably for beams and other bending elements. This permitted the more efficient construction of frame structures, although the amount of ductility in these elements was still limited. Taller buildings of wrought iron that were designed to resist lateral loads—wind was the main consideration—often used exterior infill walls of unreinforced masonry or hollow clay tile. These acted as shear walls or, more accurately, like braced frames, with the infill acting as a compression strut between beam-column joints. Diagonal iron bars were also commonly used to resist lateral loads.

Wrought iron frames probably did develop some moment frame action by virtue of their connections. It is nearly impossible to create a truly pinned joint that will resist no bending. Riveting the webs or flanges of the beams to the columns with cleats necessarily added some rigidity.

No specific accounts of the earthquake performance of iron frame buildings could be identified for this report, and it is unlikely that present-day engineers would learn much from them about modern construction, as iron is no longer used as a primary building material.

2.2 Transition to Steel

Steel was not typically employed in building construction until the last decades of the 19th century (McGuire, 1988). The first specification for structural steel was published in 1894-95 (FEMA 274, 1997). The lower proportion of carbon in steel versus iron and the use of other alloys give it better ductility and tensile strength. This made steel an ideal material for frame structures, and led to the first skyscrapers over 100 feet in height. The metallurgical properties of steel have continued to change over the past 100 years, with properties such as carbon content, yield/tensile strength ratios, and elongation specified by ASTM to meet evolving needs.

The transition from masonry bearing wall buildings to steel frames began with the construction of infilled frame structures, in which lateral loads were primarily resisted by the masonry. Infill frame buildings are still built in non-seismic regions. The relative ease with which steel shapes could be rolled permitted the use of more complex beam-column connections capable of resisting relatively large moments. This allowed beam spans to increase and permitted the design of true moment frames able to resist cyclic lateral forces in a moderately ductile manner, without infill.

The usefulness of the moment frame was still limited by the strength of its connections. Riveted, and later bolted, connections required heavy plates to join the beam flanges and webs to the column. These plates were usually structural shapes of their own (I-, T- or L-sections). They often created an eccentric load path between the beams and columns, resulting in high stresses. Furthermore, the low ductility of rivets limited the overall capacity to absorb the energy of inelastic cyclic loading. Figure 2-1 shows an example of a riveted connection. Ironically, one advantage of these connections was that configurations dominated by yielding of connection clips or angles in bending actually created a semi ductile mechanism, as long as yielding was kept out of the rivets. Still, moment frames were not typically designed with the intent that the frames would resist large lateral forces. Infill walls and even concrete fireproofing were still considered the primary stiffening elements. Engineers designing for lateral wind loads even through the 1920s considered it “unrealistic and uneconomical—indeed poor engineering—to disregard” the lateral strength and stiffness of infill (McGuire, 1988).

2.3 Evolution to Moment Frames with Bolted Connections

Between the 1920s and 1950s high strength bolts became an alternative to rivets. Much stronger than rivets, high-strength bolts are also faster to install. High-strength bolts permitted very large clamping forces, which led to the development of the slip-critical connection. Slip-critical connections rely on pressure between the mating surfaces, not solely on the shear strength of the bolt itself. Figure 2-2 shows an example of a combination bolted and riveted connection.

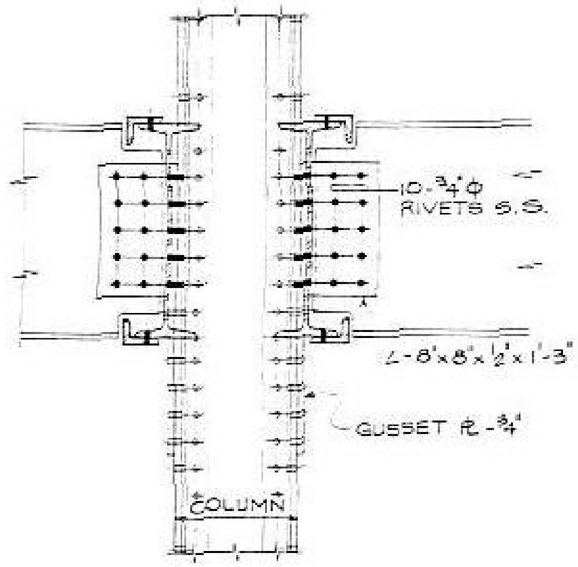


Figure 2-1 Riveted Beam-Column Connection, Pre-1920s

Source: Preece and Collin, 1991

High strength bolts allowed connections to become smaller. Because of the high clamping forces, frame joints also became more rigid, reducing distortion of the frames under lateral loads. The first specification for high strength bolts was available in 1949 (Beedle, 1963), and by 1950 “high strength bolts were being given strong consideration as a replacement for rivets in high-rise buildings” (Preece and Collin, 1991). By 1970, riveting was largely discontinued. All-bolted connections were still somewhat bulky and did not typically achieve a fully restrained connection even though moment frames were being designed to resist larger lateral forces, and masonry infill was being relied upon less.

2.4 Welded Moment Frames and the “Pre-Northridge” Connection

The use of welding, while popularized for steel frame construction mainly in the last forty years, has been in wide use since the 1920s for the mass production of electric motors and other mechanical equipment (Blodgett, 1963). Its use in building construction allowed smaller and more efficient beam-to-column connections. No longer were bent plates or structural sections required to join the members. With welds, the connection could be made purely by fusion of the materials. First used to attach shear tabs to columns, welds eventually formed the beam flange connections as well. This allowed for “fully-restrained” joints that reduced mid-span beam moments under gravity loading and increased building stiffness under lateral loading.

In 1959, AISC researchers studied fully-welded beam-to-column connections under gravity loading to establish force-deflection relationships for use with plastic design methods (see Graham et al. in Table 3-1). They noted that the “direct-welded connection” imposes more severe loads on the column but that it “has certain advantages and may eventually come into more general use.” By 1963, Beedle described welded moment connections as “familiar” and “used extensively,” although the accompanying photograph shows a “top plate” detail, not a directly welded beam flange (Beedle, 1963). Figures 2-3 and 2-4 show examples of welded moment connections.

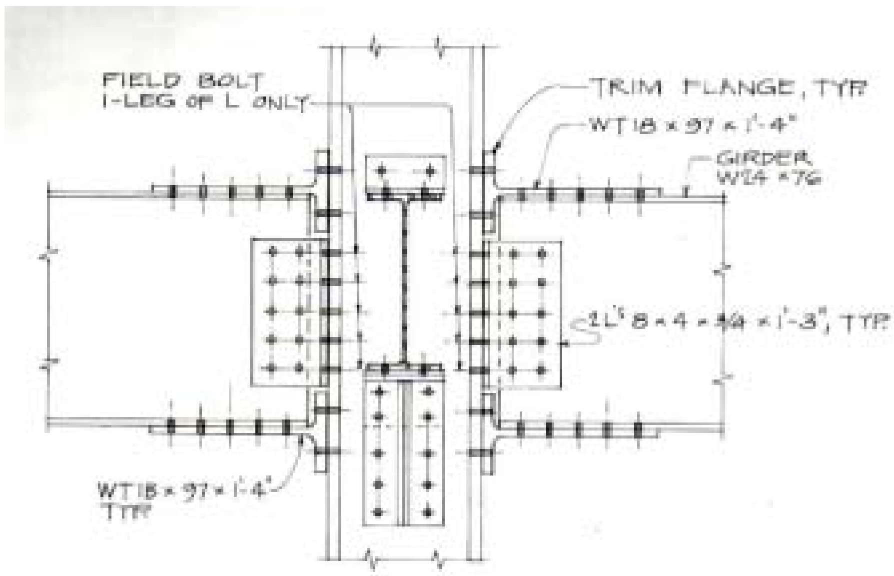


Figure 2-2 Bolted and Riveted Connection, 1930s

Source: Preece and Collin, 1991

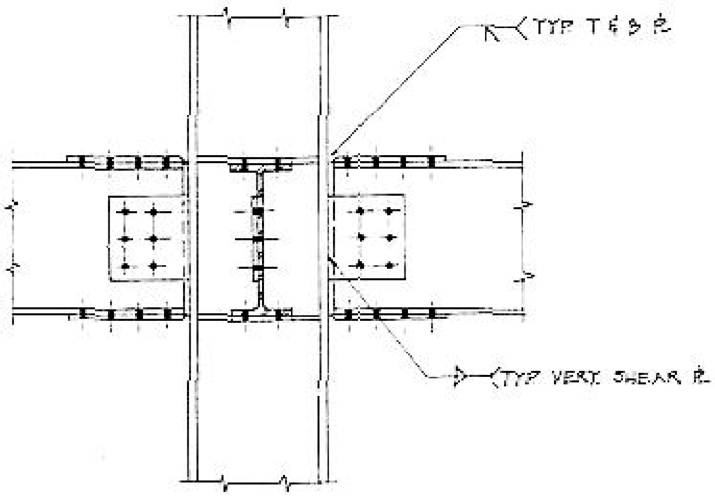


Figure 2-3 Welded and Bolted Moment Connection, 1950-1960

Source: Preece and Collin, 1991

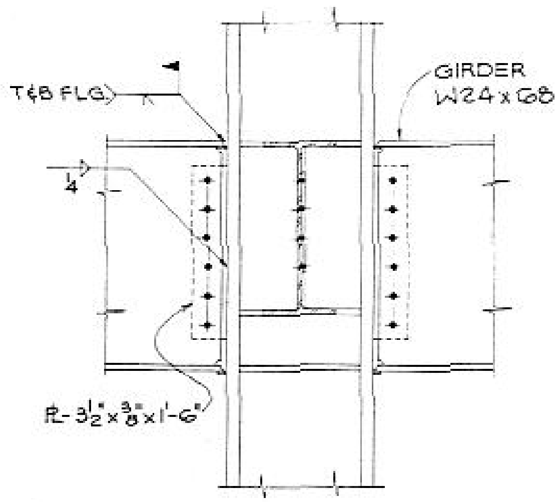


Figure 2-4 Welded Moment Connection, 1980s

Source: Preece and Collin, 1991

By the mid-1970s, the standard connection in California WSMFs (SEAOC, 1975) joined the beam to the column by welding the beam flange and bolting the beam web to a shear tab (Figure 2-4). The alternatives were simply more expensive, as shown in Table 2-2. Elimination of flange continuity plates and use of a bolted shear tab were proposed as ways to reduce internal stresses induced by weld cooling and shrinkage. Daniels and Collin (1972) cited this standard detail as both economical and capable of relieving residual fabrication stresses, but they also cautioned against the use of untested member sizes and restraint conditions.

Table 2-2 Relative Costs of Moment Connections

Beam Flange Connection	**Beam Web Connection**	**Relative Cost**		
		1979	**1983**	**1986**
Full penetration field weld	Bolted to shear tab	1.00	1.00	1.00
Full penetration field weld	Fillet welded to shear tab	1.07	1.07	1.07
Full penetration field weld	Full penetration welded to column	1.25	1.18	1.25
Bolted flange plate (shop welded to column)	Bolted to shear tab	2.00	1.75	2.00

Note: The actual cost, represented by the relative cost of 1.00 in one year is not equal to the actual cost, represented by the relative cost of 1.00 in other years.

Source: Steel Committee of California, 1979, 1983 and 1986

With the popularity of WSMFs on the rise, new welding processes were also developed. The most common and oldest shielded arc welding process is Shielded Metal Arc Welding. SMAW uses an electrode "stick," surrounded by flux, which is melted and fed into the weld area. This has been a very reliable welding method and through the 1980s more people were qualified to perform SMAW than other welding processes (Preece and Collin, 1991).

The self-shielded flux-core process (FCAW-SS), introduced by Lincoln Electric in 1958, eventually proved itself equally versatile and far less costly than SMAW welding. FCAW uses a continuous consumable wire electrode fed by machine into the welding "gun" in a "semiautomatic" process that avoids the starts and stops of stick welding. The nature of flux-core welding also allows for very high deposition rates. By 1967, high deposition "fast-fill" electrodes capable of welding in all positions were available (*Procedure Handbook*, 1973). The faster FCAW processes were quickly adopted for structural steel erection (Cary, 1970), although most fabricators continued to use separate electrodes and equipment for flat and vertical welds (Ferch, 2000).

The construction cost savings relative to SMAW welding were substantial. In 1970, Cary compared SMAW with gas-shielded FCAW and found that SMAW took more than three times as long to complete a 12-inch vertical weld. A 1973 material and labor cost comparison by Lincoln (*Procedure Handbook*) found that FCAW-SS cut the cost of a ¼-inch fillet weld in half relative to stick welding. A 1997 comparison suggested that FCAW total costs per pound of weld metal could be as little as one third those of SMAW (*Fabricators' and Erectors' Guide*). Though economical, the flux-core electrodes most commonly specified for WSMFs in the 1970s and 1980s are now considered to have lacked sufficient fracture resistance (as measured by so-called "notch toughness") for reliable seismic performance. The most commonly used electrode, E70T-4, never promised any notch toughness in its specifications.

Not all FCAW welding is the same. Self-shielded processes are different from gas-shielded FCAW, and even within FCAW-SS there are many different electrode classifications and proprietary products. While some FCAW electrodes provide a specified notch toughness, some do not. Readers are referred to the FEMA Background Reports (FEMA-288, 1997) and other SAC reports (Johnson, 2000) for more on weld processes and their application to WSMFs.

Nevertheless, it is fair to say that FCAW-SS E70T-4 welds are found in the vast majority of field-welded steel moment frames erected in the western United States since 1970 (Goltz and Weinberg, 1998). Figure 2-5 charts the rise in steel construction over the decades of the twentieth century. A California building boom in the early 1980s (Seligson and Eguchi, 1999) put tens of thousands of FCAW-welded-flange, bolted-web connections into service. This economical and ubiquitous detail is now known as the "pre-Northridge" connection.

Post-Northridge building inspections revealed that, at least in Los Angeles, welded connections in the 1980s employed a number of practices that may have contributed to poor performance. Among these were such non-conforming practices as weaving of weld beads, poor fit-up, and improper weld dams.

Backing bars were commonly left in place after completing the beam-to-column groove welds. This practice was in compliance with governing codes and standards, including those of AWS (Ferch, 2000). The practice came under scrutiny after the earthquake because backgouging of the weld root, which requires removal of the backing bar, has been considered necessary to ensure complete fusion between the beam flange, weld, and column flange (Blodgett, 1963). In response to Northridge findings, AWS (1995) would later recommend backing removal and backgouging for WSMFs. FEMA-288 and AWS (1995) offer more on this and related subjects.

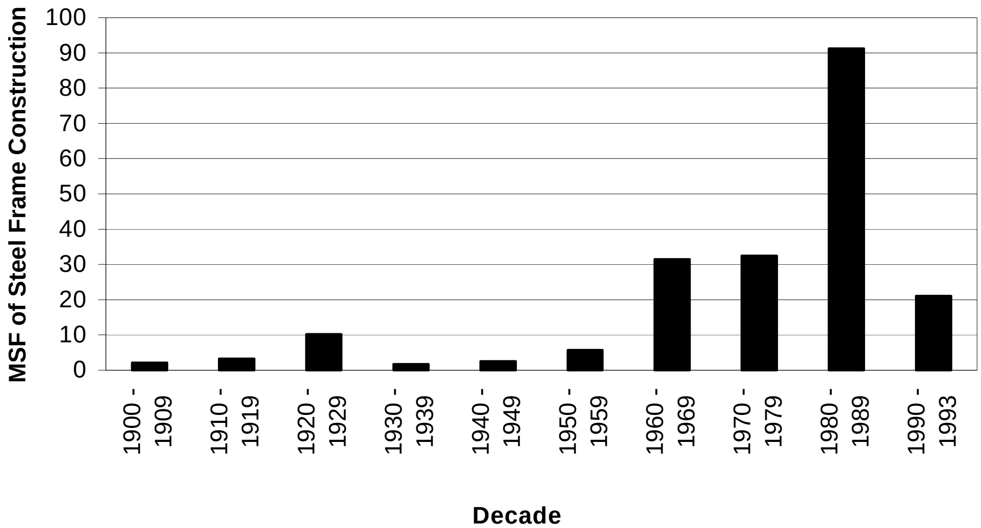


Figure 2-5 Rise in Steel Construction in Los Angeles County

Source: Seligson and Eguchi, 1999

2.5 Optimized Design

In the early 1970s, tall frames were trending toward deep girders of short spans in order to develop more efficient tube-like behavior. The resulting member and weld sizes had never been tested, and available codes did not address potential fabrication and erection problems unique to deep members with thick flanges (Daniels and Collin, 1972).

In the 1980s, expensive detailing discouraged moment connections to the weak axis of wide-flange columns (Krawinkler, 1997). Thus, the three-dimensional frame that engaged nearly all of a building's columns in both directions was replaced in practice by a number of discrete plane frames. Elimination of the "space" frame from the Blue Book in 1988 (Zsutty, 1989) sanctioned the plane frame and encouraged its use.

Steel industry publications encouraged designers to reduce fabrication costs by sizing members, through careful application of code requirements, so as to avoid continuity plates and web doubler plates (United States Steel, 1980; Thornton, 1982). In practice, this preference for a "clean column" resulted in larger column sections with thicker flanges. The resulting frames, however, were cheaper due to reduced fabrication cost. They were also bigger and stronger, so fewer were needed for the building to meet overall stiffness requirements.

Thornton (1992) published examples of the relative savings obtained by avoiding doubler and continuity plates. The effective cost of a doubler and a pair of stiffener plates, including the erection and fabrication costs, is equivalent to about 700 pounds of steel (material cost only).

Even if an average column needed to be about 35% heavier to avoid these plates, the net savings for an average building was demonstrated to be on the order of $1,000 per column.

In extreme cases, these developments led to an "optimal" configuration of one- or two-bay plane frames, with deep beams and heavy columns, spaced around the building perimeter. Given the changes in standard material strengths, weld properties, member sizes, and frame configuration, a typical 1990 WSMF should not be expected to provide the same seismic performance as a 1970 WSMF (Krawinkler, 1997).

Changing trends in building massing also affected WSMF design. As post-Northridge building surveys reveal (see the Appendices and Northridge sections of this report), most of the WSMFs built in greater Los Angeles during the 1980s were not skyscrapers but three- to five-story office buildings. Garreau (1991) described the "laws" that governed design of these semi-urban structures: heights to optimize floor area ratio relative to the cost of a deep foundation, floor plates to maximize capacity of a single central core and to facilitate corporate management, and setbacks to maximize corner offices. More atria, corners, and setbacks mean less opportunity for an uninterrupted space frame. Figure 2-6 shows an example of typical 1980s WSMF architecture.

Discussion of advancement in steel frame design must consider concurrent advancements in the technology that allowed engineers to analyze increasingly complex structural systems with continually greater speed. The personal computer, introduced in the early 1980s, was common in larger design offices by the end of that decade, and ubiquitous by the mid-1990s. Software was developed to suit the new computers (Habibullah, 1984; Wilson, 1984). Pre-processors allowed rapid creation of two- and three-dimensional models of unprecedented complexity. Post-processors summarized the results and checked code requirements. Alternative designs could be studied and refined within hours, not days. The result was a design optimized for code compliance, but not necessarily for performance. The fine-tuned frames met stress and deflection requirements with a minimum of wasted material or extra cost.

But the software did not account for fabrication or construction sequences. Nor did it analyze connections. Even today, analysis programs typically neglect connection details—and not just in steel. Interestingly, this has been the case throughout the history of steel frame analysis and design. From simplified cantilever and portal methods through the plastic design methods standardized in the 1960s, emphasis has always been more on the behavior and proportioning of members than on the essential connections (McGuire, 1988). Steel frame connection sizing and detailing in most of the United States is still left to the fabricator. In California and some other areas, engineers have shown connection details on their own drawings for at least as long as the WSMF has been a viable structural system. But even in California, the WSMF connection was essentially prescriptive by the mid-1970s (SEAOC, 1974), so it is not surprising that engineers and programmers focused on the member and assumed the connection would work.

Figure 2-6 WSMF Building with Setbacks

Source: Engineering News Record, February 23, 1998.

3. TESTING OF STEEL MOMENT-FRAME CONNECTIONS

Over the past thirty years, academic and industry researchers have tested a variety of welded steel moment frame connections to gauge their expected performance under cyclic lateral loads. These efforts have involved both physical testing of beam-column joints and theoretical analytical modeling. This research has been instrumental to the advancement of the WSMF, discussed in the previous section, and to the development of building codes, summarized in the following section.

Much of the research showed the potential pitfalls of welded construction in severe earthquake conditions. Interestingly, the observed failures often were considered insignificant aberrations, or were addressed in the authors' conclusions as avoidable simply through proper field inspection. From Popov's work in 1969 to Engelhardt's in 1993, fractures at welded joints have been shown to limit the ductility of steel moment frames. This summary and review is intended not to criticize the historical research but to draw lessons from the overall pattern that may not have been apparent from individual studies. Indeed, when the first U.S. code provisions for WSMFs were being written in the late 1960s (see the next section of this report), there was evidence that considerable weld defects could be tolerated with no loss of capacity: "[B]uckling is more of a problem than weld defects are in plastic design" (Couch and Olsson, 1965).

The test programs described below involved over a hundred individual beam-column specimens tested over thirty years. With a few exceptions, the published papers reached encouraging conclusions about the expected seismic performance of WSMF beam-column connections. And when caveats or limitations were stated, they read mostly like legal disclaimers, cautioning readers against careless extrapolation. The leading California steel researcher of the time, Egor Popov, would later regret not speaking up: "My flaw was that I wasn't sufficiently loudmouthed about how bad they were" (ENR, February 10, 1997).

In 1993, Engelhardt and Husain took advantage of broad hindsight and sounded a warning. The Northridge earthquake struck a month after their article was published.

This brief description represents most of the important WSMF testing done prior to the Northridge earthquake. Taken as a whole, and viewed from a post-Northridge perspective, the historical testing offers some broad conclusions perhaps not obvious from any of the individual studies:

- Performance can not be usefully gauged unless demand is well-defined. This is an obvious statement today, now that codes and standards explicitly acknowledge inelastic seismic demands, and now that tools to model those demands are available. When the first of these tests were being performed, practicing engineers did not work in inelastic terms.
- The beam-column weld and the weld access hole have always been critical locations for fracture initiation in the pre-Northridge connection. Many of the brittle fracture patterns observed after Northridge had been seen before in the laboratory.

- Researchers and practitioners have consistently attributed failures in the lab to poor construction quality. Viewed with hindsight, the low level of quality appears to be systemic. Fracture of welded joints should perhaps have been a focus of detailed study on its own.
- High reliability was never attained. Variable results were the rule, and premature brittle fractures accounted for some portion of nearly every test program. In statistical terms, a small program can not demonstrate the kind of reliability that engineers expect.
- Engineers extrapolated unreasonably from test results. None of the pre-Northridge research programs tested beams deeper than 24 inches or heavier than 76 lbs/ft, but many of the fractures found after Northridge involved W30 and W36 sections up to 300 lbs/ft. Design "beyond the precedent established by research" was identified as a concern as early as 1972 (Daniels and Collin, 1972).

Figure 3-1 shows an example of a WSMF connection test setup. Table 3-1 lists significant research programs that preceded the 1994 Northridge earthquake.

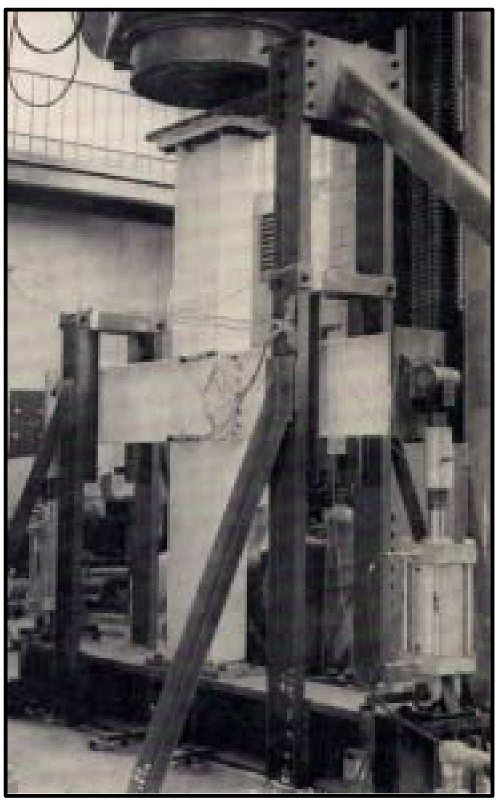


Figure 3-1 WSMF Connection Test Setup

Source: Popov et al., 1985.

Table 3-1 Milestones in Research on WSMF Connections

Ref. Date	Researcher	Research sponsors	# of tests	Description of tests and results
1959	Graham, Sherbourne, and Khabbaz	AISC	13	Monotonic gravity tests on fully-welded specimens with 16WF beams and 8WF or 12WF columns, with emphasis on moment-rotation curves. Did not address lateral loading. Overall program also included 11 direct pull tests to simulate the tension beam flange pulling on the column flange. In all of these, yielding progressed into the column web, and column flanges were visibly bent before cracks developed at the mid-length of the butt weld. As late as the 1989 Ninth Edition, the AISC Specification cited these tests (and Popov and Pinkney, 1969) in support of welded beam flanges.
1965	Bouwkamp & Clough	AISI	NA	In-situ vibration studies of actual steel moment frame buildings to calibrate actual and theoretical period and damping calculations.
	Popov and Franklin	AISI	4	Tested beam-column joints with welded flanges or flange plates (8WF20 beams). All showed good ductility with ultimate failure of welded flanges in weld.
	Beedle	not reported	1	Constructed and tested three story mockup under two cycles of reverse loading.
1965	Bertero and Popov	NSF	10	Tested the ductility and fatigue resistance of 4-inch deep beams without beam-column connections. Identified ductility ratios at beam flange buckling and at beam web tearing. Led to recommendations for compact section requirements.
1969	Popov and Pinkney	AISI	24	Static cyclic tests on a variety of beam-column joints (8WF20 beams). Most achieved good ductility, but plastic rotations were not calculated. Failures in welds at beam-column joints were observed. Warning made to provide high quality joints to avoid premature weld failures. As late as the 1989 Ninth Edition, the AISC Specification cited these tests in support of welded beam flanges.
1972	Bertero, Popov and Krawinkler	AISI	2	Tests focused on panel zone deformation (8WF column, 10B and 14B beams). Results led to code change to require stiff panel zones. Noted that excessive panel zone deformations can lead to kinking at beam flanges and subsequent weld failures.

Table 3-1 Milestones in Research on WSMF Connections (continued)

1972 1973	Popov and Stephen Popov and Bertero	AISI, Hyatt Corp.	8	Evaluated plastic rotation ductility in beam-column joints (W12 column, W18 and W24 beams) with FCAW E70T-4 welds. All specimens achieved plastic rotation of .02 to .06 radians, but four of five with bolted webs failed with abrupt fracture. Authors remark on the overall excellent ductility of the WSMF connection.
1985	Popov, Amin, Louie, and Stephen	Norland Properties, Trade/Arbed, Herrick, SOM	8	Tested one-half size mockups of joints designed for 47-story building with emphasis on panel zone behavior (W18 beams). Of eight tests only two did not fail in a brittle manner and only three exceeded 2% plastic rotation. Authors conclude that the adequacy of the connection has been validated.
1987 1988	Popov and Tsai Tsai and Popov	NSF, AISI	18	Tests evaluated a number of connection details (W18 and W21 beams). Of the eight specimens typical of pre-Northridge practice, only four achieved beam plastic rotations greater than .01 radians. Two worst performers used E70T-4 electrodes made by an inexperienced welder. Authors note that weld and fabrication quality are important factors in overall performance.
1991	Anderson and Linderman	NSF, California Field Ironworkers Trust	15	Focus on repair of nominal weld cracks associated with expected ductile performance. Seven initial specimens tested to failure and eight tests of various repairs (W16 beams). Initial specimens all failed at less than .03 radians total (elastic + plastic) rotation, several with Northridge-type fractures.
1993	Schneider, Roeder and Carpenter	NSF	5	Tested weak column-strong girder joints (W12 and W14 beams). Concluded that high ductility can be achieved and that the anticipated excellent performance of WSMFs is justified.
1993	Engelhardt and Husain	Steel Comm. of CA, Nucor and Bethlehem Steel	8	Focus on supplementary web welds with W21 and W24 beams. Only one of eight tests exceeded 1.5% plastic rotation, with failures initiating in bottom flange welds. Authors conclude that large variability in performance was of much greater concern than the web weld issue.

3.1 Early Testing

Monotonic tests of steel beam-column connections date back to 1917 (McGuire, 1988). Table 3-1 describes similar tests with welded specimens from 1959. Beedle (1963) described a number of monotonic tests of welded, bolted T-stub, and end-plate beam-to-column moment connections. Photographs of the welded specimens indicate fully welded webs and monotonic loading that put the beam bottom flange in compression. Wind loads, but not cyclic or seismic demands, were mentioned. Beedle concluded that steel moment connections of different types could all be expected to provide a ductility factor of 8 to 10. If not, "it is because some detail has been underdesigned" (Beedle, 1963).

Dynamic and cyclic behavior of frame structures was a topic of research at least as far back as 1938, when a shake table test was performed to model a 16-story steel frame and masonry infill building in San Francisco (Jacobsen and Ayre, 1938). The tests were performed to determine the elastic modal properties of the structure, not to predict inelastic performance. The complex model is shown in Figure 3-2.

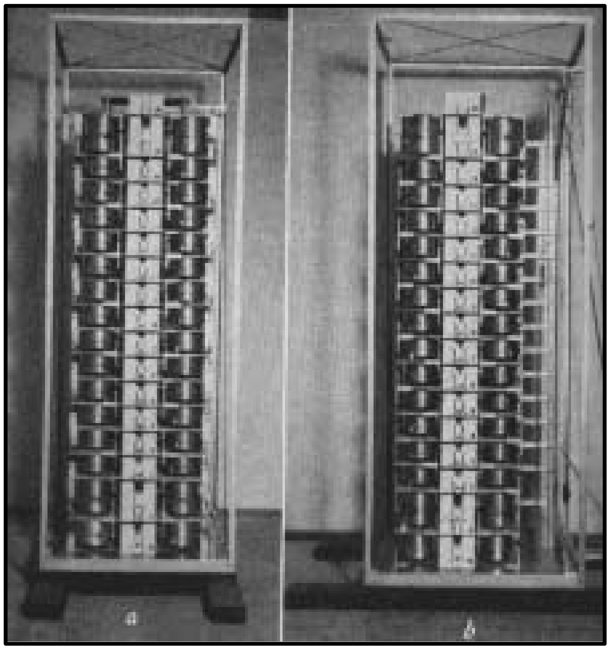


Figure 3-2 Testing Apparatus for Modeling 16-Story Infilled Frame Building

Source: Jacobsen and Ayre, 1938

3.2 Bouwkamp and Clough; Popov and Franklin; Beedle (1965)

These three papers were cited as the basis for the ductile steel-frame provisions introduced in the 1968 Blue Book, and later adopted for the 1970 *Uniform Building Code*. The tests were intended to evaluate steel moment frames under cyclic loading. Bouwkamp and Clough subjected a real building in various stages of construction to induced sinusoidal vibrations to determine the fundamental period and critical damping ratio. Popov and Franklin tested four beam column assemblies, one with girder flanges groove welded to the column flange. The sections were small: 8WF48 columns and 8WF20 beams. Test results led the authors to conclude, "it is possible to expect strains on the order of 1-½% in extreme cases, which corresponds to a fiber ductility factor μ of 12" (Popov and Franklin). Plastic deformation was measured in strain not rotation. All four specimens developed bottom flange local buckling. The authors concluded that the hysteresis loops produced by the tests were remarkably stable and that these connections can be depended on to achieve a "practically constant" amount of strain energy absorption.

Beedle tested a full height two-bay, three-story frame with light beam and column sections (6WF25 columns and 12B16.5 beams). He applied vertical loads to simulate actual gravity loading, then applied horizontal forces to simulate earthquake motion. He was able to apply one or two cycles of reverse loading to obtain a hysteresis curve for the frame. The goal of the study was to see how well the actual stress-strain relationships compared with theoretical modeling. The frames were pushed until the beam compression flanges buckled. The author noted that in the second cycle of loading, the energy absorption increased due possibly to strain hardening and kinematic effects. No study was done to determine the rotation capacity of the connections.

3.3 Popov and Pinkney (1969)

Beginning in 1965, Popov and Pinkney tested 24 specimens, evaluating the total energy absorbed by the system under cyclic, quasi-static loading and measuring the ductility of the elements and joints. Their interest was in “the manner of failure due to exceptionally high loads… and the longevity of a connection under substantial overloads.”

The 8WF20 beam specimens were able to achieve plastic rotations between .046 and .069 radians before failure (Popov and Bertero, 1973). No demand estimates were available for comparison with the measured capacities. The authors acknowledged this, but nevertheless concluded that “such an assemblage is very reliable and can be counted upon to absorb a definite amount of energy in each cycle for a prescribed displacement.” They further estimated that “the number of repeated and reversed loadings which can be safely sustained appears to be in excess of that which may be anticipated in actual service.”

Although the tests showed substantial inelastic capacity, descriptions of the ultimate failure modes are interesting from a post-Northridge perspective. After repeated inelastic excursions, “fracture was frequently in or near the welds, with several failures occurring in the groove welds of the flanges to the column face….” In addition “sharp cornered web copes were a recurring source for initiation of web cracks.”

Popov and Pinkney offered two important warnings. First, “the importance of careful inspection during fabrication was brought out by the premature failure of two improperly welded connections,” and second, “extrapolation to members with … cross sections [other than 8WF20] must be done with caution.” These can be seen as prophetic now that Northridge-type fractures have been associated with poor inspection (Paret, 1998; Kaufmann et al., 1997) and deep beam sections (Roeder and Foutch, 1996; Bonowitz, 1999a), although similar results were later achieved with W18 and W24 beams (Popov and Bertero, 1973).

Also interesting is the authors’ note that in specimens found to have been fabricated with “poor workmanship” and inspected by ultrasonics before testing, “indications produced by the unwelded contact surface were mistakenly interpreted as being due to the back-up bars.” Though noted in 1969, the ambiguity and inconsistency of ultrasonic testing has now been demonstrated by Northridge data (Paret, 1999).

3.4 Bertero, Popov, and Krawinkler (1972)

Through the early 1970s, there were no special requirements in the code for the strength of panel zones. In theory, large panel zone deformations could reduce the gravity load capacity of the column. Bertero, Popov, and Krawinkler tested two subassemblies, one with a panel zone that was relatively weak compared to the beams, and one with a panel zone relatively strong. In general, the authors concluded that "the energy absorption and dissipation capacity… exceeds the required energy, even for cases of extreme earthquakes."

In the latter test "the weak parts of these specimens were clearly the regions of plastic hinges in the beams. This required high rotation capacities of the beams." This is an important statement about ductility demand. In the weak panel zone test, lateral torsional buckling of the beams "prevented a complete stabilization of the hysteresis loops," and testing did not reach failure of either the beam flanges or the welds. (In practice, beams are typically restrained against lateral torsional buckling by a floor slab or by other members.)

An interesting note by the authors was that excessive panel zone deformation led to "kinking" of the column flanges which "precipitated failure of the beam flanges at the welds." The authors therefore recommended strong panel zones "designed for the real plastic capacity of the beams." Recent finite element studies have reached similar conclusions (El-Tawil et al., 1999).

3.5 Popov and Stephen (1972), Popov and Bertero (1973)

In 1973, Popov and Bertero revisited the issue of beam plastic hinging as a desirable mechanism for ductile frames. (Their paper is based on the same test program as Popov and Stephen, 1972.) Here, specific mention of the welding procedures was made. "For all flanges full penetration welds with backup bars were used. Beam webs were coped…. The flux-cored wire E70T-4 was used throughout. All of the structural welds were sonic tested." This is notable considering the current findings that electrodes with no notch toughness requirements (such as E70T-4) perform poorly in pre-Northridge connections (Bonowitz, 1999a).

Popov and Bertero, however, make only passing mention of the possibility of weld failures. Of the seven tests, two "exhibited superior ductile behavior," but "three of the hybrid connections failed prior to the completion of the large hysteresis loops." Failures were typically in the welds. Still, several times the authors describe the "remarkable stability" of the joints. One would assume this describes the joints that did not fail abruptly through the welds. The beam sizes used in the tests ranged from W8x20 to W24x76 sections. The authors noted that there was a "close correlation over so wide a range of beam sizes." Later work would show, however, that similar performance should not be expected over a wide range of member sizes (Roeder and Foutch, 1996; Bonowitz, 1999a).

3.6 Popov, Amin, Louie, and Stephen (1985)

These tests were performed to "verify the design criteria for beam-column joints under extreme seismic loading conditions for a 47-story building in San Francisco." Although the test specimens involved W18 beams, deeper than most specimens previously studied, the building was to use 36-inch deep members.

The authors describe the welding procedure: "the back-up plates for the welds on the beam flange-to-column flange connections were removed after the full-penetration flange welding was completed and small cosmetic welds appeared to have been added and ground off on the underside." It is interesting that the authors considered the fillet welds to be "cosmetic" and not placed to improve performance. While this may have been the case, it is noteworthy because of the post-Northridge recommendations to remove backup bars and to grind and reweld the roots.

Of the eight tests, one failed apparently due to an obvious defect in the preparation. Of the remaining seven, five developed some panel zone inelasticity, then failed abruptly in the welds or in the heat affected zone of the beam. Only two specimens remained ductile through the end of the test.

The authors set an acceptability criterion in terms of total plastic rotation (i.e. beam and panel zone contributions combined), aiming for an "essential" capacity "of at least 2% times a reasonable factor of safety." Of the seven tests, three reached total plastic rotations in excess of 5%. The other four failed at an average of 1.7%. Yet, the authors concluded that "the objectives of verifying the design criteria for the prototype were achieved by this experimental and analytical program." Interestingly, back in 1969, Popov and Pinkney had cautioned that it is important when drawing conclusions about testing to have a "statistically valid number of experiments, with as nearly identical as possible input parameters."

3.7 Popov (1987)

In 1987, Popov reviewed "the state-of-the-art for the design of steel moment connections… for regions of high seismic risk." Looking back over his tests from the early 1970s, he called attention to the "explosive flange failures" that ended many of the tests, and noted that they occurred "only after a number of large cyclic load reversals." Those specimens fractured at plastic rotations of about .02 radians; post-Northridge standards consider this an unacceptable rotation capacity. Despite his sanguine assessment of past tests, Popov would conclude his 1987 review by plainly acknowledging the unknown: "Due to the great uncertainty of the forces that a structure may have to resist during an earthquake, complete reliance on the minimum code provisions is hazardous."

3.8 Popov and Tsai (1987); Tsai and Popov (1988)

In the first paper to offer a strongly worded warning about premature connection fracture, Popov and Tsai presented a wide range of new specimens and evaluated the ductility of WSMFs relative to code provisions. (The 1987 conference paper is a summary of the full research that would be published in 1988.)

The authors noted that the ductility of the various tests was "erratic," with only about half the tests developing "satisfactory" inelastic performance. They concluded that "weld fractures at connections are particularly dangerous." They suggested that careful inspection and fabrication, especially at weld access holes, could reduce the variability of performance.

Instead of discounting a few poorly fabricated specimens, Popov and Tsai also drew attention for the first time to the real implications of welding quality control. Two of their specimens failed, they noted, because the fabricator was unfamiliar with flux-core welding. Another FCAW specimen "was fabricated with exceptional care [that] cannot necessarily be duplicated in the field." (Note that the 1987 paper is incorrect in describing these three specimens as having SMAW welds; the welds were FCAW-SS.)

3.9 Popov, Tsai, and Engelhardt (1988)

This study compared the plastic rotation capacities of the 18 Tsai and Popov (1988) specimens to theoretical demands. The researchers analyzed a hypothetical six-story building subjected to the Parkfield, Mexico City, and El Centro ground motions. Parkfield and Mexico City each imposed plastic rotation demands of about 1.5%. Of the 18 tests, only seven had capacities exceeding this demand by at least 10%. The other eleven either just barely reached 1.5% or failed at under 1% rotation. Six of the eleven were considered to have been poorly fabricated by welders unfamiliar with the specified welding process. The authors conclude that "the experimental data for these connections shows that well fabricated connections are adequate."

The study also concluded that plastic rotation demands in the beams could be notably reduced by making the panel zones more flexible. They cautioned, however, that excessive inelastic panel zone shear deformation and kinking may lead to column instability, and to damage to the beam-to-column groove welds. This latter concern is similar to earlier comments by Bertero, Popov, and Krawinkler (1972).

3.10 Anderson and Linderman (1991)

This study is noteworthy because it explicitly acknowledges that WSMF connections will develop cracks even when they perform as intended. Despite the record of marginal performance through the 1980s, however, brittle fractures were not anticipated. Only a .02 radians total (elastic plus inelastic) rotation demand was imposed. Three of the seven initial specimens barely reached that rotation, and several developed full-width weld or beam flange fractures before the onset of local flange buckling.

3.11 Schneider, Roeder, and Carpenter (1993)

The authors tested four weak column-strong beam joints using very light elements: W12x26-30 columns and W12x16 to W14x26 beams. They concluded that the joints "sustained a major earthquake and exhibited a tremendous amount of ductility. Special moment-resisting steel frames are highly regarded for their seismic performance and the results from these four tests justify this reputation." This enthusiastic endorsement of the WSMF less than a year before the Northridge earthquake reflects little concern even among academics regarding the likely performance of WSMFs.

3.12 Engelhardt and Husain (1993)

Published one month before the Northridge earthquake, this critical paper represented at least two years of study into the seismic performance of WSMFs. The authors tested eight specimens with relatively large members: W12x136 and W18x60 columns, and W21x57 and W24x55 beams. Their primary goal was to evaluate supplemental shear tab welds. Self-shielded flux-core welding was used because it "is frequently used in actual field welding for this connection detail." (E70T-7 electrodes were used. Like E70T-4, the T-7 electrode has no minimum specified notch toughness.) Welds were made in a manner to simulate actual field conditions. Backup bars remained in place.

A plastic rotation of at least 1.5% was sought for each of the tests "as a reasonable estimate of beam plastic rotation demand in steel moment-resisting frames subject to severe earthquakes." Only one of the eight tests reached 1.5% rotation, and all failed in a "fracture at or near a beam flange groove weld." This failure occurred at the beams' bottom flanges. The failures "generally initiated at the edge of the beam flange and gradually spread across the width of the flange as the loading progressed." Once failure occurred, the beam was deflected in the opposite direction to yield the top flange. In all of the tests, the top flange failure occurred not in the weld but in the flange itself, outside the heat affected zone.

The test matrix considered various web connection details and Z_f/Z ratios, but the authors were forced to conclude that "variability in the performance of the beam flange welds appears to have had a much greater influence on plastic rotation capacity than Z_f/Z ratio or web-connection detail." The paper concludes with "concerns about the welded flange-bolted web detail for severe seismic applications" and calls for "a careful review of design and detailing practices."

In a wide-ranging February, 1993 presentation, Popov noted briefly that the work of Engelhardt and Husain "raises the question of reliability of field flange welds" (Popov et al., 1993). He then showed a number of possible reinforced details (with various cover plates, wing plates, and ribs) "for situations where large rotations are anticipated."

Suspecting that incomplete weld fusion undetected by ultrasonic testing might be causing the premature fractures, Engelhardt and Husain also called for review of welding and quality control issues. This suggestion, which appears to have been first made by the authors in a 1991 paper, prompted some criticism by a prominent welding expert. "The assumption that the [flawed] fabrication and inspection of the test specimens was typical of the state of the art in present day structural steel construction is wrong and very much out of line" (Collin, 1992). The events of January, 1994 would show that Engelhardt and Husain had identified problems that were indeed common in current construction (Paret, 1999).

3.13 Roeder and Foutch (1996)

In the wake of Northridge, Roeder and Foutch compiled and analyzed results from various test programs, including most of those summarized above. Using a consistently defined Flexural Ductility Ratio, they found that the inelastic capacity of 91 comparable specimens was highly scattered. Nevertheless, they identified a useful inverse relationship between beam depth and expected ductility. The deeper the section, the lower the inelastic capacity. (Bonowitz, 1999a, found the same relationship when the older results were removed and tests done after Northridge were included.)

FEMA 273 (1997) has incorporated this relationship for use in evaluating existing structures. The implications are profound. Despite the fact that testing from the previous two decades had rarely used beam sections deeper than a W18, many of California's "optimized" steel frames built in the 1980s and early 1990s employed W30 and even W36 beam sections.

3.14 Connection Testing Since Northridge

As described later in this report, codes and standards changed after Northridge to require designs based on comparable cyclic test results. In the six years between the earthquake and publication of the SAC Guidelines, hundreds of full-scale beam-column subassemblies have been tested both in academic studies and in qualification tests for specific building designs. The SAC Joint Venture has compiled over 500 recent test results, many privately funded.

Some of the recent tests involved pre-Northridge details, either as benchmark specimens or as initial specimens to be later repaired and reloaded. Database summaries from late 1998 and mid-1999 confirmed that deeper beams have less capacity and that notch-tough weld metal significantly improves performance of the pre-Northridge detail (Bonowitz, 1999a and 1999b).

4. CODES AND STANDARDS FOR STEEL MOMENT FRAMES

After the 1906 earthquake, San Francisco adopted a 30 pounds per square foot lateral design load for new buildings. The new requirement was intended to account for both wind and earthquake effects (SEAOC, 1968). This was probably the first quantified seismic code provision in the world, even if it accounted only indirectly for a building's key dynamic properties: mass and stiffness. Following the 1925 Santa Barbara earthquake, engineers began to focus on the complex interaction of parameters that affect building performance: structural system and material, period of vibration, soil conditions, etc.

With each earthquake, building codes progress. The observed performance of real buildings—especially poor performance—can have a profound impact on provisions for structural materials and systems. Though changes are sometimes written and adopted slowly even after earthquakes, they frequently take effect before thorough investigations are complete.

For steel moment frames, it was more the *lack* of earthquake damage data that propelled the standards for their design. Until Northridge, WSMF buildings simply did not produce the multiple and repeated failures that force building codes to change. As shown in the next section of this report, that was as much due to their absence as anything else. But without notable failures, seismic code provisions for steel frames developed incrementally, and almost always in ways that would encourage and broaden their use. As a result, WSMF design practice was shaped more by design and construction feasibility than by code limitations.

From a post-Northridge perspective, a review of code provisions and standards for steel frames offers the following lessons:

- WSMF code provisions have developed incrementally, based largely on specific academic research. Since welding became feasible for building structures in the 1960s, provisions have been adjusted to reflect the latest test results.
- While code provisions have been based on research, they have not kept engineers from extrapolating specific research results to untested conditions.
- The lack of real data on the seismic response of WSMFs was perhaps misinterpreted by code writers as evidence of "excellent" performance. This may have contributed to the code's preference for steel moment frame construction.

The preface to the 1927 *Uniform Building Code* thanked the individuals and organizations who contributed to its development: building inspectors, contractors, and engineers, with suggestions from "sales engineers for building materials" (Freeman, 1932). Since then, seismic codes in the U.S. have been written largely by practicing engineers, academics, and building officials who volunteer their time and expertise. And since then, code writers have been assisted by vendors and industry representatives. Proponents of all structural materials, not just steel, have contributed to code development efforts by sponsoring important research and by participating on code-writing committees. As one would expect, these professionals, often engineers, researchers, or contractors themselves, express preferences for their own products and innovations, and support their positions with research results.

A study of the many influences on code provisions is beyond the scope of this report. The building code is, after all, a public policy document. It suffices to note here only that technical, political, and financial interests have sometimes been complementary, sometimes competitive. Code provisions are a synthesis of these interests and frequently represent a series of unavoidable compromises.

Tables 4-1 and 4-2 summarize some of the more important milestones in code development, with emphasis on pre-Northridge steel moment frames. Since 1961, the *UBC* provisions listed in Table 4-2 were based on the SEAOC Blue Book; Blue Book developments are described in the text following the Tables.

Table 4-1 Milestones in Code Development for Steel Moment Frames

Date	Milestone in Code Development
1906	In response to 1906 earthquake, multistory buildings in San Francisco must be designed for 30 psf lateral (wind and seismic) loads.
1927	First seismic provisions written into the *UBC* as an appendix. Seismic loads a function of mass and soil profile only.
1933	In response to 1993 Long Beach earthquake, California passes the Field Act, regulating the design of certain state buildings including schools. California passes the Riley Act, specifying base shear as a function of both soil and building height.
1948, 1950s	*UBC* incorporates the *K* factor to differentiate between buildings and other structures. Base shear becomes a function of period.
1959	SEAOC publishes its first *Recommended Lateral Force Provisions* (Blue Book). After 1961, *UBC* adopts Blue Book recommendations directly into the Code. *K* factor refined to be a function of building material and structural system. Section j favors steel moment-resisting frames.
1968	In response to perceived excellent performance of steel structures, Code defines special *K* factor for ductile moment frames. For steel, defines properties of a ductile frame, including material specifications, strength of girder-column connection, and an inspection program for complete penetration girder welds.
1975	Blue Book further defines characteristics of ductile steel moment frames. Introduces panel zone and continuity plate requirements. Recommends web welds for better performance of girder flange welds.
1985-1988	*UBC* requires that webs be welded if Z_f/Z ratio is less than 0.7. Relaxes panel zone requirements to permit yielding, thereby reducing girder stresses. Requires strong column-weak beam design in most frames. The “prequalified” moment frame connection is included in the *UBC*; alternative details require design for 125% of the girder flexural strength.
1989-1993	*UBC* moves toward strength design in most materials. Lateral force equations are changed to use an R_w factor in place of *K* factor.
1994	In response to Northridge earthquake, ICBO enacts emergency code change requiring cyclic testing of moment frame joint designs.

Table 4-2 *Uniform Building Code* Provisions for Steel Moment Frame Buildings

UBC	**Steel Frame Designation**	**Steel Frame Detail Provisions**
1949, 1952, 1955, 1958	No special designation. Appendix 2312b: F=CW, C constant for all building and bracing types	Appendix 2312d: Plans must include floor load assumptions, "a brief description of the bracing system used, the manner in which the designer expects such system to act, and a clear statement of any assumptions used" including assumed inflection points, and a sample bent calculation.
1961	2313b: **Space Frame – Moment Resisting**. May or may not be enclosed by rigid elements that would prevent sidesway. Table 23-F: K=.8 (frame resists 25%), or .67 (frame resists100%) 2313f: Story drift limits per "accepted engineering practice." 2313j: Buildings taller than 160' must have complete MRSF.	2313j: "The [moment resisting space] frame shall be made of a ductile or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment-resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption."
1964	Same as 1961, but moved to Section 2314.	Same as 1961, but steel chapter is more detailed.
1967	2314b: **Space Frame – Ductile, Moment-Resisting**. Table 23-H: K=.8 (DMRSF in "dual bracing system") or .67 (DMRSF alone) 2314f: Same as 1961. 2314j1: Buildings taller than 160' or with K=.67 or .8 must have DMRSF "of structural steel (complying with Chapter 27) or reinforced concrete (complying with Section 2630 …)."	2314j1: DMRSF "may be enclosed by or adjoined by more rigid elements which would tend to prevent the space frame from resisting lateral forces where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral load-resisting ability of the space frame." 2314j2: "The necessary ductility for a ductile moment-resisting space frame shall be provided by a frame of structural steel conforming to ASTM A7, A36, or A441 with moment-resisting connections." Chapter 27: Nothing system-specific or K-related.

Table 4-2 *Uniform Building Code* Provisions for Steel Moment Frame Buildings (continued)

UBC	**Steel Frame Designation**	**Steel Frame Detail Provisions**
1970, 1973	2314b: Same as 1967. 2314f: Same as 1961. 2314j1: Same as 1967, but steel DMRSF to comply with section 2722.	2314j2: "The necessary ductility for a ductile moment-resisting space frame shall be provided by a frame of structural steel with moment-resisting connections (complying with Section 2722 …)." 2722: **Steel Ductile Moment-Resisting Space Frames, Seismic Zones No. 2 and No. 3** (Note: the most severe seismic zone at this time was Zone 3.) a: Welding to comply with *UBC* Std 27-6. b: Defines joint and connection. c: Defines materials. d: Connections. "Each beam or girder moment connection to a column shall be capable of developing in the beam the full plastic capacity of the beam or girder." Exception granted if "adequately ductile joint displacement capacity is provided." Also, if F_u/F_y<1.5, no plastic hinges allowed where beam flange area is reduced by bolt holes. (Note: A36 would not be subject to this limitation, but A572Gr50 would.) e: Local buckling (b/t limits) f: Slenderness ratios g: Nondestructive Weld Testing. "Tension butt welded connections between primary members of the ductile moment-resisting space frame shall be tested by nondestructive methods for compliance with *UBC* Std No 27-6 and job specifications. A program for this testing shall be established by the person responsible for structural design."
1976, 1979, 1982, 1985	2312b: **Ductile Moment-Resisting Space Frame**, similar to 1967, Section 2314b. 2312h: Story drift (using K = 1.0) limited to 0.005 times story height. 2312j1: Ductility requirements, similar to 1970, Section 2314j1.	2312j1F, similar to 1970, 2314j2. 2722: Same as 1970, but slenderness provision removed, and testing specified: f: Nondestructive Testing. Subject to special inspection per 305. "All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by radiography." For an individual welder, test rate may be dropped to 25% if defect rate after 40 welds is 5% or less. Also, base metal thicker than 1.5 inches subject to through-thickness weld shrinkage strains shall be ultrasonically tested after welding.

Table 4-2 *Uniform Building Code* Provisions for Steel Moment Frame Buildings (continued)

UBC	Steel Frame Designation	Steel Frame Detail Provisions
1988	2312b: **Special moment-resisting space frame (SMRSF)** Table 23-O: R_w=12 2312e.8: Story drift limited to story height times: 0.0033, for buildings up to 65 ft tall 0.0025, for taller buildings.	2722f: **SMRSF Requirements** 1: Scope 2: Girder to Column Connection Required strength: the strength of the girder in flexure, or the moment corresponding to development of the panel zone per equation 22-1. Connection strength: "The girder-to-column connection may be considered to be adequate to develop the flexural strength of the girder if … the flanges have full penetration butt welds to the columns" and the web connection can resist gravity plus seismic shear at the required flexural strength. Supplemental web welds may be required. Alternative details must be designed for 125% of the girder flexural strength. 3: Panel zone must resist gravity plus 1.85 times prescribed seismic forces, but need not exceed 80% of the combined strength of the girders. 4: Flange b/t must be less than $52/\sqrt{F_y}$. 5: Continuity plates to resist flange force of $1.8btF_y$. 6: Strong column requirement.
1991	2331: **Special moment-resisting frame (SMRF)** Similar to 1988.	2710g: **SMRF Requirements**, similar to 1988 2722f.

4.1 1906-1924

Prior to the 1906 San Francisco earthquake, no quantified seismic provisions are known to have existed within building codes. Tall structures were required to meet wind criteria, so some measure of lateral force resistance was provided by most codes. Lateral forces were typically resisted by unreinforced masonry shear wall elements, or by steel girts and braces. After the earthquake, San Francisco placed in its building code a provision that structures be designed to resist earthquake forces as well as wind. The provisions called for the capability to resist 30 psf of lateral pressure (SEAOC, 1968). The provision for earthquakes was only indirectly dependent on building mass as a function of surface area. There were rules governing the design of materials, but no provisions for steel frame design with specific reference to cyclical demands imposed by earthquakes.

Between 1906 and 1925, engineers had begun to understand the effect of building mass on seismically induced inertial forces. They also learned that soil properties affected demands. The 30 psf lateral pressure adopted in San Francisco after 1906 as a surrogate for seismic shear would actually be reduced in the following years to as little as 15 psf by 1926 (Tobriner, 1984). Meanwhile, a 1911 Italian code required design for lateral forces equal to one twelfth of the building weight (Holmes, 1998).

4.2 1925-1932

The Santa Barbara earthquake of 1925 caused widespread damage to structures. Coming in its wake, the 1927 *Uniform Building Code* was the first edition to include specific provisions for earthquake resistant design. It endorsed some fundamental concepts that remain the basis for provisions even today. They included:

- Masses are assumed concentrated at the floors.
- Only permanent dead and live loads are included in the seismic mass.
- The design force at each level is proportional to the level's mass.
- Forces are taken orthogonal to the building's primary axes.
- Stiffness should be symmetric about the center of mass (to control torsion).
- Different lateral forces should be used for different soil conditions.
- Calculations provided by the engineer to the building official should include a summation of the seismic masses, a description of the bracing system and its intended behavior, and a calculation of the stresses on a typical building frame.

The code also specified allowable stresses for different materials subject to earthquake forces. Masonry stresses were limited to 40 psi and reinforced concrete stresses to $0.04f'_c$. For gravity load design, the allowable stresses in concrete beams with or without stirrups was $0.02f'_c$ or $0.06f'_c$, respectively. This suggests that no special allowable stress increase was permitted for concrete structures subjected to seismic forces. For wood, however, a one third increase was permitted. And most interestingly, combined stresses in steel could exceed working stress limits by 50%. This provision clearly identifies steel as a material more suitable for earthquake resistant design than either concrete or masonry. The perceived performance of steel in the 1906 and 1925 earthquakes had made an impact on the design and construction community.

4.3 1933-1958

The Long Beach earthquake of 1933 also changed seismic design in California. In the codes that followed, masonry bearing wall buildings were to be designed for 0.10g, while concrete shear wall and frame structures could be designed for between 0.02 and 0.05g. In 1937, taller (over three stories) shear wall buildings of all types were to be designed for 0.06 to 0.10g, while complete moment frame buildings could be designed for 0.02 to 0.06g, provided the frame could resist 0.02g on its own (SEAOC, 1968). The ranges of design values were based on soil conditions. Interestingly, some practices adopted in the Los Angeles area after Long Beach did not become standard in San Francisco until about 1950 (Steinbrugge and Moran, 1954).

Code provisions developed in the 1940s began to incorporate the period of the building into the base shear, recognizing that the fundamental periods of taller structures resulted in lower amplification of ground motion. Los Angeles adopted base shear provisions in 1943 that varied with building height. In 1948, San Francisco adopted provisions making design base shear inversely proportional to building period, based on recommendations of the Joint Committee on Lateral Forces, formed from local chapters of ASCE and SEAONC (Popov et al., 1993). This committee also developed the "*K*" factor, which initially distinguished buildings from non-building structures, but which would come to represent the system ductility factor for different materials and framing systems (SEAOC, 1968).

4.4 1959-1965

In 1959, SEAOC issued its first *Lateral Force Recommendations*, also known as the Blue Book. The 1959 Blue Book redefined the "K" factor as a function of building material and structural system. Lower K values for ductile systems and materials recognized better inelastic performance and energy absorption (Strand, 1984).

Also in the 1959 Blue Book, buildings over 160 ft tall were required to have a lateral system that included a complete moment resisting space frame "made of a ductile material or a ductile combination of materials. The necessary ductility shall be considered to be provided by a steel frame with moment-resistant connections or by other systems proven by tests and studies to provide equivalent energy absorption" (SEAOC, 1959). In 1961, the *UBC* adopted the Blue Book provisions; the *UBC* has incorporated SEAOC's *Lateral Force Recommendations* ever since.

The new moment frame requirement in code section 2313(j), or "section j," was immediately and highly controversial (Layne et al., 1963; Kellam, 1966). The height trigger of 160 ft was held over from earlier Los Angeles codes so as not to suddenly render obsolete all older buildings without moment frames. Nevertheless, together with the explicit endorsement of steel, the height trigger was perceived as an arbitrary limit on concrete (Layne et al., 1963).

Engineers who participated in the drafting of "section j" were seeking a tough, reliable system for "major buildings" and clearly favored steel frames. One later noted that if not for political and legal considerations, steel frames would simply have been mandated for tall buildings (Layne et al., 1963). The Blue Book writers instead accepted any system that could demonstrate ductility equivalent to that of a steel frame.

4.5 1966-1985

In response to the *UBC's* "section j" endorsement of steel frames, specific provisions for ductile concrete frames were completed by SEAOC in 1966 and adopted by the *UBC* in 1967. But while the new concrete provisions were expected to provide *sufficient* ductility and energy absorption, the SEAOC code writers did not consider this concrete system *equivalent* to steel (Kellam, 1966). The 1968 Blue Book commentary (SEAOC, 1968) made clear that steel remained the standard for seismic performance:

> "Moment-resisting frames of ductile materials have shown particularly good earthquake-resistant characteristics…. The ability of various building materials to achieve desired ductility is not equivalent, by any means. The property exhibited by moment-resisting space frames of structural steel of ASTM A-7, A-36 and A-441 has long been accepted as the desirable standard."

Though intended only to distinguish new concrete provisions from established steel practice, this is, in retrospect, a bold statement. Regarding the new ductile concrete frame provisions, the Blue Book commented that “it has not been possible to make a comprehensive evaluation of … the performance of such structures in response to earthquakes.” But the same could have been said for welded or even bolted steel moment frames. Only a handful of steel moment frame buildings were investigated thoroughly after earthquakes prior to 1968, so the perception of good earthquake resistance must have relied on research and testing. However, while testing had been performed as early as the late 1950s, significant study of the cyclic inelastic behavior of steel members began in 1959 (Bertero and Popov, 1965), the same year that “section j” was drafted. The first cyclic connection tests began only in 1966 (Popov and Pinkney, 1969).

Thus, while ductile *material* behavior had “long been accepted as the desirable standard,” a track record of actual building performance had not been established. Indeed, Popov and Bertero (1970) cited damage to the Cordova Building in the 1964 Alaska earthquake as motivation for later tests. Still, the Blue Book assigned the lowest (best) “K” factor to ductile moment-resisting space frames of steel or ductile reinforced concrete. Requirements for ductile steel moment frames included:

- Steel of grade A-36, A-440, A-441, A572, or A588.
- Moment connections capable of developing the full plastic capacity of the girder. An exception was made if “adequately ductile joint displacement [was] provided.”
- For high strength steels, plastic hinges away from bolt holes.
- Testing of butt welded connections between the girder and column flanges. A testing program was to be established by the engineer.

These 1968 provisions represented the first codified description of a WSMF. The exception given in the second requirement is interesting because the necessary calculations, involving inelastic response to unreduced seismic loads, would have been unusual for most engineering offices of the time.

The last requirement shows an attempt to address quality control in the welded joints from the very introduction of WSMFs. At the time, ultrasonic testing (UT) was relatively new in building construction. Standards and specifications did not match those available for radiography or magnetic particle testing, and the value of UT depended on the skill of individual technicians. Still, the nature of UT was considered well suited to WSMF construction. Experts from Bethlehem Steel noted in 1965 that “Moment connection welds, and especially the vertical fusion areas of such welds, are more susceptible to definitive ultrasonic examination than to inspection by other methods” (Couch and Olsson, 1965). The value of UT for inspection of 1980s era WSMFs would be questioned after the Northridge earthquake (Paret, 1999).

The 1976 *UBC* required all welds to be inspected until a rejection rate “consistently” under 5% was established, at which time the testing rate “might be reduced to 25%.” It has been suggested that the relaxation of inspection may have contributed to the performance of buildings in Northridge (Goltz and Weinberg, 1998). This concern may not be justified, as many buildings exhibited damage in the lower floors, where 100% of the welds would have been tested during construction.

In 1975, the SEAOC Blue Book recommended additional requirements for ductile steel moment frames. With respect to connection welds, the commentary notes that residual stresses in beam-column welds can result from the welding sequence. This may have been a reference to damage to the ARCO tower discovered after the 1971 San Fernando earthquake (see the section on past performance below). Several figures are included in the Blue Book commentary showing welded moment frame and stiffener plate details. The use of welded webs is also suggested to increase the ductility relative to a bolted web connection. Mention is made of column panel zone and continuity plate requirements, which were intended to limit shear deformation and column flange distortion.

Ironically, the need for reliable inelastic behavior at the beam end may have hastened the transition to the welded pre-Northridge detail now considered inadequate. From 1963 until after 1978, Part 2 of the AISC Specification, which addresses plastic design, warned against punched holes in the beam tension flange, perhaps discouraging the use of bolted connections. The 1973 Specification, citing Popov and Pinkney (1969), made a point of noting that full plastic capacities could be achieved with bolted connections "instead of full penetration groove welds." But by 1975, the standard connection in California joined the beam to the column by welding the beam flange and bolting the beam web to a shear tab (SEAOC, 1975). A bolted flange connection was simply more expensive (see Table 2-2).

FCAW single-bevel groove welds were prequalified by the AISC Specification for the first time in the 1973 Seventh Edition.

4.6 1986-1988

UBC provisions for steel moment frames remained essentially unchanged from 1970 through 1985. Perhaps this was related to the attention given to concrete after the 1971 San Fernando earthquake. Research performed in the 1970s and early 1980s eventually led to significant modifications in the 1988 *UBC*. Krawinkler (1985) summarized this work and SEAOC's tentative provisions that were ultimately adopted by the *UBC*.

- **Beam-to-column connections.** Welded beam web connections should be used to develop the moment strength of the beam. Krawinkler noted that in beams with a Z_f/Z ratio (flange to beam plastic section modulus ratio) less than 0.7, a connection that develops the moment capacity of the beam as required by the code will rely heavily on the moment strength provided by the beam web. Under bending and shear loading, a bolted connection can slip, leaving the flange weld to carry the moment by itself. This may result in flange failures "within the weld, at the toe of the weld, or at the interface between the weld and the column flange." Krawinkler, citing Popov and Stephen (1972), noted that a welded web that supplements or replaces the bolted connection can delay this failure until larger plastic rotations have been achieved.
- **Panel zones.** High panel zone strengths demanded by earlier codes often required the use of doubler plates to thicken the panel zone. In part to avoid the substantial fabrication costs of

attaching doubler and continuity plates, engineers in the 1980s began to design WSMFs with larger column sections. These sections had flanges and webs of sufficient thickness to avoid doubler and continuity plates.

SEAOC's 1985 tentative provisions relaxed the panel zone requirements to permit some yielding there. This would reduce the need for doubler plates and lessen the plastic deformation requirements in beams. Kinking of the column flange at high panel zone rotations remained a concern, however.

- **Strong column–weak beam.** The 1988 *UBC* also formalized the strong column–weak beam concept. The goal was to avoid frame configurations that would be subject to single story mechanisms or collapse due to P-delta effects. This was primarily considered a problem when columns had initially high axial loads. This condition would have been more pronounced in single-bay moment frames which typically had relatively large overturning forces.

Popov (1987), aware of coming 1988 provisions, remarked, "Due to the great uncertainty of the forces that a structure may have to resist during an earthquake, complete reliance on the minimum code provisions is hazardous."

The 1988 *UBC* also finally included language for the "prequalified" WSMF connection that had been standard practice in California since the mid-1970s (SEAOC, 1975). A connection was considered adequate to develop the moment capacity of the beam if the beam flanges had full penetration welds to the column and if the beam web connection was able to resist both its gravity and seismic shear demands. In addition, if the Z_f/Z ratio of the beam was less than 0.7, the web had to be welded to the shear tab to provide additional moment strength.

4.7 1989-1993

After 1988, the *Uniform Building Code* and the SEAOC *Lateral Force Recommendations* did not revise any detailed provisions for steel moment frames. However, the *UBC's* general seismic provisions have undergone substantial changes. The transition to ultimate stress design began or was completed for most materials, and changes in the calculation of lateral-force demands were introduced, using an "R_w" factor to replace the "K" factor. Issues relating to building irregularities were fleshed out in more detail, considering the performance of buildings in the 1985 Mexico City and 1989 Loma Prieta earthquakes. The concept of strength design and more realistic earthquake force and drift demands were incorporated into the code with the introduction of the $(3/8)R_w$ factor.

In 1992, AISC published its first set of seismic provisions. With respect to WSMFs (designated as Special Moment Frames), the AISC provisions endorsed the same prescriptive detail as the 1988 *UBC*. Later editions of the AISC provisions would be adopted by both NEHRP and the International Building Code.

Code changes in response to the January 1994 Northridge earthquake are discussed in a separate section below.

5. PERFORMANCE OF STEEL FRAME BUILDINGS IN PAST EARTHQUAKES

The effects of large earthquakes on structures in the United States have been observed since the early 1800s. Since around 1900, steel framed buildings have experienced heavy shaking in almost every major event. By observing their performance, engineers are able to advance the state of the art and practice in steel moment frame design. While buildings erected before the late 1960s generally did not employ welded moment connections, a review of steel performance in prior events teaches us how the use of steel evolved into a common and popular material. During the 1970s, welded steel moment frames began to quickly replace the all-bolted moment frame in most regions of high seismicity. Lessons learned from the performance of steel frames since the 1970s are crucial to the advancement of the WSMF state of the art and practice.

One problem reviewing records of damage in past earthquakes is that the term "steel framed" has been used to mean more than just moment resisting frames and certainly more than just typical pre-Northridge WSMFs. For example, diagonally braced frames (Yanev et al., 1991), infilled frames (Steinbrugge and Moran, 1954), intended or unintended dual systems (Berg and Stratta, 1964; Yanev et al., 1991), steel gravity frames not designed to resist lateral load, and prefabricated "rigid frame" warehouse structures have all been categorized as "steel frame structures."

Another problem is that careful postearthquake inspection of beam-column joints was rarely, if ever, performed before Northridge. In several cases, WSMF damage from the 1989 Loma Prieta earthquake was found only after the Northridge experience prompted some reinspections of structures previously considered undamaged. Frames analyzed after the 1971 San Fernando earthquake (see Table 5-3) were also classified as undamaged based on nominal inspections. Except for one that was studied again after Northridge and found damaged, those San Fernando case study buildings probably have not been looked at closely since.

But the major problem in trying to gauge the past performance of steel moment frames is the simple lack of data. Preece (EERI, 1976) recognized this in a reconnaissance report after the 1976 Guatemala earthquake:

> One high-rise structural steel building 22 stories (sic) in this City can hardly be considered a test of structural steel performance, especially when a 21-story reinforced concrete building next to it also came through unscathed.

Despite these difficulties, the SEAOC *Blue Book* noted through the 1970s that steel moment frames "have shown particularly good earthquake-resistant characteristics," and through 1990 that WSMFs "are believed to be a *proven, reliably* ductile structural system" (emphasis added). A few successes, or rather the lack of any notorious failures, established a reputation that spanned decades, even as details and construction techniques changed profoundly.

The previous section of this report showed how a 1968 Blue Book statement about steel's ductility could have been misinterpreted as a record of actual building performance. Postearthquake observations may have been misinterpreted or misapplied by later engineers as well. Goel made this point, if somewhat obliquely, in 1968. Commenting on a lack of test data, he described contemporary building codes—in general, not just the steel provisions—as having produced “designs that have successfully withstood severe earthquakes in the past with little or no damage at all.” Yet his citation was to a 1955 report on the 1952 Kern County earthquake, which affected engineered structures built mostly in the 1920s and 1930s.

The AISI-sponsored study by Yanev et al. (1991) is noteworthy. The authors intentionally chose modern era earthquakes affecting steel, concrete, and masonry structures in order to make useful comparisons. Their basic conclusion:

> Buildings of structural steel have performed excellently and better than any other type of substantial construction in protecting life safety, limiting economic loss, and minimizing business interruption due to earthquake-induced damage.

Yanev would emphasize the point at an AISC conference (Melnick, 1991):

> [S]teel will always outperform concrete in an earthquake … If you want to go beyond code without paying for it, go steel. What is the only building designed for Zone 2 that can survive a Zone 4 earthquake? Steel.

Indeed, steel has outperformed other structural materials in earthquakes (see Table 5-2). But in the Northridge earthquake at least, WSMFs did not live up to engineers’ high expectations. Whether or not the WSMF remains the system of choice for seismic resistance, it is clear in retrospect that enthusiastic pre-Northridge endorsements suffered from all three of the problems noted above: a conflation of reports from various structural systems old and new, cursory post-earthquake inspection, and generally sparse data. This is perhaps the singular lesson of this report.

From a post-Northridge perspective, other principal lessons from a review of earthquake records prior to Northridge include:

- Until Loma Prieta in 1989, only a handful of modern WSMF buildings had ever been shaken by a major earthquake. Fewer than a dozen WSMFs were closely inspected after the 1985 Mexico City, 1971 San Fernando, and 1964 Prince William Sound earthquakes combined.
- WSMF buildings are still relatively rare, and WSMF damage is less obvious than concrete, wood, masonry, or even steel braced frame damage. When damaging earthquakes occur, it is therefore easy to overlook potentially damaged WSMF structures and to conclude, perhaps in error, that as a class they offer excellent performance. That early post-Northridge reports were prepared to make such an assessment (AISC, 1994) is instructive.
- Engineers have consistently attributed structural failures to poor construction quality.

Table 5-1 summarizes the recorded performance of steel buildings of the WSMF era (post-1960) prior to Northridge. Due to the volume of collected data, WSMF performance in the 1994 Northridge earthquake is discussed in a separate section. Table 5-2 briefly compares the broad performance of different structure types in major North American earthquakes since 1960. An exhaustive comparative study is beyond the scope of this report. The text that follows describes effects of selected pre-Northridge earthquakes relevant to the design, testing, regulation, and performance of steel frame structures. Appendix C includes descriptions of steel moment frame performance in the 1995 Kobe (Japan) and 1999 Ji-Ji (Taiwan) earthquakes.

5.1 San Francisco, 1906

Buildings that experienced the 1906 San Francisco earthquake did not have welded steel moment frames. San Francisco and Oakland did have, however, many multi-story steel, cast iron, or wrought iron frame or skeleton buildings with riveted connections and masonry infill.

Reports compiled by the United States Geological Survey (USGS, 1907) described some of the damage to steel buildings. Steel framed structures "braced" for wind resistance performed better than those without bracing. (In this era, the term "bracing" referred to any means of resisting lateral loads, including diagonal rods, portal frames with rigid moment connections, and knee braces.) For example, of the Call Building it was written, "Had the building been as well designed to resist fire as to resist earthquake, it is probable that the total damage would have been very much less than it was." But from the damage descriptions it is clear that most steel frames were not as heavily braced as the Call Building.

Partial infill and masonry piers in steel frames were widely damaged by story racking. Damage to frame components included buckled column plates associated with failure of terra cotta or plaster fireproofing (e.g. Aronson Building, Bullock & Jones Building, Hotel Hamilton) and some sheared rivets (e.g. Union Trust Building). Much of the damage to riveted connections was attributed to "faulty construction" or "careless workmanship." Diagonal rod bracing in some steel structures (e.g. Call Building, Union Ferry Building) was yielded and buckled.

Earthquake or fire damage to masonry or plaster fireproofing left the steel frames vulnerable: "In the San Francisco fire, for the first time, the collapse of protected steel frames, due to the destruction of the fireproof covering at a comparatively early stage in the fire, was a matter of common occurrence (USGS, 1907)." Interestingly, however, while some city blocks were dynamited to create fire breaks, one writer noted in the USGS report how unsuccessful it was and how difficult it would be to bring down a steel frame building with dynamite.

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Prince William Sound, Alaska, 1964 8.4, 6.7 (Berg and Stratta, 1964; Yanev et al., 1991. See also Table 5-2)	Multistory steel frames rare. At 120 km: Cordova building: At first story, buckled steel columns and damaged concrete core walls. Steel frame connections had beam flanges bolted to top and bottom clip angles.	At 120 km: Several complete collapses of multistory concrete and masonry structures. Small rigid masonry structures mostly undamaged due to low frequency shaking. Much damage related to soil failures. Most casualties due to tsunami.
Venezuela, 1967 6.5 (Hanson and Degenkolb, 1969)	At 50 km (Caracas): "There were a few multistoried steel buildings in Caracas—none of which suffered significant damage." Only the Simon Bolivar Center is described: mid 1950s 30-story "steel frame."	At 50 km (Caracas): Many reinforced concrete buildings with "major damage," including four complete collapses of 10 to 12-story buildings.
Tokachi-Oki, 1968 7.9 (Yanev et al., 1991)	No steel moment frame data.	At 200 km: Hundreds of collapses, mostly wood residences. Several concrete buildings with severe damage or collapse. In steel braced frames, some buckling and fracture at splices.
Peru, 1970 (EERI, 1970)	No steel moment frame data.	At 25-50 km: Extensive and severe damage to concrete, historic unreinforced masonry, and adobe.
San Fernando, California, 1971 6.6 (Steinbrugge et al., 1971; Yanev et al., 1991. See also Tables 5-2 and 5-3.)	At 10-40 km: 30 steel moment frames with no observed damage. Connection types not given. All but 5 erected before 1967. At 25 km: One 16-story 1969 building described as undamaged (Yanev) would later be significantly damaged by 1994 Northridge earthquake (Kariotis and Eimani, 1995). At 40 km: cracked welds in two WSMF highrises under construction. At 3 km: "some working of the connections" observed (Yanev).	At 10-40 km: Many concrete collapses, including medical facilities. Tilt-up damage and collapses. Some damage to steel diagonal braces.
Managua, Nicaragua, 1972 6.2 (Yanev et al., 1991)	Only one steel building, frame and connection type not described. "Signs of yielding" in some ground floor columns. Nonstructural damage included broken exterior glass panels.	Most modern commercial buildings, typically with soft stories and masonry infill, "performed very poorly." Some masonry and concrete buildings "escaped serious damage."

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era (continued)

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Guatemala, 1976 7.5 (EERI, 1976)	No steel moment frames. One 19-story steel braced frame / dual system described (150 km from 7.5 main shock, 40 km from 5.8 second shock): A36 steel, E60XX full-penetration girder-column welds, A490N bolted webs, Seventh Edition of AISC specified. Postearthquake visual inspection of some welds by reconnaissance team revealed “excellent” quality and, by inference, no visible damage. No structural damage and very light nonstructural damage (although brand new, so no contents in place). Adjacent 21-story concrete building also undamaged.	At 50-150 km: Four collapses of infilled concrete frames. Extensive damage and collapse to adobe residential construction.
Friuli, Italy, 1976 6.5, 6.0 (Stratta and Wyllie, 1979)	No steel frame data.	At 0-30 km: General building damage: about 80% of private buildings heavily damaged or destroyed.
Romania, 1977 7.1 (Berg et al., 1980)	No steel frame data.	General building damage: 35 mid-rise concrete collapses in Bucharest (at 170 km), 32 of them pre-1940.
Miyagi-Ken-Oki, Japan, 1978 $M_S = 7.4$ (EERI, 1978)	Likely several hundred steel structures, few with obvious damage, and few studied. Three steel moment frame buildings described: one 1973 17-story tower apparently undamaged, but no postearthquake joint inspection. One 18-story dual system with minor shear wall cracks. One 4-story steel frame with precast panel failure.	In Sendai (100 km, 0.25g to 0.40g pga): “Good general performance of modern, engineered buildings up to 20 stories high.” But at least four complete collapses of concrete structures. Some steel brace buckling and fracture.

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era (continued)

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Oaxaca, Mexico, 1978, and Guerrero, Mexico, 1979 M_S = 7.8, 7.6 (Forell and Nicoletti, 1980)	No steel frame data. Sheared high strength bolts in trussed portal frame at steel mill building near Guerrero.	Oaxaca: Generally minor damage, no collapses. Heavy damage to one 2-story concrete frame. Guerrero: Widespread damage to unreinforced brick, adobe. Isolated concrete frame damage. Significant damage to masonry infill. At 300-500 km: Felt strongly in Mexico City despite distance. Pounding and nonstructural damage to tall buildings. Collapse of one 3-story concrete frame.
Montenegro, Yugoslavia, 1979 M_S = 6.6 (EERI, 1980)	No steel frame data.	At 10-25 km: Severe damage to old unreinforced stone masonry, minor damage to 1950s brick and block masonry, good performance of concrete bearing wall and precast bearing wall buildings, poor performance of concrete frames (many infilled).
Campania-Basilicata, Italy, 1980 6.8 (Stratta et al., 1981)	No steel frame data.	At 20-90 km: General building damage: many collapses and near collapses of stone masonry residences and infill concrete frames.
El-Asnam, Algeria, 1980 M_S = 7.3 (EERI, 1983)	No steel frame data.	At 0-20 km: 20% of El-Asnam buildings collapsed, 60% severely damaged. Collapses included many “modern” multistory concrete structures.
Central Greece, 1981 6.7, 6.3 (Carydis et al., 1982)	No steel frame data.	At 20-70 km: General building damage: severe damage to block masonry and infill concrete frames.

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era (continued)

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Coalinga, California, 1983 6.7 (Tierney, 1985; Yanev et al., 1991)	Little steel frame data. Two 1940s steel frame buildings with no visible damage (Yanev).	At 10-15 km: Severe damage to about half of wood cripple-wall residences, and most unreinforced masonry buildings. Very light damage to wood-frame commercial, concrete block, and cast-in-place concrete buildings.
Borah Peak, Idaho, 1983 7.3 (*Earthquake Spectra*, November 1985)	No steel frame data.	At 30 km (Mackay): slight to moderate unreinforced masonry damage. At 0-60 km: mostly minor damage; worst damage involved masonry parapet and veneer failures.
Morgan Hill, California, 1984 M_L = 6.2 (*Earthquake Spectra*, May 1985)	One 1976 steel frame building described (20 km, 0.04g recorded pga, 0.18g roof acceleration): Nonstructural and contents damage, but evacuated because of long duration lightly damped response. Retrofitted in 1991 with dampers. No systematic joint inspection after Northridge, but 1991 retrofit exposed about 200 connections, and no obvious visible damage was reported (Crosby, 1999).	At 10-30 km, no structural damage or light structural damage to "engineered" steel and concrete buildings. Some tilt-up damage. Structural damage, including a few collapses, to less than 10% of residences.
Chile, 1985 M_S = 7.8 (*Earthquake Spectra*, February 1986)	The few steel industrial structures performed well, but unbraced frames had substantial nonstructural damage. At 64 km: Wide-flange sections with bolted connections used as horizontal braces suffered some web tearing and gusset buckling in one industrial building.	At 100 km (Santiago): Heavy damage to historic adobe and URM structures. Hundreds of 5-25 story concrete buildings; most appeared to perform well, but many with significant damage, and several collapses.
Mexico City, 1985 8.1 (Osteraas and Krawinkler, 1989; Yanev et al., 1991. See also Table 5-2.)	Few "modern" WSMFs in the region. Some older moment frames with infill, knee braces, or riveted connections collapsed. Few post-1957 moment frames damaged, but some weld fracture noted. Collapse of 3 steel frame structures with braced bays due to column overload unrelated to frame action.	At 300-500 km: Hundreds of collapses, thousands of buildings damaged. Unique ground motion and soil conditions hit mid-rise buildings especially.

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era (continued)

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Whittier, California, 1987 $M_L = 5.9$ (*Earthquake Spectra*, February 1988; H.J. Degenkolb Associates.)	No modern steel frame data, but Los Angeles area WSMFs certainly were shaken by the earthquake. No structural damage to 1920s steel frame with masonry cladding.	At 0-30 km: Substantial damage to masonry bearing wall buildings, with lower damage rates among reinforced buildings. Significant structural damage to several modern concrete structures, including parking garages and tilt-ups.
Spitak, Armenia, 1988 $M_S = 6.8$ (*Earthquake Spectra*, August 1989)	No steel frame data.	At 10-30 km: collapses and severe damage to typical multistory stone masonry and precast frame structures.
Loma Prieta, California, 1989 7.1 (Phipps, 1998; Yanev et al., 1991. See also Tables 5-2 and 5-4.)	About 30 WSMFs inspected, nearly all after Northridge damage found. Five with connection damage. Pounding damage at seismic joints between sections of ductile steel frame complex.	At 60-100 km: Most fatalities from URM failures and collapse of concrete highway structure. Severe damage to isolated concrete, wood frame, and tilt-up structures.
Luzon, Philippines, 1990 $M_S = 7.8$ (*Earthquake Spectra*, October 1991; EQE, 1990)	Based on preliminary observations, and in contrast to adjacent concrete structures, "performance of steel-frame buildings was excellent, consistent with observations in other earthquakes." Even in areas of extensive liquefaction and settlement, steel buildings were observed to be undamaged.	At 40-60 km: Many concrete collapses in Baguio. Central business district of Dagupan "essentially destroyed" due in part to soil spreading.
Landers and Big Bear, 1992 $M_S = 7.5, 6.6$ (Phipps, 1998; Reynolds, 1993)	One two-story steel frame structure with severe cracking discovered after Northridge. Los Angeles area WSMFs were shaken by the earthquake, but no steel frame data reported.	Near epicenter: Isolated wood frame and concrete block structure damage. At 170 km: 0.04g maximum ground acceleration recorded in Los Angeles.

Table 5-1 Earthquake Performance of Steel Moment-Frame Buildings in the WSMF Era (continued)

Earthquake and Magnitude **(References)**	**Performance of Steel Moment-Frame Buildings**	**Overall Performance at Nearest Urban Center (epicentral distance in km)**
Hokkaido, Japan, 1993 M_W = 7.8, 6.3 (*Earthquake Spectra*, April 1995a)	At 50-80 km: 2 steel frames noted, both undamaged, but neither similar to typical California office building. At batch plant hopper, first story steel frame bent undamaged despite buckling of braced frame structure above. Two-story steel frame building survived devastating tsunami with no structural damage.	General building damage: light to nonexistent due to shaking, moderate to total due to tsunami. Extensive liquefaction.
Guam, 1993 M_S = 8.1 (*Earthquake Spectra*, April 1995b)	No steel frame data.	Significant damage to non-ductile concrete buildings. Dozens of buildings severely damaged but no deaths.

Table 5-2 Damage by Structure Type in Selected North American Earthquakes of the WSMF Era

Earthquake	Wood frame residential	Unreinforced Masonry	Precast Concrete	Concrete frame or wall	Steel frames
Prince William Sound, 1964	Excellent performance (Wood, 1967).	Not a common construction type in the area.	Typically slight to moderate damage, less than expected (Wood, 1967).	1-5 story: generally no significant structural damage, one partial collapse. Taller: considerable structural damage.	Inconclusive. Considerable structural damage, but only a few buildings had complete steel frames.
San Fernando, 1971	Majority of buildings under 20% loss. Most structural damage in foundation anchorage and open fronts.	Moderate or severe damage to half of brick buildings in downtown San Fernando.	Multiple tilt-up collapses.	Many collapses generally caused by poor ductility and irregularities.	No significant structural damage was observed in general. Cracked welds observed in two buildings under construction.
Mexico City, 1985	Not a common building type in the area.	Many URMs severely damaged. Wall-floor connections and out-of-plane failures.	Not a common construction type in the area.	7-15 story, frame and infill structures heavily damaged or collapsed. Similar low-rise structures perform better (Bertero and Miranda, 1989).	Some pre-1950 steel frames collapsed. Collapse of isolated braced frame buildings due to column failure. Little other moment frame damage reported.
Loma Prieta, 1989	Severe damage to wood structures with open fronts or tuck-under parking, especially on soft soils.	Many URMs severely damaged or collapsed. Wall-floor connections and out-of-plane failures.	Many examples of connection damage although collapses were uncommon.	Nonductile frames and wall structures damaged, including fatal freeway collapse. Newer structures generally performed well.	Five buildings known to have slight to significant weld damage, some discovered only upon post-Northridge inspection.
Northridge, 1994	Severe damage to multi-story wood structures with tuck-under parking.	Many URMs damaged but many collapses avoided by previous retrofits.	Many examples of connection damage and several significant collapses.	Older structures damaged, including freeways. Newer structures generally performed well. Deflection compatibility issues noted.	Widespread, unexpected connection damage, several buildings declared unsafe, at least one irreparable. No collapses.

Note: See Table 5-1 for additional information and references.

Frank Soulé, dean of the University of California college of civil engineering, was given the last word in the USGS report. About twenty years after the first steel frame buildings were erected, and with only the San Francisco earthquake as an historical record, he described structural steel as "no longer in the experimental stage as to resistance … to earthquake tremors." While calling diagonal bracing an "absolute necessity," he nevertheless praised the San Francisco performance:

> Undoubtedly many of the high steel buildings in San Francisco were designed without reference to earthquakes, but they have nobly withstood their effects, and steel frames have proved themselves entirely adapted to earthquake countries. [In the San Francisco earthquake], they suffered comparatively little injury, … confined to the shearing of rivets and connections … and to some buckling of braces.

Despite such an endorsement, it is hard to draw any lessons of particular usefulness from this event with respect to the performance of modern WSMFs. Any buildings from 1906 that are still in service face potential problems different from those of WSMFs. Indeed, several steel structures with diagonal bracing or knee braces, with or without masonry infill, collapsed in the 1985 Mexico City earthquake (Osteraas and Krawinkler, 1989).

5.2 Kanto, Japan, 1923

The use of steel in Japan at the time of the Kanto earthquake was relatively new, with a history of only about five years. Four large buildings had been completed and two were almost complete. These buildings suffered little to no damage. The use of masonry infill was common. Damage to the infill and facades was significant, but the frames performed well. Braced frames also suffered relatively little damage. Two steel bridges, one on masonry piers and one made entirely of steel, performed quite differently, with the former collapsing and the latter suffering almost no harm (Hadley, cited by SAC, 1998). Japanese building codes adopted seismic design coefficients after this earthquake (Tobriner, 1984).

5.3 Santa Barbara, 1925

As did the 1906 earthquake, the smaller event in Santa Barbara highlighted the good performance of steel frame infill buildings relative to masonry and concrete structures. Seventeen concrete and masonry buildings were destroyed or eventually demolished, but two steel frames close to the epicenter were not severely damaged. The largest of the two was a post office in an area severely hit by the earthquake. The other building was a church. (California Institute of Steel Construction, cited by SAC, 1998).

The lack of damage to these buildings again led people to think of steel as a material well suited to resisting earthquakes. The concept of incorporating flexibility into a building to help it resist damage was becoming popular. This earthquake gave rise to the first seismic design provisions in U.S. building codes, most of which were probably based on work done in Japan (Tobriner, 1984; Strand, 1984).

5.4 Long Beach, 1933

While few steel structures were affected by the 1933 Long Beach earthquake, the damage to other buildings was again used to gauge the relative performance of steel versus masonry and concrete. The sixteen story Villa Riviera building was a steel frame built in 1928. It suffered virtually no damage, while several concrete and masonry structures nearby collapsed or were heavily damaged (SAC, 1998).

Though not of great seismic magnitude, the Long Beach earthquake did extensive damage to a densely populated area with many vulnerable buildings, including several schools (Coffman and von Hake, 1973). As noted in section 4 of this report, the Long Beach earthquake was followed by significant earthquake safety legislation (Strand, 1984; SEAOC, 1968).

5.5 Kern County, 1952

Following the 1952 Kern County earthquake, Karl Steinbrugge and Donald Moran (1954) studied the “damagability” of different structural systems. The steel structures affected by this earthquake were almost all infilled frames erected in the 1920s or 1930s in Los Angeles, over 100 km from the epicenter. Some of them suffered minor damage. The only nearby steel frame building suffered some pounding damage unrelated to its frame connections.

Steinbrugge and Moran considered the affected steel frames to be in the second best category of building performance. Ranking as best were small wood structures under 3,000 square feet. Following steel structures in order of performance were: concrete frames and shear walls, large wood frame buildings, steel frames with URM infill, concrete frames with URM infill, precast concrete and other flexible diaphragm buildings, and finally URM and adobe structures.

5.6 Prince William Sound, Alaska, 1964

Most of the research into building performance in the 1964 Alaska earthquake focused on concrete buildings and on geotechnical effects. Of the two dozen or so buildings discussed in detail by Wood (1967) and Berg and Stratta (1964), only one, the Cordova building, had a steel frame as its primary lateral force-resisting system.

Berg and Stratta identified one welded beam-to-column failure in the steel framed shop building at the Alaska Highway Department Yard. The damage was in a welded angle clip connecting the beam web to the column. The clip appears to have torn away from the flange of the column. While certainly a fracture of the weld, it is not typical of the WSMF damage seen in Northridge. The clip was attached to the column with fillet welds that appear to have failed in prying. Elsewhere in the building, Berg and Stratta noted that “some columns yielded below the beam connections.” The authors also point out that anchorage of the steel columns to base plates and of base plates to the footings failed in many places. The failures were attributed to poor quality welds between the columns and base plates, shearing of the anchor bolts, or spalling of the footings. Tearing or fracture of base plates was not mentioned.

The Hill Building, an eight-story office building, was a steel and concrete frame with concrete and CMU walls. The concrete and CMU were badly damaged in several locations, but

Berg and Stratta noted that “there was apparently no damage to the steel frame. Several of the beam-to-column connections were exposed for the purposes of inspection and found to be in good condition.” Reports of sheared bolts in the building may have led them to inspect the connections, which turned out to be simple bolted connections. The authors point out that the frame was designed to carry gravity loads only.

The Cordova building, a six-story office building, suffered perhaps the worst steel frame damage. The building had a full steel moment frame in one direction and “partial moment-resisting beam-to-column connections [in the other].” Moment connections were made using shop welding and field bolting with high-strength bolts. Damage was generally concentrated at the first floor where a number of wide flange columns (typically 14WF30 or 14WF61) buckled. The damage appears to be axially induced with the column flanges sometimes tearing away from the column web directly below the beam-column connection. A damaged column is shown in Figure 5-1. Popov and Bertero (1970) would later cite damage at the Cordova Building as an indication that monotonic testing is inadequate for demonstrating seismic performance.

Figure 5-1 Damaged Moment-Frame Column, Prince William Sound Earthquake, 1964

Source: Berg and Stratta, 1964

Several other steel buildings were investigated, including a three-story psychiatric institute, a six-story hospital, and a one-story university building. None were found to have any significant structural damage, although it is unlikely that the joints were uncovered and closely inspected.

Whether or not other steel frame buildings in Anchorage had weld failures will probably never be known at this point. This highlights the natural tendency of engineers to focus on the more obvious building damage, especially in such a powerful event. Damage to concrete structures and landslide effects were clearly dramatic and evident and kept engineers and researchers busy. Since the use of welded steel moment frames was not widespread at the time, especially so in Alaska where there were many more concrete buildings, it may not have occurred to engineers to look behind lightly damaged furring and curtain wall systems for evidence of damage to structural joints.

Berg and Stratta concluded in their study of the Alaska earthquake that “structures with steel frames generally withstood the earthquake well. Steel frames which were damaged were repairable with ease, speed and economy.” While probably speaking to concrete performance, they also note that “connection details deserve special attention in earthquake zones. To take advantage of the energy absorbing capacity of the structural members, one should design the connections so that first failure would occur in a member rather than in the connection.”

5.7 San Fernando, 1971

To structural engineers, the San Fernando earthquake is infamous for exposing the seismic hazards of non-ductile reinforced concrete frames and for highlighting the dangers of soft stories. Several prominent failures led to changes in the building code. There was also some study of steel structures’ performance. However, as with the Alaska earthquake seven years before, apparently little attention was paid to buildings that did not exhibit obvious structural damage.

The Pacific Fire Rating Bureau quickly studied thirty completed steel buildings and two under construction at the time of the earthquake (Steinbrugge et al., 1971). Some stairs, concrete walls, and nonstructural elements were damaged, but no structural damage to the completed steel frames was noted: “With respect to complete buildings, the authors know of no significant structural damage to steel frame high-rise buildings as opposed to several cases known in reinforced concrete construction.”

By 1973, more data had been collected and analyses completed, including case studies of instrumented buildings. Table 5-3 lists the recorded peak ground acceleration (PGA) for these buildings, along with a description of some of the observed damage. Noteworthy is the low shaking intensity, except at Bunker Hill, where WSMFs would have qualified for inspection per FEMA-267. Also, the likely inspection scope probably did not involve close scrutiny of connections. As Jennings (1971) had noted earlier, “it should be emphasized that this earthquake was too far away from downtown Los Angeles to be a test of the ultimate strength of [the tall buildings there].”

Records from two of the buildings in Table 5-3 were later analyzed by Foutch et al. (1975). They noted that the Kajima and Union Bank buildings, about a mile apart in downtown Los Angeles, were both subjected to an unusual displacement pulse about ten seconds into the shaking. Accelerogram records also indicated that after initial cycles, each building oscillated at a natural period longer than had been measured by pre-earthquake ambient vibration tests. The authors attributed the period shifts in both cases to “cracking and other types of degradation of nonstructural elements” during the early strong shaking.

Table 5-3 Case Studies of Instrumented WSMF Buildings Affected by the 1971 San Fernando Earthquake

Building	Recorded PGA (g)	Observed Damage
Bunker Hill Tower, 32 stories 800 West First Street, Los Angeles	0.29	"The owner of the building reported that no earthquake damage to any structural elements was observed, and that only minimal damage to nonstructural elements occurred … such as cracking to drywalls … Four elevators were temporarily out of service" (John A. Blume & Associates, 1973).
KB Valley Center, 16 stories 15910 Ventura Boulevard	0.15	No observed structural damage. Minor nonstructural damage: partitions, seismic joints (Gates, 1973b).
Kajima International Building, 15 stories 250 East First Street, L.A.	0.14	No observed structural damage. Nonstructural damage to plaster partitions around elevator and stair cores (Gates, 1973a).
Union Bank Square, 42 stories 445 S. Figueroa Street, L.A.	0.14	Nonstructural damage only: superficial plaster cracking in core walls and stair shafts, elevators out of service temporarily. (Albert C. Martin & Associates, 1973)
1901 Avenue of the Stars, 19 stories, Century City (moment frame in NW-SE direction only)	0.12 (NW-SE)	"[No] major structural damage, and only minor nonstructural damage." (Hart, 1973)

Gates (1973b) performed a thorough analysis of the KB Valley Center, a 16-story WSMF near Sherman Oaks with 42-inch deep plate girders. A peak ground acceleration of 0.15g was recorded in the building basement. Gates described the damage as follows:

> "There was no observed structural damage to the structural elements of the building as a result of the San Fernando earthquake. Minor nonstructural damage occurred in partitions, at seismic joints, and in mechanical equipment mounts."

This building would later be the subject of a detailed post-Northridge case study (Kariotis and Eimani, 1995. See also the Northridge section below). In the 1994 earthquake, for which the PGA at the site is estimated as 0.38g (see Appendix B), the elevators were damaged and permanent drift was measured in the top third of the building. About 20% of the connections in the north-south frames were damaged. In seven places, all in the upper stories of the north-south frames, cracks went through the column flange into the column web.

Another steel frame shaken by both earthquakes was a nine-story structure with about 460 moment frame connections at 18321 Ventura Boulevard. The building was reported as undamaged after San Fernando (Jennings, 1971), but the scope of inspection is unknown. It is unlikely that connections were carefully inspected. After Northridge, thirty-one connections were inspected, and one was reported damaged (Los Angeles Building & Safety, 1998). The other four towers listed in Table 5-3 were in downtown Los Angeles or Century City, two areas exempted from mandatory post-Northridge inspection.

The two buildings listed by Steinbrugge et al. as under construction did suffer structural damage, and the report by the authors gives some insight as to the types and causes of damage. Their report, which remains the best-known account of the damage and which represents the thinking of many engineers at the time, states:

> "The twin 52-story office towers of the $175 million dollar Atlantic-Richfield [ARCO] Plaza Towers in downtown Los Angeles were in the latter stages of construction when the earthquake occurred. Apparently, a 25% increase in the number of cracks in the welds in the two lower stories of both steel framed towers was found after the earthquake, with this increase seemingly due to the earthquakes. Miniscule cracks in the welds connecting heavy metal members occur during the normal welding process and these cracks are normal to this work; routine ultrasonic testing is used to discover these cracks and allow for repairs. It is premature to speculate very far into this particular case due to the lack of time and detailed information, but the potential problem of earthquake induced weld stress cracks in modern steel frame buildings is disquieting. Additionally, there is no assurance that all welded steel frame buildings will be as adequately inspected as was the Atlantic-Richfield towers. The cost of the repair of all welds, regardless of origin, has been placed at $400,000."

Notable are the comments that miniscule but rejectable cracks are common when welding large sections (no reference was cited), that testing and repair was routine, and that the testing at ARCO might have been better than standard practice. Post-Northridge inspections would later find that weld inspection might not have been reliable at ARCO or any number of other WSMFs (Goltz and Weinberg, 1998; Paret, 1999).

In hindsight, the most compelling statement made by Steinbrugge et al. was that "the potential problem of earthquake induced weld stress cracks in modern steel frame buildings is disquieting." While true in general, the nature and cause of the damage at ARCO remains debatable. A private study for one of the building's tenants by the J.H. Wiggins Company (1971) reported cracking in 29% of the second and third floor joints. But in these towers, the second and third floor framing is part of a full story transfer truss, so joints at those levels are not typical of the framing above or of WSMFs in general (Phipps, 1998).

In its summary, the Wiggins report described three types of cracks that occurred either alone or in combinations: "(1) lamellar tearing within the thick column flanges, (2) brittle fracture within the flanges of the girders and (3) brittle fracture within the webs of the girders." Lamellar tearing and pure base metal fractures away from the welds would suggest fracture mechanisms different from those most commonly seen after Northridge. But later in the Wiggins report the three damage types are described again: "(1) tearing within the column material … opposite either the flange or web parts of the girder … (2) cracking *in the weld* within each flange of the girder … (3) cracking *in the weld* of the web of the girder" (emphasis added). These latter descriptions *are* consistent with Northridge patterns.

Three sketches in the Wiggins report schematically describe the crack types "within or adjacent the welds." Though imprecise and inconclusive, they do suggest fractures initiating near the mid-length of the groove weld or at a weld access hole and running from there either up the column flange or across the beam flange and up the beam web. While relatively rare, some Northridge damage did involve fracture outside the heat-affected zone, typically where the beam web is coped to form a weld access hole (see, for example, Uang et al., 1995). This location, especially when not ground smooth, has been a fracture-sensitive point in past tests and in post-Northridge tests (Lee et al., 2000). It now appears that in the pre-Northridge connection, if fracture at the weld root can be avoided, the weld access hole is the next weakest link.

The Wiggins report attributed the ARCO fractures to earthquake exacerbation or "triggering" of "internal stresses generated during the original fabrication process." The potential for cracking or tearing due to weld cooling in restrained conditions is well-recognized (Daniels and Collin, 1972; Putkey, 1993). Recent studies support the hypothesis that welding-induced residual stresses can reduce plastic deformation capacity and promote brittle fracture of the type seen after Northridge (Zhang and Dong, 2000). With respect to this particular building, however, other experts discount the likelihood that pre-earthquake residual stresses would have led to fractures under the relatively small additional effects of the earthquake (Tide, 2000). If the fractures had initiated before the earthquake, however, they might have grown during the shaking (Tide, 2000). Whether or not the earthquake made any contribution to the observed damage, the atypical framing conditions at the second and third floors clearly played a role, as no fractures were found anywhere else in the building (Phipps, 1998).

5.8 Mexico City, 1985

Osteraas and Krawinkler noted that "the 1985 earthquake was probably the first event in which a significant number of steel buildings, including modern ones, were subjected to a severe test." A 1986 damage survey counted about 100 steel structures in Mexico City, including about sixty built after Mexico's benchmark 1957 earthquake (Martinez-Romero, 1986, cited in Osteraas and Krawinkler, 1989). Nevertheless, none would be considered typical of pre-Northridge WSMF practice in California. The 1985 performance of modern steel structural systems in Mexico City, as reported by Osteraas and Krawinkler (1989), is summarized in Table 5-4.

Osteraas and Krawinkler cited poor construction quality in most of the damaged buildings surveyed by Martinez-Romero, and drew the general conclusion that "in most cases, the damage in the post-1957 [steel] structures was minor to moderate." They studied three steel frame structures in detail.

The first, 77 Amsterdam Street, was an 11-story structure built around 1970. Its beams and columns were built up from channels and plates to form box columns and I-beams. The beam-to-column connections consisted of a cover plate fillet welded to the beam flange and full penetration welded to the flange plate of the box column. The beam web was attached to the column plate with a bolted and partially welded shear tab. Failures in this joint typically occurred in the vertical fillet welds connecting the box column flange plates to the column web channels. The full penetration weld from the beam flange plate to the column did not fail. While interesting, the observed failure mode does not give significant insight into the pre-Northridge WSMF problem. Osteraas and Krawinkler computed that the connection was barely able to resist gravity loads considering the types and eccentricity of the welds.

Table 5-4 Performance of Modern Steel Structural Systems, 1985 Mexico City Earthquake

Structural system	**General structural performance**	**Remarks**
Moment resistant frame	41 surveyed, all at least 12 stories. 1 with severe damage 1 with repairable damage 3 with minor damage	Typical MRF has box columns, rolled beams up to W18 or truss girders. Damage “concentrated at welded beam-to-column connections or in truss girders.”
Moment resistant frame with braced bays (similar to *UBC* Dual System)	17 surveyed 2 total collapses, 1 partial collapse 4 with structural damage	Almost all reported damage was at the Pino Suarez Complex.
Steel frames with concrete shear walls	21 surveyed 1 with significant damage 3 with minor damage	Most steel damage to truss girders.

Source: Osteraas and Krawinkler, 1989.

As at 77 Amsterdam, the failures were not typical of those seen in the Northridge earthquake. The connections of the girders to the columns were in most cases weak, and the combined stresses on the flanges of the box columns were large. Generally, axial overstress of the columns due to large brace forces is considered the most likely cause of failure. Indeed, following the Mexico City earthquake, U.S. seismic provisions were changed to prohibit hinges in columns of braced frames (SEAOC, 1988).

The second study addressed the Pino Suarez Complex. This group of five structures suffered some of the most dramatic damage in the earthquake. A 21-story building collapsed onto a 14-story building, and two other 21-story buildings sustained “severe structural damage,” with one of them close to collapse.

The Pino Suarez buildings used a combination of steel moment frames and braced frames. Box columns were formed from four welded plates. Girders were built up sections consisting of plates welded to angle sections forming a flange, with diagonal angle webs. The flange plates were welded to horizontal shear tabs, which were welded to the columns. Figure 5-2 shows a damaged column. Damage was typically in the box columns, which buckled and were probably the ultimate cause of the collapse and near collapse of the 21-story structures.

Figure 5-2 Damage to Steel Frame Column, Mexico City Earthquake, 1985

Source: EERI Annotated Slide Collection

The third building was Torre Latino Americana, a 44-story structure built in 1956 with large built-up wide-flange columns and I-beams. Moment connections were made with all riveted T-flange and web tabs as shown in Figure 5-3. No structural damage was noted in the building, which according to Osteraas and Krawinkler came "as no surprise [considering the building was] a well designed long-period structure." This tower also experienced earthquakes in 1957 and 1962, apparently without any damage obvious or remarkable enough to have been noted in a 1962 presentation on its instrumentation (Zeevaert, 1962).

5.9 Loma Prieta, 1989

The Loma Prieta earthquake caused connection damage in several steel buildings in the San Francisco Bay Area. Some damage to architectural finishes was observed immediately following the earthquake, but in all but one case investigation of beam-column connection failures was not initiated until after the Northridge earthquake more than four years later. It is estimated that inspections have been performed on about thirty buildings. Most were made during pre-purchase investigations. Some were required by refinancing, and a few others were initiated at the request of concerned building owners. Most of the investigations relied on ultrasonic testing in addition to visual inspection (Phipps, 1998).

Of the buildings inspected, five were found to have damage. The three most heavily damaged buildings are all located on soft soil where ground accelerations exceeded 0.20g. Each of the five buildings is at least 35 miles from the epicenter of the earthquake (Phipps, 1998).

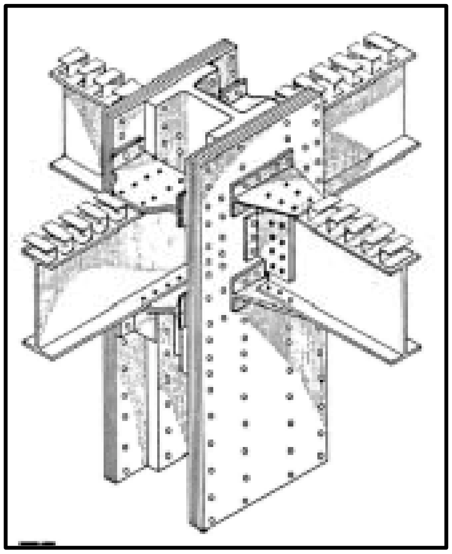


Figure 5-3 Typical Undamaged Joint in Torre Latino Americana, Mexico City Earthquake, 1985

Source: Osteraas and Krawinkler, 1989

The SAC Joint Venture brought this damage to the attention of Bay Area building officials and engineers with a special notice (SAC Steel Project, September 1996). The damage to the five buildings is tabulated and described in Table 5-5. Figure 5-4 maps the buildings' approximate locations.

Table 5-5 Damage to WSMF Buildings in the 1989 Loma Prieta Earthquake

Building description	Distance from epicenter (mi)	Connection damage (% of joints inspected)		Approximate repair costs
		Longitudinal	Transverse	
Building 1 6-story, 200,000 sf, 1989	37	<10%	50%	$2,500,000 (FEMA-267)
Building 2 12-story, 234,000 sf, box columns, 1978	57	15%	None	$630,000
Building 3 20-story, 400,000 sf	38	5% total		$300,000
Building 4 20-story	53	One connection		Not available
Building 5 14-story, under construction	50-60	28 connections total		Not available

Source, unless noted: Phipps, 1998.

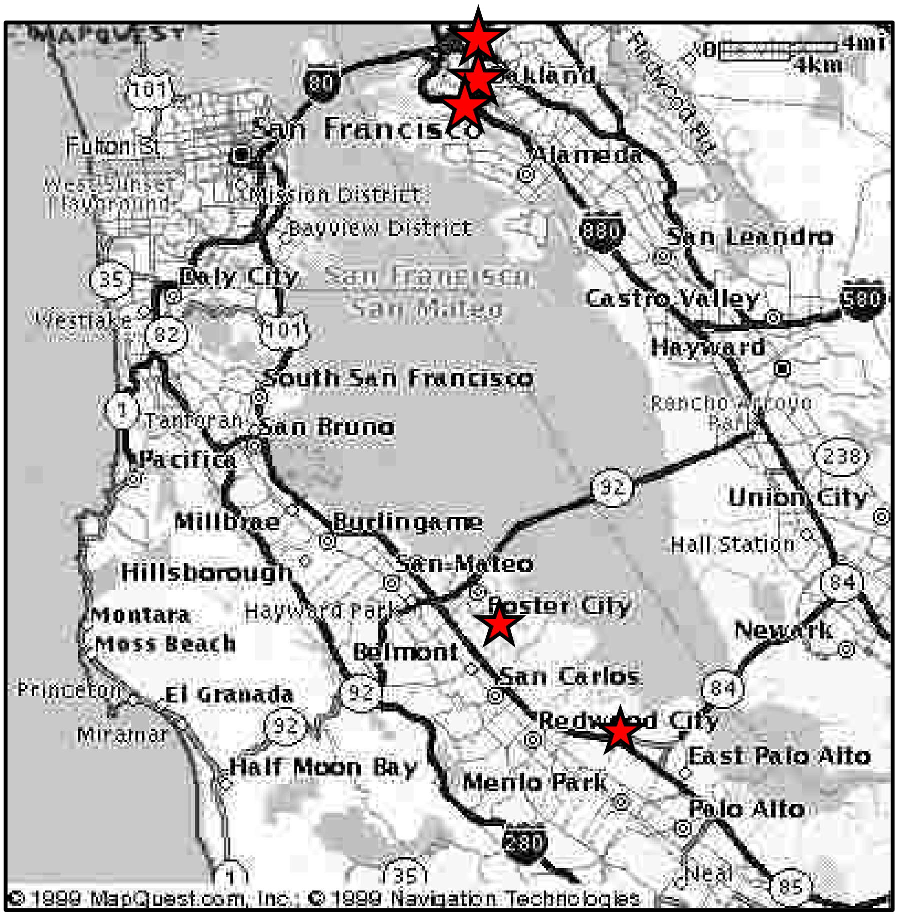


Figure 5-4 Location of WSMF Buildings with Known Connection Damage, Loma Prieta Earthquake, 1989

The following building descriptions are taken from Phipps (1998).

Building 1 is located on bay mud along the edge of San Francisco Bay. Constructed in 1989, it was nearly complete at the time of the earthquake. Peak ground accelerations of 0.29g (E-W) and 0.26g (N-S) were recorded approximately three miles from the site during the earthquake. Immediately following the earthquake, some minor cracking of architectural finishes was observed in the building and some movement of the glazing within window frames was found. The steel frame was not investigated until 1996. Initially, about sixteen connections were visually inspected. Twelve had readily observable damage. The investigation was expanded, and ultimately a total of 107 damaged connections were identified. Damage included girder bottom flange fractures, column divots, panel zone fractures, and girder top flange fractures.

Building 2 is also located on the edge of San Francisco Bay. A peak ground acceleration of 0.26g was recorded approximately 0.6 miles from the site. Minor damage to architectural finishes was observed and repaired following the Loma Prieta earthquake. In 1996, approximately 210 connections were inspected, and 41 were found with damage. Repairs were made using procedures consistent with the recommendations of FEMA-267.

Building 3 is located on soft soil, again along the edge of the Bay. The nearest strong motion data, recorded five miles from the site, gave peak ground accelerations of 0.26g (N-S) and 0.29g (E-W). In 1998, the steel frame was investigated as part of a pre-purchase due-diligence survey following procedures of FEMA-267. One hundred and fifty connections were visually and ultrasonically inspected. Damage was found in eight connections at the third, fourth, and fifth floors.

The Building 4 site experienced an estimated PGA of 0.18g, recorded about one mile away. An inspection of the building immediately following the earthquake revealed damage to one WSMF connection in a stairwell. The observed damage was reported to be similar to the damage found after Northridge. After the Northridge earthquake, a limited investigation of the frames was undertaken, and no additional damage was found.

Building 5 was under construction at the time of the Loma Prieta earthquake. It is located in downtown Oakland and uses a dual system of concrete shear walls and WSMFs. The nearest free-field record was located less than a mile from the building, and the recorded peak ground acceleration was 0.18g. During the earthquake, about half of the freestanding columns fell over, and almost all of the perimeter beams at the fifteenth and sixteenth floors fell off their bolted seats. From the seventh to the tenth floor, several of the bottom flange welds in the N-S WSMF cracked. A total of twenty-eight cracked welds was discovered by visual inspection or NDT. It was reported that the vast majority of the cracked welds were made by one welder, who had been fired prior to the earthquake, and that most of the welds had not been inspected. Detailed investigations conducted on the damaged steel connections revealed cracking of the same types found in the Northridge earthquake. Samples taken from the building showed a lack of fusion at the root pass, particularly in the E-W moment frames, which had been inspected prior to the earthquake.

5.10 Landers and Big Bear, 1992

The following information is taken entirely from Anderson and Bertero (1997).

The 1992 Landers earthquake was the second largest in California in the 1900s, measuring 7.3 on the Richter scale. It was closely followed by a magnitude 6.8 event about twenty miles away in Big Bear. Damage, however, was not as significant as in either the Loma Prieta or Northridge earthquakes because the epicenter was far from a major metropolitan area. Nonetheless, damage estimates exceeded $100 million.

A two-story steel frame building was damaged in the city of Big Bear Lake, although damage to the steel frame was not recognized immediately. Only after Northridge was the building studied in more detail. It was built in 1986 with several lines of moment frames in each direction. Cracks were found in many of the moment frame connections. They were severe enough that the owner decided to retrofit the building with new braced frames, rather than restore the frame to its original strength and stiffness. Near field effects and directivity may have contributed to the observed damage.

6. PERFORMANCE OF WSMFs IN THE 1994 NORTHRIDGE EARTHQUAKE

The 1994 Northridge earthquake was the event that triggered the nationwide study of WSMF seismic performance. Confidence in the performance of steel frame buildings seemed to increase incrementally with each major earthquake from 1906 to 1992. As described above, however, observations from past events and from research prior to 1994 suggest that the problems observed after Northridge were not new and should not have been wholly unexpected. Still, the quantity and severity of the 1994 damage was disturbing. “The Northridge earthquake of January 17, 1994, has fundamentally shaken engineers’ confidence in the seismic performance and safety of WSMF buildings” (Mahin et al., 1996).

By mid-1994, the engineering and research community had outlined a program of data collection to determine the extent of the problem. In the years since, dozens (if not hundreds) of articles, technical papers, and research reports large and small have been published on the topic of steel frame connections alone. Many, including even some studies sponsored by SAC, were based on preliminary data and are already obsolete. Others were prepared concurrently and are therefore incomplete, unable to reference contemporary findings, whether supportive or contrary. This is to be expected. In a few more years, perhaps, we will be removed enough from the event to recognize its lasting lessons. For now, this section summarizes briefly the latest data available, including damage counts. Readers are urged to consult the original sources and authors.

Without question, pre-Northridge WSMFs must now be considered more vulnerable than they were thought to be. But much of the research since the event paints a less dramatic picture than the one that emerged in 1994. Some important Northridge lessons that can be drawn at present are:

- Over the population of WSMF buildings, actual earthquake damage was far less extensive than suspected in mid-1994. Nearly half of all inspected buildings had no connection damage at all.
- Despite a low overall damage rate, some WSMF buildings were left in damage states that must be considered hazardous. Those with very high fracture rates were left vulnerable to future earthquakes. Those with fractured column webs or severely damaged shear connections may have posed local collapse hazards in aftershocks.
- The quality of pre-Northridge welding and inspection was poor in enough buildings that it cannot be considered an anomaly. Even if nonconforming practices did not directly cause damage, their widespread presence indicates how engineers, contractors, and inspectors had not been in complete control of WSMF design and construction.
- For typical WSMF buildings, the presence of connection damage is not predictable from broad building attributes known in advance (height, frame configuration, age, etc.).
- Structural connection damage cannot be ruled out in the absence of nonstructural “indicators,” such as elevator or partition damage. Damage does correlate mildly with broad

ground motion parameters, but not closely enough to raise the threshold for postearthquake inspection. Damage is predictable from analysis, but only in a probabilistic sense; the relationship is strong enough to aid postearthquake inspection in large buildings.

6.1 Early Findings and Engineering Response

Welding contractors working on buildings still under construction were the first to discover the damaged WSMF connections (Gates and Morden, 1995; Buildings 9070 and 9054 in Appendices A and B). "Damage was first identified by examining the distance between the back-up bars and columns in buildings that were visibly damaged. Others reported that in some buildings, elevators were not functioning properly, and in the process of examining them, the damage to welded joints was discovered" (Goltz and Weinberg, 1998). SAC case studies by Green and Hajjar et al. (see below) have described the process of finding unexpected fractures. Typically, there were no obvious signs of structural distress. An April 1994 article described some weld damage discovered "during routine tenant improvement work" (*Modern Steel Construction*, 1994).

Some early published reports underestimated the damage, capping it at "perhaps as many as a dozen" structures, and noting that it had mostly been repaired within a few months of the earthquake (*Modern Steel Construction*, 1994; AISC, 1994). These figures were low; published estimates that followed were almost certainly high. Gates and Morden (1995) have described how the damage count rose steadily through 1994. Most interesting and instructive, however, is how the count continued to rise through estimates, speculation, and misunderstanding. First, nearly all inspections through 1994 counted weld flaws as damage. As described below, these "W1" flaws are now considered pre-existing conditions. But at the time, they accounted for more than half of all the "damage" found. Several buildings were considered extensively damaged even though only W1 flaws had been found.

Second, a tentative list of WSMF buildings identified by the City and a list of buildings *scheduled* for inspection were probably both misunderstood at one time or another as lists of *damaged* structures. Although the two leading testing firms in Los Angeles estimated in early 1995 that they had already inspected about 200 steel frame buildings between them (Gates and Morden, 1995), that number is almost certainly incorrect. At that time, inspections in Los Angeles had not yet been mandated, and even with contributions from twenty-five engineering firms, the SAC survey had identified fewer than 100 inspected buildings, many of which were undamaged (Bonowitz and Youssef, 1995). Nevertheless, articles and presentations too numerous to mention, even some by highly knowledgeable authors, managed to repeat the phrase "over 100 damaged buildings" as established fact. An article in the October 1996 newsletter of a Northern California engineers association even claimed *two* hundred damaged WSMFs.

The inflated damage counts, as well as the mistaken impression among some engineers that the worst damage patterns were typical, certainly changed some minds about steel frames. A year after the earthquake, despite no steel frame collapses or casualties, some engineers familiar with the issues had come to regard WSMFs as less likely to provide Life Safety performance than well-designed concrete shear wall buildings (Gates and Morden, 1995).

Now, the figure of a hundred verified damaged WSMF buildings is probably correct. By May 1998, the City's records indicated only about 230 inspected buildings, 90 of which reported no damage at all (Los Angeles Department of Building & Safety, 1998). Many more reported only one or two "damaged" connections from their first phase of inspection; undoubtedly, many of these had weld flaws only. Inflated figures are still frequently cited. Even SAC reports have recently cited over 200 Northridge-damaged steel frames, a figure that is unsupported (Goltz and Weinberg, 1998).

If the damage numbers were exaggerated, the findings of inadequate fabrication and welding probably were not. Postearthquake joint inspections revealed widespread conditions of poor fit-up, improper joint preparation, undersized weld access holes, unacceptable weld weaving and bead thickness, use of weld dams, and other nonconforming practices. Some inspectors felt that the poor construction quality was especially prevalent in low-rise buildings (Gates and Morden, 1995).

6.2 New Regulation

In response to the damage (real and imagined) and unexpected questions about in-place weld quality, the City of Los Angeles organized a Steel Frame Building Task Force of local structural engineers and Building & Safety Staff. Membership quickly grew to include researchers, contractors, steel and welding industry representatives, and the Building Owners & Managers Association (Gates and Morden, 1995). Task Force members shared, confidentially, the data available to them, and developed tentative procedures for inspection, evaluation, and repair. Many of these are reflected in SAC Advisory 3 (SAC 95-01). Research had also begun. Principal among these efforts were full scale tests of connection specimens that matched the conditions where some of the first fractures were found (Engelhardt and Sabol, 1994) and a systematic and centralized data collection effort (Youssef et al., 1995).

Meanwhile, the City had ten subcommittees and Task Forces looking into the performance of other structural materials and systems. By the end of 1994, the Department of Building and Safety would enforce emergency measures regarding wood frame construction, reinforced concrete structures, and tilt-ups, as well as WSMFs (Deppe, 1994). As a result of Northridge, the City would ultimately adopt new code provisions for voluntary earthquake hazard reduction in hillside structures, wood cripple walls, and infilled concrete frames (ICBO, 1999).

The many WSMF-related technical guidelines, directives, and code interpretations to arise from the early efforts of the Los Angeles Task Force and others included the following. These were eventually compiled into or superseded by SAC Advisory 3 (SAC 95-01) in February, 1995, the SAC Interim Guidelines (FEMA 267) in August, 1995, and subsequent building codes and standards.

- City of Los Angeles, March 18, 1994: The first post-Northridge requirements for welding in repair and new construction, calling for reinforcing fillet welds after removal of backing bars, and specifying "small diameter wire electrode" for new full penetration welds.
- City of Los Angeles, May 11, 1994 (revision): Specific welding and procedure requirements for repair and new construction.

- County of Los Angeles, July 25, 1994: "Emergency Regulations" for repair, and suspension of the prequalified connection for new construction.
- City of Los Angeles, August 1, 1994: "Effective immediately, the use of Section 2710(g)1B of the *Uniform Building Code* for the design of girder-to-column connections is suspended."
- *Building Standards* (1994): A one-page article describing "the emergency code change action taken by the ICBO Board of Directors on September 14, 1994." The code change essentially replaces the prequalified WSMF connection with a requirement for designs supported by cyclic test results.
- City of Los Angeles, December 27, 1994: Interdepartmental Correspondence setting forth City building department requirements for welding in repairs and new construction.
- DSA, March 17, 1995: Interpretation by the California Division of the State Architect, commenting on the recent deletion from the state building code of the prequalified WSMF connection. This 34-page document includes some of the earliest post-Northridge cyclic testing requirements and acceptance criteria.
- AWS (1995): Recommendations of an AWS Task Group for changes to the D1.1 Structural Welding Code regarding WSMF weld design, fabrication, etc.

After the Interim Guidelines came a number of model building codes and standards that addressed aspects of WSMF design with a post-Northridge perspective and in light of early post-Northridge research. Principal among these were FEMA-267A, a supplement and update of the Interim Guidelines, and the AISC *Seismic Provisions* (1997). The AISC provisions included an appendix dedicated to cyclic testing of beam-column connections.

The Reliability/Redundancy Factor, ρ (rho), in current building codes was motivated in part by early observations of Northridge WSMF damage. The ρ factor represents a substantial change in the codified seismic design philosophy for moment-resisting frames. It was developed and introduced concurrently in the 1996 SEAOC Blue Book, the 1997 *UBC* (ICBO, 1997), the 1997 NEHRP provisions (FEMA 302 and 303), and FEMA-267A. The commentaries to those documents describe its motivation and its intended effect. For a typical low-rise WSMF office building with a floor plate of about 25,000 square feet, the *UBC* ρ requirements would effectively require frames providing 12 to 16 WSMF connections in each principal direction at each floor (for example, three or four two-bay frames, or six one-bay frames).

While ρ was motivated by general trends in the design of all types of structures, the 1996 Blue Book commentary refers specifically to "findings by the SAC joint venture (sic)." Indeed, some of the first WSMF damage found after the earthquake was in a five-story building in construction with one-bay frames (building 9070 in Appendices A and B). However, as described below, further work with Northridge damage data has not found a useful correlation between damage and structural redundancy.

6.3 Social, Economic, and Political Effects

Changes in WSMF engineering practice were accompanied by financial, legal, and political effects. Perhaps the first of these was a change in working relationships between engineers, contractors, and inspectors. Market demands put some immediate pressure on steel contractors, stalled designs of new steel frames, and spurred development of new proprietary details. The City ordinance mandating inspection of some WSMFs offered useful lessons on politics and the legislative process. Ultimately, financial losses led to lawsuits.

6.3.1 Changes in Practice

Goltz and Weinberg (1998) investigated the ramifications of sudden WSMF damage on the market for qualified welders, steel fabricators, and inspectors. They concluded that the regional market was able to withstand unexpected pressures, in part because Los Angeles' mandatory inspection ordinance (discussed below) was limited in scope and was implemented in phases.

At the same time, inspectors, welders, and engineers came to realize how little they knew about each other's work. Engineers, for example, did not know why E70T-4 electrodes were routinely used or to what effect backing bars were left in place. Welders did not know the specific requirements for the processes and electrodes they were using. Neither did the inspectors who checked their work. Engineers could not gauge the significance or the validity of ultrasonic test reports. None of the three groups was sufficiently familiar with the Welding Procedure Specifications they were each supposed to have approved. Indeed, production of a WPS had been recognized as essential but often overlooked (Putkey, 1993). Much of this changed, or was expected to, after Northridge. (For more discussion on the nature of early findings and changing relationships between various parties, see Gates and Morden, 1995; Goltz and Weinberg, 1998; and SAC 95-01.)

Uncertainties in the wake of the earthquake changed the relationships between engineers and building owners as well. The technical questions—Are the frames damaged? Are the damaged frames safe?—had no certain answers and quickly led to questions of liability, insurance, due diligence, construction scheduling, and budgets. Engineers could not perform limited intrusive investigations, recommend remedial measures, or even complete in-progress designs with any assurance that local building departments would not later require something different. Except for repairs to the most severely damaged frames, the result was an industry-wide "wait and see" policy. While engineers and owners waited, insurers were apparently not bothered; Goltz and Weinberg report a "disappointing level of detachment" among the insurance professionals they surveyed.

New requirements for connection qualification tests affected new WSMF design throughout California for several years after the earthquake. With no standards or consensus acceptance criteria, building departments were reluctant to approve designs. Fast track projects could not afford the time it took to design and execute a series of tests. And if any of the tests were to fail, the cost of redesign and retesting could kill a project. Developers and engineers turned away from steel frames to other "proven" systems. By late 1996, a body of successful tests had developed, and some engineers began to cite the available results in support of new designs. Unfortunately many of those tests had intentionally chosen large member sizes to match the 1994 Engelhardt and Sabol tests. As a result, a disproportionate number of early tests involved heavy W36 beams and jumbo columns. New designs relying on those tests were thus constrained to the tested sizes. More important, the eventual requirement to provide two or three matching tests ignored questions of reliability, just as engineers and researchers had done in the years *before* Northridge (SEAONC, 1998; Bonowitz, 1999b).

Technical solutions had interesting nontechnical aspects as well. A few innovative engineers designed, tested, and began to market alternative connection details for both retrofit and new construction. To offset their costs, they sought patents for their designs. Two proprietary designs that would later undergo peer reviews and receive approvals from Los Angeles County and other permitting bodies were the “slotted web” detail (SSDA, 1996) and the SidePlate system (LACOTAP, 1997). Trade Arbed, a Luxembourg steel manufacturer, had patented a beam with shaved flanges for seismic applications in the 1970s but had never enforced the patent. The “dog bone” or “reduced beam section” (RBS) designs that would emerge later were similar to Arbed’s concept. Taiwanese researchers had also developed a proprietary RBS design. But the idea of a patented steel connection, to be designed by a sole-source contractor, was relatively new to California engineers (even though proprietary technologies for isolation and energy dissipation had been on the market there for years).

While the proprietary designs were in development, the rest of the steel construction and engineering world was following the recommendations of SAC, a federally funded joint venture. Because of its limited public funding, SAC did not support or participate in these proprietary efforts. However, SAC documents did acknowledge and refer to them.

6.3.2 Legislation and Public Policy

Building code changes and interpretations motivated by the Northridge steel fractures are discussed above. Nontechnical public policy actions that arose from WSMF issues included the following.

California Assembly Bill 3772 was signed into law after the 1995-96 legislative session. According to an engineering association newsletter, the law allows the California Building Standards Commission “to adopt emergency regulations outside the regular cycle for adopting building codes. The need for this measure was precipitated by the failures in steel moment frame structures after the Northridge earthquake” (*SEAOC Plan Review*, 1996).

Through the spring of 1994, new construction was proceeding with the old detail even in Los Angeles, as building departments had no authority and insufficient data to support a moratorium. On July 21, the L.A. City Council passed emergency ordinance 169949, granting “blanket authority” to the Department of Building & Safety (EERI, 1996). By August, the City and County had proscribed the prequalified pre-Northridge connection. In September, it was removed from the *Uniform Building Code* (*Building Standards*, 1994).

In Los Angeles, however, it also remained to find the damage and mitigate the hazards. The damage count was still rising, and many WSMF owners were understandably reluctant to have their buildings inspected. Their buildings were functional, and their nonstructural damage had been repaired. Finding damage would leave them in financial limbo until consensus repair standards could be developed (Smith, 1995). Building officials saw a need for mandatory inspection.

On February 22, 1995, the Los Angeles City Council passed ordinance number 170406 mandating connection inspections and repairs in some WSMF buildings. The story of its passage has been told by Gates and Morden (1995) and by an EERI White Paper (1996), from which the following chronology is taken:

- January 1994: The Department of Building & Safety (B&S) establishes task force groups to study performance of various building types. With the expectation of little damage, the steel frame task force is assigned to the busy Chief of the Building Bureau, Richard Holguin. Each task force head reported to Councilman Hal Bernson.
- Spring 1994: The Building Owners and Managers Association (BOMA) requests a seat on the steel task force group. BOMA advocates case-by-case inspections and repairs, as opposed to broad prescriptive procedures.
- June-July 1994: B&S consults with Bernson and drafts an ordinance mandating inspection and repair of all WSMFs in the city. Efforts stall when early tests suggest that simple repairs may be costly and no better than the previous condition. High repair costs are expected for residential buildings (which have more expensive finishes), and condominium owners contest the proposed repair ordinance.
- June 1994: For political viability, B&S cuts the geographic scope of the proposed inspection ordinance to areas where serious damage had already been found, and estimates that the number of potentially affected buildings would drop from 1000 to 300.
- July 1994: Under pressure from owners groups, B&S drops residential buildings—only about 30 out of 300—from the proposed ordinance.
- October 1994: B&S modifies the proposed ordinance, lengthening the time to comply. A minimum inspection scope is also removed to protect the City from potential liability in the event that no damage would be found.
- Autumn 1994: B&S lobbies individual Council members. Bernson now feels that the ordinance is too weak and not prescriptive enough, and he too must be convinced to schedule a vote.
- February 1995: With the 1995 Kobe earthquake as an anniversary reminder, the Council passes the ordinance by a 12-0 vote.

Ordinance 170406 has been incorporated into the City of Los Angeles building code as section 8908. Damage data collected as a result of Ordinance 170406 is discussed below and included in Appendices A and B. The final ordinance covered a geographic area that excluded some parts of the city with high concentrations of WSMFs. In particular, the highrise buildings downtown, some of which had been analyzed after the San Fernando earthquake, were excluded. The included areas were chosen as those where significant damage had already been found in WSMFs or other structure types. Although the shaking was lighter downtown than on the west side or in the San Fernando valley, it is possible that no damage was found in downtown structures because so little inspection had been done there early in 1994.

The Los Angeles County building code (Chapter 94) would later adopt a similar inspection and repair requirement. The county defined a geographical area that included Universal City and areas adjacent to Santa Clarita.

There are dozens of small, incorporated jurisdictions in and around the City of Los Angeles. Most have only a small number of WSMF buildings, if any. As of April 2000, some of the jurisdictions with or near known WSMF damage had taken the following steps:

- Burbank in 1998 adopted section 7-140 into its Municipal Code, requiring inspection per FEMA and SAC Guidelines and repair to the pre-earthquake condition, upon notice by the building official (City of Burbank, 1999). A total of ten notices were sent out, and about half of the notified buildings reported some significant damage (Sloan, 2000).
- Santa Monica adopted chapter 8.76 of its Municipal Code Article 8 in June, 1999. The provision requires inspection and demonstration of conformance to the latest FEMA and SAC Guidelines (City of Santa Monica, 1999). The time allowed for repairs is given as a function of the number of occupants. As of April 2000, the requirements had only been applied to WSMFs seeking permits for other work. The Building Official expects proactive notification of WSMF building owners to begin in July 2000 (Mendizabal, 2000).
- Glendale has not mandated any inspections. All WSMF connections in City buildings were inspected since the earthquake, however, and no damage was found. The Glendale Building Official estimates that his jurisdiction has more and taller WSMFs than Burbank (Tom, 2000).
- Santa Clarita, which has fewer than ten WSMFs, has not mandated inspections, but the Building Department did send letters advising inspections and did issue several permits to repair weld damage (Bear, 1999).
- San Fernando (Mendoza, 1999), Beverly Hills (Moon, 1999), and Simi Valley (McDonald, 1999) have not required inspections.

Outside of Southern California, the Northridge damage prompted investigations of some WSMF buildings that had been subject to strong ground motion in the 1989 Loma Prieta and the 1992 Landers and Big Bear earthquakes (see the section above on past earthquakes). While no inspections have been mandated, the fact of observed damage has been brought to the attention of Bay Area building officials and structural engineers (SAC Steel Project, September 1996).

Once the Los Angeles ordinance went into effect, the costs of repairs became a real issue. B&S staff speculated in 1995 that they might have to identify and develop financing options in order for the program to succeed (EERI, 1996). Indeed, in 1997, the City Council approved an “unusual” $200 million bond issue specifically to fund long-term loans for mandated WSMF repairs (ENR, January 27, 1997). Los Angeles had been through mandatory seismic hazard reduction before, when it addressed its thousands of unreinforced masonry (URM) buildings. Interestingly, URM retrofit costs fell as contractors gained experience; with WSMFs, whose repairs were mandated before any standards had been proven, costs rose (EERI, 1996).

6.3.3 Legal Implications

Financial losses to some WSMF building owners led to litigation involving design engineers, contractors, fabricators, erectors, electrode manufacturers, and insurance companies. Most cases concerned individual buildings, but one alleged $1 billion in damages to WSMF structures as a class. This class action was later voluntarily withdrawn by the owners who had initiated it. As of December 1999, no court had rendered a decision on any legal claim, but several lawsuits have been settled. About eight lawsuits remain pending.

The following details from the first of the individual lawsuits and the aborted class action (ENR, January 20, 1997 through February 10, 1997; September 1, 1997; February 23, 1998) offer some indication of the non-engineering response of the courts and various stakeholders to the unanticipated WSMF damage.

In April 1995, during repair of a five-story WSMF in Santa Monica, workers discovered previously undetected damage. The structural engineers assessed the newly found cracks and called for evacuation of the building's tenants. (The building is number 9017 in Appendix A.) In January 1996, the owners filed suit against the original general contractor, steel fabricator, inspector, and structural engineer, seeking $10 million. What followed:

- May 1996: The Santa Monica plaintiffs amend their suit to include Lincoln Electric, the nation's leading maker of welding materials, citing the prevalent E70T-4 electrode as a factor in the damage. A jury trial is scheduled for May 1997.
- January 1997: Lincoln is sued in a $1 billion class action. By September, Lincoln would be named in seven other individual building suits.
- August 1997: Lincoln settles with the Santa Monica plaintiffs for $6 million. The suit against the original defendants remains, with a trial scheduled for January 1998. Lincoln attempts to recover its loss from the other defendants, but later abandons the effort.
- February 1998: The remaining Santa Monica defendants settle for a combined $5.5 million.
- February 1998: Lincoln remains a defendant in eight lawsuits. The class action is eliminated, perhaps, ENR speculates, because identical causation could not be shown.

The March 1997 issue of *California Construction Law* featured a series of articles that debated the charges against Lincoln (Castro, 1997; Jenks and Ritts, 1997). They offer a decidedly non-technical perspective on research, design, and the meaning of structural performance. Comparing pre-Northridge connections to faulty automobile airbags, Attorney Joel Castro presents a variety of "theories of recovery" premised on the claim that Lincoln's ubiquitous E70T-4 electrode was defective. In particular, he argues that mere repair, like tape over a punctured airbag, is inadequate compensation. He quotes (without citation) from Lincoln's marketing materials: "Where buildings must be designed to withstand seismic disturbances, Innershield is the architect's choice." (Innershield is the name of a Lincoln product line that includes their E70T-4 electrode.) The brittle weld metal and Lincoln's support of it were "substantial factors" in the damage, he argues, so Lincoln bears responsibility.

Lincoln's attorneys respond that too many factors were at work for the weld metal alone to be held responsible. Besides, they argue, E70T-4 specifications never included a notch-toughness requirement. Taking up the airbag analogy, they characterize the pre-Northridge connection as an accident waiting to happen:

> Imagine a person driving down an icy mountain road on a foggy winter night. He is speeding, his tires are bald, his brakes are worn, he is not wearing a seat belt. When he bought the car he chose to buy an ordinary air bag, not the heavy-duty one that would have cost an extra $1,000. He slides into a guard rail at a speed guaranteed to produce injury whatever the air bag chosen. He is injured—and he sues the airbag manufacturer. The lawsuit has no chance of success.

6.4 Damage Data

How bad was the damage? This has been a principal question since the first fractures were discovered in the spring of 1994. Unfortunately, the verifiable answer has changed as the scope of the problem became known, as inspections went from voluntary to mandatory, as certain damage types were discounted, and as analyses and case studies attempted to describe damage in the context of structural performance. The *speculative* answer was even more difficult to quantify, as photographs of the most severe damage circulated and as lists of buildings scheduled for inspection were mistaken for lists of buildings damaged.

In mid-1999, SAC researchers compiled and cross-checked a master list of over 200 WSMF buildings inspected after the Northridge earthquake (Maison and Bonowitz, 2000). The source lists, described in Table 6-1, varied in their size, completeness, and intended use. Appendices A and B give the master list and a building by building damage summary. Tables B.2 and B.3 summarize the damage data from the master list.

Table 6-1 Source Lists of WSMF Buildings Affected by the 1994 Northridge Earthquake

Reference	Sponsor	Scope	Notes
Los Angeles Department of Building and Safety, 1998	City of Los Angeles	City of L.A., about 220 buildings in various stages of inspection	Departmental record tracking mandatory inspection and repair. Commercial buildings only. Specified areas within the city only. Hard copy data available from LAB&S in May 1999 was current only through May 4, 1998.
Youssef et al., 1995	NIST	All available data, 51 buildings	The first systematic post-Northridge data collection effort, from August through November 1994. Voluntary inspections only, with various inspection scopes.
Bonowitz and Youssef, 1995	FEMA, SAC	All available data, 79 buildings	Continuation, expansion, and completion of the NIST effort, ending in March 1995. Voluntary inspections only, with various inspection scopes. Damage reported for each set of connections in a "floor-frame."
Durkin, 1995	FEMA, SAC	Random survey of 150 buildings within 0.2g contour	Intended to characterize the local WSMF population and the response of building owners by late 1994.

Table 6-1 Source Lists of WSMF Buildings Affected by the 1994 Northridge Earthquake (continued)

Reference	Sponsor	Scope	Notes
Dames and Moore, 1998	FEMA, SAC	49 selected buildings, most with nearly complete inspection	Buildings in West L.A. and southern San Fernando Valley selected based on available damage and construction data. Data compiled for individual connections with specific damage types noted. Some buildings overlap with Bonowitz and Youssef, but Dames and Moore data likely to be more complete and updated.
Paret, 1999	FEMA, SAC	35 selected buildings, most with nearly complete inspection	Regular buildings in West L.A. and southern San Fernando Valley selected for study of W1 causes and effects. Data reported on a building level from review of postearthquake inspection reports.
Durkin, 1999	FEMA, SAC	100 randomly selected buildings	Intended to characterize the local WSMF population, the response of building owners, and repair costs and approaches by late 1998.
Maison and Bonowitz, 2000	FEMA, SAC	Compilation of all of the above	Compiled and cross-checked for loss estimation study. Damage collected on a building level, with site specific ground motion data added. See Appendices A and B.

The master list summarized here and given in Appendix A is believed to be representative of the greater Los Angeles WSMF population. For purposes of regional impact studies and loss estimation, Seligson and Eguchi (1999) used Assessor’s records to estimate the number of WSMF buildings in Los Angeles County. They made some assumptions about age and lateral systems and concluded that the steel buildings covered by the City of Los Angeles ordinance are representative of the complete class of steel frame buildings identified from the Assessor’s records. Therefore, since the master list is made largely from buildings covered by the L.A. ordinance, the damage data may, for predictive purposes, be reasonably extrapolated to the wider population of moment frames. Nevertheless, there are some significant differences to keep in mind:

- The Los Angeles ordinance mandated inspection of commercial buildings only. The collected data might not be representative of other occupancies. For example, according to Seligson and Eguchi, steel-framed residential structures have total floor areas much smaller than typical office buildings. Some hospital buildings may also be missing.
- The data summarized here (and listed in Appendix A) include about two dozen inspected buildings from jurisdictions other than the City of Los Angeles, such as Santa Monica and Santa Clarita. These buildings were inspected voluntarily, sometimes because there was substantial nonstructural damage. They are more likely to have been damaged than an average or random WSMF in the same area. On the other hand, because jurisdictions outside the City of Los Angeles might not have mandated inspections, there may be damaged buildings in those areas that are missing from the Appendix A database.
- Though statistically representative, the Los Angeles ordinance data does exclude some parts of Los Angeles with high concentrations of WSMF buildings, notably downtown, the eastern

half of the mid-Wilshire district, and the area around LAX airport. WSMFs in these areas generally were not inspected after the earthquake.

- By excluding the downtown area, the collected data probably does not offer a good data sample for buildings taller than about twenty stories.

In Tables B-2 and B-3, the 155 buildings with at least 16 inspected WSMF connections are counted. Small buildings with fewer than 32 connections are included if 50% of their connections were inspected. This eliminates the few buildings whose inspection was so nominal as to be deemed inconclusive. (Note that these screening criteria differ from those used in the text and figures of Appendix B.) The damage rate is counted as the number of damaged connections divided by the number of connections inspected. Each connection consists of two beam flange welds and a beam web connection. The connection is considered damaged if there is fracture at or near either the top or bottom weld, whether the cracking is in the beam, the column, or the weldment. Weld flaws, labeled W1 or W5 in SAC documents, are not counted as damage. These flaws, some of which are acceptable by AWS standards, are now widely believed to have pre-dated the earthquake (Bonowitz and Youssef, 1995; Paret, 1999). Typical W1 flaws are planar discontinuities at the weld-column interface, frequently up to ¼" in height and as long as half of the weld length, sometimes longer.

6.4.1 W1 Flaws

The structural significance of W1 flaws is not yet fully resolved. The principal question is whether original flaws increase the likelihood of earthquake damage. Readers are referred to Paret (1999) and Kaufmann et al. (1997) for more on the detection and interpretation of W1 flaws. Some basic findings on the subject from post-Northridge data collection and analysis include:

- Until W1 flaws were shown to have predated the earthquake, they accounted for about two thirds of all the so-called damage (Paret, 1999).
- Within the greater L.A. population, one can now expect to find W1 flaws in about 15% of existing WSMF connections (Bonowitz and Youssef, 1995; Paret, 1999). The original occurrence rate was probably higher, since some original flaws certainly grew into full-fledged fractures in the earthquake (Paret, 1999; Kaufmann et al., 1997).
- Ultrasonic testing (UT) is not well suited to finding W1 flaws due to technical limitations and unreliable application (Paret, 1999). UT has been used for field inspection of WSMF welds since the late 1960s, but UT findings have always been highly dependent on the skill of the operator (Couch and Olsson, 1965; Preece, 1981; Preece and Collin, 1991). Nevertheless, engineers and contractors appear to have relied on the technique almost exclusively, and this may have invited abuse (Goltz and Weinberg, 1998). Under pressure during construction, UT technicians may have been predisposed to read flaws as the reflection of the backing bar gap. After the earthquake, technicians may have felt similar pressure to find damage.

6.4.2 Damage Data

Table 6-2 summarizes the damage data given in Appendix B. In Table 6-2, the building's damage rate, DR, is equal to the number of connections found damaged divided by the number of connections inspected. Each connection represents the joining of one beam to one column. A typical connection comprises two beam flange welds and a bolted or welded beam web. A connection is considered damaged if any part of it is damaged.

The most typical damage involved fractures at or near beam flange-to-column flange welds. In the worst cases, fracture extended through the column flange into the column panel zone. In other critical cases, the beam flange detached from the column completely (or nearly so), and damage to the beam shear connection—either the bolts or the shear tab—followed. These last two damage classes, shear and panel zone, were especially rare; Table 6-2 gives the number of inspected buildings in each height range with even one incident of shear or panel zone damage.

A few observations on the summarized data:

- About 40% of buildings overall and in each height range had no connection damage at all. Among the 1-story buildings, 11 of 13 were undamaged.
- Overall and in each height range, the median damage rate is around 5%, and the worst damage is around 50%. Even with the potential impact of clustered damage, it is unlikely that more than four or five of these 155 buildings would have been classified as hazardous by the 2000 SAC Guidelines. (The Guidelines require a determination of damage severity at each inspected connection and of the expected damage rate within each critical *group* of connections. A hazard is recognized if any group has an expected capacity loss exceeding 50%. A more complete study of the damage with respect to “tagging” criteria is warranted.)
- About a quarter of all damaged buildings had at least one damaged shear connection. The mechanism of failure is such that shear damage never happens without flange fracture. In a building with shear damage, typically less than 5% of the building’s connections and less than a third of its *damaged* connections are affected, although in the hardest hit buildings the numbers are higher. In many cases, typical shear damage does not affect the connection’s gravity capacity, since most of the bolts are needed not to carry expected gravity loads but to develop the beam’s flexural strength. The type of shear connection is not reported in much of the collected data. However, buildings erected before 1975 are likely to have fully-welded beam webs, and those built after 1988 are likely to have high strength bolts and supplemental shear tab welds. Most of the surveyed buildings have shear connections with bolts but without supplemental welds (Bonowitz and Youssef, 1995).
- Shear connection damage in *non*-moment frame connections was observed in some heavily damaged buildings (including the Borax building described below). Astaneh and Liu (1999) have studied the deformation capacity of pre-Northridge single plate shear connections.
- About a third of all damaged buildings had at least one damaged column panel zone. Panel zone fracture only happens if the column flange fractures first. In a building with panel zone damage, up to 50% of the damaged connections may involve some fracture into the panel zone. In the worst cases, panel zone fractures severed the column over nearly its entire depth. Most of the panel zone damage was less severe, typically involving a fracture that extended just barely into the column web. Anderson, Johnston, and Partridge (1995) have studied the residual capacity of severely damaged columns with panel zone fractures, using specimens taken from their case study building, described below.

- The data summarized here and presented in Appendix B have not been analyzed with respect to the location of damage within a building. Previous studies with a preliminary data set suggested that damage does tend to cluster, that 3- to 4-story buildings tend to have more damage at lower floors, and that buildings taller than 18 stories had especially light damage, if any, in their lowest eight floors (Bonowitz and Youssef, 1995).

Despite relatively low damage to the WSMF population as a whole, it is important to note that there was serious damage to a wide variety of steel frame buildings. Among the most heavily damaged were:

- Building 9069 (see Appendices A and B), a one-story frame with column flange and panel zone damage to about half of its twenty connections.
- Building 9068, the Borax building described below, a four-story building with a 75% damage rate and 21 panel zone fractures in its 112 connections.
- Building 9017, St. John's Medical Plaza (the subject of the lawsuit described above), a five-story building with 50% damage and ten panel zone fractures in 96 connections.
- Building 9008, a ten-story frame with 26% of its connections damaged, including 23 out of 688 with panel zone fractures.

Table 6-2 Number of WSMF Buildings with Various Northridge Earthquake Damage Rates

	1 story	2-4 story	5-12 story	13+ story	All
All buildings	13	69	47	26	155
No damage	11	26	16	12	65
0 < DR ≤.05	0	7	6	5	18
.051 < DR ≤ .10	0	10	8	1	19
.11 < DR ≤ .20	0	12	11	6	29
.21 < DR ≤ .50	2	13	4	2	21
DR > .50	0	1	2	0	3
Shear damage	0	9	10	4	23
Panel zone damage	1	16	8	4	29

6.4.3 Using the Damage Data

Collected damage data is most useful if it can address two questions: Where did damage occur in the last earthquake, and where will damage occur in the next one?

Appendix B shows the general scatter of damage rates with respect to demand. Additional plots are included in Maison and Bonowitz (2000). The observed scatter is, of course, due in large part to variability in design, construction, and material quality. Two other major contributors to the scatter are the large number of buildings with no damage and substantial uncertainty in demand estimates.

Preliminary statistical analyses of the Appendix B data were performed in mid-1999 for purposes of developing loss estimation models (Maison and Bonowitz, 2000). The analyses first considered whether building damage rates are significantly related to the following global demand estimates: peak ground acceleration, peak ground velocity, spectral acceleration, spectral displacement, spectral displacement divided by building height (a surrogate for drift), and Modified Mercali Intensity. Derivation of the various building-specific demand estimates is described in Appendix B and its references.

Given the demand estimates and known damage rates, one-sided *t*-tests determined whether the mean demands of buildings in various damage ranges were significantly distinct from each other. Chi-squared tests determined whether higher demands were associated with higher damage rates to a degree greater than would be expected by chance.

Of the various demand estimates, peak ground acceleration (PGA) and spectral displacement were found to correlate best with the observed damage. However, these damage-demand relationships are clear in a probabilistic sense only. That is, there is no identifiable ground shaking level above which WSMFs are damaged and below which they are not. Nevertheless, the statistical link between damage and demand supports the notion of a demand-based postearthquake inspection trigger. It also requires that studies of potential damage predictors must use data with statistically equivalent demands.

Within a narrow demand range, are certain building configurations more prone to damage? Analysis of the Appendix B data found no strong correlation at all between damage and building height. While a previous analysis with preliminary data found a strong relationship in low-rise buildings between damage and floor area per connection (Bonowitz, 1998), the 1999 analysis of the updated and more complete data did not support that pattern.

Because the Appendix B data was compiled principally to study loss estimation, it was not broken down by location in the building, by floor or frame, by member size, etc. Therefore, it supports only very limited correlation studies. Analysis of SAC Phase 1 data has suggested some useful relationships (Bonowitz, 1998), but failure of the newer, larger data set to support one of them (area per connection, as noted above) suggests that the following should still be considered preliminary:

- More damage occurred in the lowest floors of 3- and 4- story buildings.
- Connections with supplemental shear tab welds (as opposed to bolted or fully welded beam webs) appear to have been more prone to damage. This may be an indirect predictor, as supplemental shear tab welds were required only from 1988 to 1994 and only for the lightest wide-flange sections of a given depth.

- There is mixed evidence that 1- and 2-bay frames were more prone to damage than multi-bay frames.
- Far more damage was observed at beam bottom flanges than at top flanges. Composite behavior is at most only partly responsible for the discrepancy. The relative ease of welding and inspecting top flanges may explain some of the difference. Top flange fractures happen at a greater rate in lab testing than the postearthquake field observations would predict. It is also likely that much top flange damage was not found in buildings because, compared with easily accessible bottom flanges, top flanges were less frequently and less completely inspected.
- After an earthquake, the general presence or absence of nonstructural damage or of non-WSMF structural damage is not a useful indicator of WSMF connection fractures. All of the highly damaged WSMFs had both nonstructural and other structural damage, but the data was not robust enough to be statistically meaningful (Bonowitz and Youssef, 1995).

If damage cannot be reliably found or predicted by obvious building attributes, can postearthquake analysis help? A number of case studies have suggested that analysis *can* help locate the areas of a building most likely to be damaged, but again, the damage-demand relationship is probabilistic. Statistical analyses of case study data by Uang et al. and Naeim et al. in SAC 95-04 have shown that beam ends with higher computed elastic demand-capacity ratios are clearly more likely to have been damaged. Other case studies have shown the same probabilistic relationship (Bonowitz, 1998). On the other hand, there are cases of twin frames and twin buildings similarly shaken but quite differently damaged (for example, Paret and Sasaki, 1995). The reliability and usefulness of analysis is expected to be greatest in buildings with several hundred connections, where the data is robust and where postearthquake inspection is most in need of direction.

6.5 Case Studies

Several case studies of specific WSMF buildings have been completed, many with sophisticated computer modeling. Some actually included testing of damaged joints removed from the buildings. These studies and the research prompted by them played a substantial role in the development of FEMA-267.

Table 6-3 describes the published case studies known to date. The earliest of these were published in SAC documents 95-04 and 95-07. Researchers are encouraged to consult the original reports for details. The 95-04 studies have been summarized and analyzed separately (Deierlein, 1995; Bonowitz and Youssef, 1995; Bonowitz, 1998). Extended reports for some of them have since been published as university research reports, and many shortened versions have been published in journals and conference proceedings. Some of the listed case study buildings have also been the subjects of parameter studies and experimental analyses by others (for example, Maison and Kasai, 1997; Song and Ellingwood, 1999).

In addition to case studies of actual buildings, SAC also designed a matrix of generic “model” pre-Northridge WSMF buildings for intensive parameter studies by SAC (Krawinkler, 2000; Cornell and Luco, 1999) and others (Maison and Bonowitz, 1999).

Table 6-3 Case Studies of WSMF Buildings Affected by the 1994 Northridge Earthquake

Building or Reference	**Building ID** (Appendix A)	**Description**	**Recorded or estimated PGA [g]**	**Connection Damage**
SAC 95-04				
Krawinkler et al.	9096	Northridge, 1993, 2 stories	0.40	no damage
Krawinkler et al.	9095	Northridge, 1993, 4 stories	0.40	13% damage rate, shear damage
Engelhardt et al.	9075	Santa Monica, 1988, 6 stories	0.64	59% damage rate
Hart et al.	9059	Woodland Hills, 1993, 5 story hospital	0.4 to 0.6	no damage
Hart et al.	9060	Woodland Hills, 1993, 5 story hospital	0.4 to 0.6	10% damage rate, shear and panel zone damage
Naeim et al.	9023	West Los Angeles, 1982, 11 stories	0.26 to 0.41	14% damage rate, shear and panel zone damage
Uang et al.	9107	Canoga Park, 1975, 13 stories	0.41	10% damage rate, shear damage
Kariotis and Eimani	9088	Encino, 1969, 16 stories	0.38	16% damage rate, shear and panel zone damage
Paret and Sasaki	9121	Canoga Park, 1987, 17 stories	0.41	9% damage rate
Paret and Sasaki	9122	Canoga Park, 1987, 17 stories	0.41	12% damage rate
SAC 95-07				
Santa Clarita City Hall (Green)	9098	Santa Clarita, 1986, 3 stories	0.59	unknown damage rate, shear damage
Borax Corporate Headquarters (Hajjar et al.)	9068	Valencia, 1993, 4 stories	0.6	75% damage rate, shear and panel zone damage
Anderson et al., 1995	9114	Santa Clarita, 1991, 2 stories	0.6	50% damage rate, panel zone damage

Table 6-3 Case Studies of WSMF Buildings Affected by the 1994 Northridge Earthquake (continued)

Building or Reference	**Building ID** (Appendix A)	**Description**	**Recorded or estimated PGA [g]**	**Connection Damage**
NISTIR 5944				
Kaufmann et al.: A	9084	Simi Valley, 1980, 6 stories	0.3	19% damage rate, panel zone damage
Kaufmann et al.: B	9022	West Los Angeles, 1984, 4 stories	0.2	14% damage rate
Kaufmann et al.: C	9021	Sherman Oaks, 1983, 4 stories	0.4	14% damage rate, shear and panel zone damage
Kaufmann et al.: E	9023	West Los Angeles, 1982, 11 stories	0.2	14% damage rate, shear and panel zone damage
Kaufmann et al.: F	9020	Sherman Oaks, 1985, 4 stories	0.4	33% damage rate, shear and panel zone damage
CSMIP				
Naeim et al., 1999	none	Encino, 20 stories	0.41	4% damage rate, panel zone damage
Naeim et al., 1999	none	Tarzana, 10 stories	0.47	2% damage rate
Naeim et al., 1999	none	North Hollywood, 8 stories	0.30	no damage
Naeim et al., 1999	9148	Sherman Oaks, 16 stories	0.45	2% damage rate
Others				
Islam et al.: A	9017	Santa Monica, 1987, 5 stories	0.6	50% damage rate, shear and panel zone damage
Islam et al.: B	9028? 9044? 9045?	West Los Angeles, 1981, 11 stories	0.3	26% damage rate, shear and panel zone damage
Bertero et al., 1994	9050	Sherman Oaks, 6 stories	not available	60% damage rate

Observations from the SAC 95-04 case studies provided guidance for the development of the FEMA-267 Interim Guidelines. Aggregate conclusions from those studies included:

- Analytical procedures can find general locations within buildings that are more likely to be damaged.
- Higher mode effects seem to have been the cause of concentrated damage in upper stories.
- Ground motions generated by the Northridge earthquake did not generate large inelastic joint rotation demands.

Following are brief descriptions of the SAC 95-04 and 95-07 case studies. Readers are encouraged to consult the references for details. Damage types refer to those defined in FEMA-267.

6.5.1 Krawinkler et al.

Adjacent two and four-story buildings with perimeter moment frames were evaluated and inspected. Both were designed by the same structural engineer under the provisions of the 1988 *UBC*. According to construction reports, 100% of the first 40 welds were ultrasonically inspected, and 25% thereafter. Both buildings had base shear capacities on the order of 3.5 to 4 times *UBC* demands. The four-story building was found to have damage to 14 bottom flange and two top flange connections out of approximately 120 total (91 inspected). Damage was confirmed with magnetic particle and ultrasonic testing. Typical failures included pullout of column flange material above the toe of the weld and cracks through the weld throat. The two-story building had no visible damage.

Researchers analyzed these buildings to determine whether the damage could have been predicted. Results were mostly inconclusive. High elastic demand-to-capacity ratios and interstory drift generally were fair predictors of damage, as was excessive inelastic deformation of the panel zones. However, while both the two-story and four-story buildings had relatively high inelastic deformation demands, only the taller one was damaged.

6.5.2 Engelhardt et al.

This six-story building in Santa Monica consisted of one- and two-bay frames with relatively large members (W24 to W33 beams and W14x176 to W14x193 columns at the base). The column sections did require doubler plates. The typical connection used welded flanges and bolted webs.

After the earthquake, discontinuities were found in 92 of 120 welded joints. The discontinuities were of various widths, but all stayed within the girder flange welds. About a third were classified as W1. Some of the W2, W3, or W4 fractures could have propagated from original W1 flaws.

6.5.3 Hart et al.

Two adjacent six-story hospital buildings were evaluated, one with observed weld fractures and one without. The two buildings were constructed at the same time, and each had both perimeter and interior frames. The damaged building was found to have 134 damaged connections, all but 19 of which were classified as type W1 or W2. Most of the remaining 19 had column divots. One also had a panel zone crack.

The buildings were studied extensively to try to correlate the observed damage with modeling parameters. The type W incidences appeared to be spread randomly throughout the building since they could not be accurately predicted by analysis. But the study also concluded that the locations of clear earthquake damage, such as type C fractures, were not well predicted.

6.5.4 Naeim et al.

This 11-story building in West Los Angeles was designed to the provisions of the 1979 *UBC*. 913 of 920 total connections were inspected, with damage observed in 258. The building exhibited few outward signs of obvious nonstructural or structural damage, although a one-inch permanent drift was measured.

Damage to the moment connections was more varied than in the buildings discussed above. While the most common type was the W1 flaw (41% of all incidences), column flange damage types C3 and C2 represented 25% and 18% of all damage incidences respectively. The remaining fractures occurred about equally in the panel zones and as type C5 tearing of the column flange. The damage patterns were well distributed throughout the building.

Researchers concluded after analysis that the overall correlation between observed and predicted damage was tenuous. Elastic demand-capacity ratios did predict damage better than other analysis parameters.

6.5.5 Uang et al.

This 13-story structure was located approximately three miles from the epicenter of the Northridge earthquake. The building has perimeter frames and was probably designed to the requirements of the 1973 *UBC*. Notably, the code design criteria at this time did not include specific provisions for panel zone strength. An analysis of the building following the Northridge earthquake indicated that substantial panel zone shear yielding should have been expected and may have contributed a significant amount of energy dissipation.

Damage in this building indicated a directionality of the shaking, as two parallel perimeter frames sustained significantly more damage than the two frames in the other principal direction. The researchers concluded that while analysis could not locate specific damage, elastic demand-capacity ratios could identify subsets of connections that were significantly more likely to have been damaged.

6.5.6 Kariotis and Eimani

This building has an aspect ratio of about 2.5:1. In the longitudinal direction, the lateral system consists of two perimeter six-bay moment frames. In the transverse (north-south) direction, there are two heavy three-bay moment frames with W14x370 to W14x420 base columns and 42-inch plate girders. The building was designed to the 1969 L.A. City code, whose base shear requirements were similar to those in the 1970 *UBC*. After the 1971 San

Fernando earthquake, base shear requirements were modified. If designed to the 1976 *UBC*, this building would have been designed for almost twice the base shear.

Damage was concentrated in the transverse frames. Widespread W1 and W2 incidences could not be consistently predicted by analysis. Nineteen type C fractures in the transverse direction were through the column flange, with some extending into the panel zone.

This building had also been analyzed after the 1971 San Fernando earthquake. See Table 5-3 (KB Valley Center) for additional information.

6.5.7 Paret and Sasaki

This 17-story building is an excellent example of trends toward less structural redundancy and very large moment frame sections. The building has two two-bay moment frames in each direction, with columns ranging from W14x311 to W14x730 and beams from W30x99 to W36x300. The weld fractures all occurred in the beam bottom flanges in one direction. Over 80% of the fractures occurred above the 9^{th} floor of the building.

6.5.8 Santa Clarita City Hall (Green)

Santa Clarita City Hall was among the first buildings to be found with Northridge earthquake WSMF damage. The building initially appeared to have experienced only non-structural damage, including partition and tile cracking, fallen ceiling panels, and overturned cabinets. After ceiling tiles and fireproofing were removed, inspectors found sheared bolts at several beam web-to-column connections. Bottom beam flange welds had also fractured. Repair of the damaged joints took ninety days and included a seven-day per week schedule with overtime. The total cost of the repairs was about $2,000,000.

6.5.9 Anderson, Johnston, and Partridge

This two-story structurally irregular building was near Santa Clarita City Hall, but its damage was more obvious. The building had a noticeable permanent drift of about 2% in the first story. All moment connections at the second floor were damaged. The majority of the damage appeared to have started at the bottom beam flange welds with cracks propagating through the column flange and in many cases into the panel zone.

The owner decided to demolish the building. Before demolition, the researchers were able to remove some of the WSMF joints for testing. Based on the tests, the researchers concluded that:

- Even in its badly damaged state, the beam-column joint had substantial ability to resist additional load cycles. While the researchers believe that the damaged building might have withstood moderate aftershocks, they could not conclude that it would have survived another major earthquake.
- The original strength and stiffness of the joints could be restored if the right repair methods were used. It was typically not sufficient to simply reweld the cracked bottom flange weld.

6.5.10 Borax Corporate Headquarters (Hajjar et al.)

This building was one of a two-building complex. No structural damage was immediately suspected from an initial postearthquake inspection. However, one of the two buildings experienced obvious permanent interstory drift, prompting inspection of the WSMF connections. When connection damage was found, inspection continued to the other structure. Damage was found in about 75% of all moment connections, but the second and third floor connections saw 100% and 93% damage respectively. Damage occurred only at girder bottom flanges. Cracks observed in top flange welds were determined to have occurred prior to the earthquake. The most common fracture types involved cracked welds or divots torn from the column face. In at least 23 locations, the cracks extended into the panel zones.

The researchers cited inadequate weld fusion as the critical factor in the damage. They found lack of fusion in three main locations: at the backing bar, at weld ends where weld dams were used, and at the weld access hole by the beam bottom flange where the welder must stop and restart the weld.

APPENDIX A. WSMF DATA FROM THE NORTHRIDGE EARTHQUAKE

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9001	21700	Oxnard St	Woodland Hills	91367	34.18	-118.60	22	156		78	5		
9002	21650	Oxnard St	Woodland Hills	91367	34.18	-118.60	25	16	ESI8	68	9		
9003	15260	Ventura Blvd	Sherman Oaks	91403	34.15	-118.47	22	65		43	10	65	
9004	14144	Ventura Blvd	Sherman Oaks	91423	34.15	-118.44	3	198			11		
9005	21041	Warner Center Ln	Woodland Hills	91367	34.17	-118.66	3	27		1	12		
9006	16650	Sherman Way	Los Angeles	91406	34.20	-118.50	2	206			13	206	
9007	21051	Warner Center Ln	Woodland Hills	91367	34.18	-118.59	2	80			15		
9008	9200	Oakdale	Chatsworth	91311	34.24	-118.56	11	4	BJ05	28	16		
9009	21550	Oxnard St	Woodland Hills	91367	34.18	-118.60	11	59			18		
9010	21800	Oxnard St	Woodland Hills	91367	34.18	-118.60	11	124		95	20		
9011	5950	Canoga Ave	Woodland Hills	91367	34.18	-118.60	6	152		5	21		
9012	5850	Canoga Ave	Woodland Hills	91367	34.18	-118.60	6	159		48	23		
9013	3301	Barham Blvd	Los Angeles	90068	34.13	-118.34	4	151			26		
9014	15821	Ventura Blvd	Encino	91436	34.16	-118.48	7	149		69	28	149	
9015	5990	N Sepulveda Blvd	Van Nuys	91411	33.98	-118.39	6	52			27		
9016	15315	Magnolia Blvd	Sherman Oaks	91403	34.16	-118.47	4	19	SOA	19	39		
9017	1301	20th Street	Santa Monica	90404	34.03	-118.48	5	na	BJ19		43		Islam/NIST
9018	20700	Ventura Blvd	Woodland Hills	91364	34.17	-118.58	3	20	MNH02?	60	42		
9019	11900	Olympic Blvd	Los Angeles	90064	34.03	-118.45	8	106			45		
9020	15165	Ventura Blvd	Sherman Oaks	91403	34.15	-118.46	4	6	JAM7484		49	6	Kaufmann/NIST
9021	15060	Ventura Blvd	Sherman Oaks	91403	34.15	-118.46	4	5	JAM7482		50		Kaufmann/NIST

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9022	12300	Wilshire Blvd	Los Angeles	90025	34.04	-118.47	4	71	JAM7485		51		Kaufmann/NIST
9023	11150	Olympic Blvd	Los Angeles	90025	34.04	-118.44	11	14	JAM7480		52		Naeim/SAC; Kaufmann/NIST
9024	5602	DeSoto Ave	Woodland Hills	91367	34.17	-118.59	5	162?			58		
9025	21860	Burbank Blvd	Woodland Hills	91367	34.17	-118.60	3	10		84	17		
9026	19809	Prairie	Chatsworth	91311	34.24	-118.56	2	182	BJ06		25		
9027	21600	Oxnard St	Woodland Hills	91367	34.18	-118.60	20	172		49	24		
9028	11444	Olympic Blvd	Los Angeles	90026	34.05	-118.24	11	86			59		Islam/NIST?
9029	16830	Ventura Blvd	Encino	91436	34.16	-118.50	6	31	NYA550		62		
9030	15301	Ventura Blvd	Sherman Oaks	91367	34.15	-118.46	5	210		22	14		
9031	20000	Prairie	Chatsworth	91311	34.24	-118.57	2	79			19		
9032	5900	Canoga Ave	Woodland Hills	91367	34.18	-118.60	4	153		4	22	153	
9033	11300	Olympic Blvd	Los Angeles	90064	34.04	-118.44	9	89			29		
9034	5200	Lankershim Blvd	North Hollywood	91601	34.17	-118.37	8	90			30		
9035	15350	Sherman Way	Van Nuys	91406	34.20	-118.47	4	216			31		
9036	6800	Owensmouth Ave	Canoga Park	91303	34.19	-118.90	5	51		8?	32	51	
9037	15400	Sherman Way	Van Nuys	91406	34.20	-118.47	5	217		8?	33		
9038	16030	Ventura Blvd	Encino	91436	34.16	-118.48	6	39	MNH04	88	38		
9039	19634	Ventura Blvd	Tarzana	91356	34.17	-118.56	3	171		74?	40		
9040	9045	Corbin Ave	Northridge	91324	34.23	-118.56	3	208		31	41		
9041	12121	Wilshire Blvd	Los Angeles	90025	34.04	-118.47	14	63	NYA577		44		
none			Simi Valley	93065	34.30	-118.74	6	na			47		
9043	12424	Wilshire Blvd	Los Angeles	90025	34.04	-118.47	13	70	JAM7486		48		
9044	11355	Olympic Blvd	Los Angeles	90064	34.04	-118.44	10	67.2		36?35?	53		Islam/NIST?

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9045	11355	Olympic Blvd	Los Angeles	90064	34.04	-118.44	10	67.1		35?36?	57		Islam/NIST?
9046	11835	Olympic Blvd	Los Angeles	90064	34.03	-118.45	12	8			63		
9047	1950	Sawtelle	Los Angeles	90025	34.04	-118.44	3	11	AC1				
9048	701	N Brand	Glendale	91203	34.16	-118.26	8	na	AC2				
9049	27125	Sierra Hwy	Santa Clarita	91351	34.41	-118.46	2	na	AV1				
9050	--	Riverside Dr	Sherman Oaks				6	115?	BAK				Bertero
9051	2796	Sycamore Dr	Simi Valley	93065	34.29	-118.74	2	na	BBRS1				
none	--	Riverside Dr & Woodman	Sherman Oaks	91403	34.16	-118.43	2		BC1				
9053	1919	Santa Monica Blvd	Santa Monica	90404	34.03	-118.48	4	na	BJ01				
9054	na	na	Universal City		34.14	-118.35	3	na	BJ02E				
9055	4730	Woodman Ave	Sherman Oaks	91423	34.16	-118.43	4	225	BJ04				
9056	5601	DeSoto Ave	Woodland Hills	91365	34.17	-118.59	3	162	BJ07		58?		
9057	5601	DeSoto Ave	Woodland Hills	91365	34.17	-118.59	3	162	BJ08		58?		
none	5601	DeSoto Ave	Woodland Hills	91365	34.17	-118.59	5	162	BJ09		58?		
9059	5601	DeSoto Ave	Woodland Hills	91365	34.17	-118.59	5	162	BJ10		58?		Hart/SAC 95-04
9060	5601	DeSoto Ave	Woodland Hills	91365	34.17	-118.59	5	162	BJ11		58?		Hart/SAC 95-04
9061	321	N Canon	Beverly Hills	90210	34.07	-118.40	3	na	BJ14				
9062	401	Wilshire Blvd	Santa Monica	90401	34.02	-118.50	13	na	BJ16				
9063	5550	Topanga Cyn Blvd.	Woodland Hills	91367	34.17	-118.61	3	37	BJ18				
9064	6041	Cadillac Ave	Los Angeles	90034	34.04	-118.37	4		BJ20				
9065	550	Wilshire Blvd	Santa Monica	90401	34.02	-118.50	4	na	BLC1				
9066	12020	Chandler	North Hollywood	91607	34.17	-118.40	4	?	CAB				
9067	101	Continental	El Segundo	90245	33.92	-118.39	15	na	DM1				

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9068	26877	Tourney	Valencia	92385	34.41	-118.57	4	na	EQE1				Hajjar/SAC 95-07
9069	26877	Tourney	Valencia	92385	34.41	-118.57	1	na	EQE2				
9070	1200	Getty Center Dr	Los Angeles	90049	34.08	-118.48	5	94	ESI1				
9071	--	Circle Drive South & Westwood/LeConte	Westwood	90095	34.07	-118.45	6	?	ESI10				
none	808	Wilshire Blvd	Santa Monica	90401	34.02	-118.49	5	na	ESI2				
none	3601	W Olive Ave	Burbank	91505	34.15	-118.34	8	na	ESI3				
9074	10580	Wilshire Blvd	Los Angeles	90024	34.06	-118.43	27	?	ESI4				
9075	1250	4th St	Santa Monica	90401	34.02	-118.50	6	na	ESI5				Engelhardt/SAC 95-04
9076	13949	Ventura Blvd	Sherman Oaks	91423	34.15	-118.44	3	64	ESI7	38			
9077	--	UCLA Campus nr Westwood/ LeConte	Westwood	90095			6	?	ESI9				
9078	11000	Wilshire Blvd	Los Angeles	90024	34.06	-118.45	17	?	FE1				
9079	11755	Wilshire Blvd	Los Angeles	90025	34.05	-118.46	24	69	JAM7479	66?			
9080	16000	Ventura Blvd	Encino	91436	34.16	-118.48	12	81	JAM7487				
9081	16133	Ventura Blvd	Encino	91436	34.16	-118.48	13	73	JAM7488				
9082	16027	Ventura Blvd	Encino	91436	34.16	-118.48	6	33	JAM7489				
none	8949	Wilshire Blvd	Beverly Hills	90211	34.07	-118.39	7	na	JAM7586				
9084	3041	Cochran	Simi Valley	93065	34.28	-118.74	6	na	JAM7642				Kaufmann/NIST
9085	15451	San Fernando Mission Blvd	Mission Hills	91345	34.27	-118.47	4	101	JM1				
9086	15451	San Fernando Mission Blvd	Mission Hills	91345	34.27	-118.47	4	102	JM2				
9087	6200	Canoga Ave	Woodland Hills	91367	34.18	-118.60	4	221	KAR2				

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9088	15910	Ventura Blvd	Encino	91436	34.16	-118.48	16	21	KAR3				Kariotis/SAC 95-04
9089	--	--	Santa Clarita	91355			2	na	KPFF1A				
9090	--	--	Santa Clarita	91355			1	na	KPFF1B				
9091	--	--	Santa Clarita	91355			2	na	KPFF1C				
9092	7033	Owensmouth Ave	Canoga Park	91303	34.20	-118.60	3	179	L&M				
9093	18000+/-	Plummer & Etiwanda	Northridge	91330	34.24	-118.53	4	?	LCIB				
9094	18111	Nordhoff	Northridge	91330	34.24	-118.53	2	?	LCICH				
9095	18111	Nordhoff	Northridge	91330	34.24	-118.53	4	?	LCIEA1				Krawinkler/SAC 95-04
9096	18111	Nordhoff	Northridge	91330	34.24	-118.53	2	?	LCIEA2				Krawinkler/SAC 95-04
9097	--	Plummer	Northridge	91330			3	?	LCIED				
9098	23920	Valencia	Valencia	91355	34.41	-118.56	3	na	MG1				Green/SAC 95-07
none							3		MNH02				
9100	12233	Olympic Blvd	Los Angeles	90064	34.03	-118.46	3	82.1	MNH03AB				
9101	12233	Olympic Blvd	Los Angeles	90064	34.03	-118.46	3	82.2		MNH03CDE			
9102	12233	Olympic Blvd	Los Angeles	90064	34.03	-118.46	3	82.3	MNH03F				
9103	12233	Olympic Blvd	Los Angeles	90064	34.03	-118.46	3	82.4	MNH03G				
9104	12233	Olympic Blvd	Los Angeles	90064	34.03	-118.46	3	82.5	MNH03H				
9105	11845	Olympic Blvd	Los Angeles	90064	34.03	-118.45	13	8?	NYA501				
9106	21900	Burbank Blvd	Woodland Hills	91367	34.17	-118.61	3	45	NYA539				
9107	21555	Oxnard St	Woodland Hills	91367	34.18	-118.60	13	147	NYA544	65?		147	Uang/SAC 95-04
9108	10100	Santa Monica Blvd	Los Angeles	90067	34.06	-118.42	28	?	NYA591				

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9109	1888	Century Park East	Los Angeles	90067	34.06	-118.41	20	?	NYA592				
9110	2701	Ocean Park Blvd	Santa Monica	90405	34.02	-118.46	2	na	NYA629				
9111	501	Colorado	Santa Monica	90401	34.01	-118.49	3	na	NYA630				
9112	611	N Brand	Glendale	91203	34.16	-118.26	14	na	NYA631				
none	303	Glenoaks	Burbank		34.18	-118.31	10	na	NYA653				
9114	--	American & Valencia	Santa Clarita		34.41	-118.55	2	na	RCRJ				Anderson/SAC 95-07
9115	3903	W Olive Ave	Burbank	91505	34.15	-118.34	6	na	SGH1				
9116	--	Fairfax & 3rd	Los Angeles	90036	34.07	-118.36	4		SOM1				
9117	111	N Hollywood Way	Burbank	91505	34.15	-118.34	4	na	WEA				
9118	--	Valley & Soto	Boyle Heights		34.06	-118.20	5		WHLHSC				
9119	--	Valley & Soto	Boyle Heights		34.06	-118.20	4		WHLHSE				
9120	--	--	Glendale				20	na	WHLOF				
9121	6320	Canoga Ave	Woodland Hills	91367	34.19	-118.60	18	?	WJE1				Paret/SAC 95-04
9122	6320	Canoga Ave	Woodland Hills	91367	34.19	-118.60	18	?	WJE2				Paret/SAC 95-04
9123	11111	Santa Monica Blvd			34.05	-118.44	21	57		2			
9124	4605	Lankershim Blvd			34.15	-118.37	8	193		3			
9125	6345	Balboa Blvd		91316	34.19	-118.50	3	56		6			
9126					34.18	-118.59	1	99?		7			
9127					34.19	-118.46	5	51?2 17?		8			
9128					34.17	-118.59	5			9			
9129	6345	Balboa Blvd		91316	34.19	-118.50	3	56		10			

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9130	21011	Warner Center Ln	Woodland Hills	91367	34.18	-118.59	1	110		11		110	
9131	14130	Riverside Dr			34.16	-118.44	3	189		12			
9132					34.20	-118.63	6			13			
9133	20950	Warner Center Ln			34.18	-118.59	1	98		14			
9134					34.17	-118.59	1			15			
9135	4640	Lankershim Blvd			34.15	-118.37	6	22		16			
9136					34.24	-118.57	6			17			
9137	21530	Oxnard St			34.18	-118.60	1	1		18			
9138	12001	Ventura Place			34.14	-118.39	6	186		20			
9139	11175	Santa Monica Blvd			34.05	-118.45	9	41		21			
9140	1964	Westwood Blvd			34.05	-118.43	4	83		23			
9141					34.17	-118.59	3	48?		24			
9142					34.20	-118.50	3			25			
9143					34.15	-118.44	3			26			
9144					34.06	-118.45	17			27			
none								126?		29			
9146	1940	Bundy Dr	Los Angeles	90025	34.03	-118.46	9	168		30			
9147					34.16	-118.41	5			32			
9148	15303	Ventura Blvd	Sherman Oaks	91403	34.15	-118.47	15	42		33		42	Naeim/CSMIP
9149					34.07	-118.47	1	163?		34			
9152	1663	Sawtelle			34.05	-118.45	3	194		37			
9153					34.20	-118.50	3			39			

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9154	11150	Santa Monica Blvd			34.05	-118.45	16	3		41			
9155	15319	Chatsworth St			34.26	-118.57	3	72		42			
9156					34.17	-118.61	22			44			
9157	11040	Santa Monica Blvd			34.05	-118.44	3	203		45			
9158	10605	Balboa Blvd	Granada Hills		34.26	-118.50	3	190		46			
9159	6345	Balboa Blvd		91316	34.19	-118.50	3	56		47			
9160	21150	Dumetz Rd			34.16	-118.59	2	88		50			
9161	14500	Roscoe Blvd			34.22	-118.47	6	62		51			
9162	11911	San Vicente Blvd			34.05	-118.47	3	145		52			
9163					34.18	-118.59	1	26?109?		53			
9164	22120	Clarendon St	Woodland Hills	91367	34.17	-118.61	3	107		54		107	
9165	500	S Sepulveda Blvd			34.07	-118.46	6	150		55			
9166	21820	Burbank Blvd		91367	34.17	-118.60	3	9		57			
9167					34.17	-118.58	3			58			
9168	6325	Topanga Cyn Blvd			34.19	-118.61	5	114		59			
9169					34.17	-118.58	3			61			
9170	22020	Clarendon St			34.17	-118.61	3	104		62			
9171	4050	Lankershim Blvd			34.14	-118.36	3	161		63			
9172	7855	Haskell Ave	Los Angeles	91406	34.21	-118.48	3	212		64		212	
none					34.18	-118.60	12	147?		65			
none					34.05	-118.46	25	69?		66			

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9175	11925	Wilshire Blvd			34.05	-118.46	3	132		67			
9176					34.20	-118.44	3			70			
9177	15490	Ventura Blvd			34.15	-118.47	3	160		71			
9178	20951	Burbank Blvd		91367	34.17	-118.59	1	95		72			
9179					34.26	-118.50	3			73			
none					34.17	-118.56	4	171?		74			
none					34.27	-118.44	4			75			
9182	12345	Ventura Blvd			34.14	-118.40	2	174		76			
9183	6355	Topanga Cyn Blvd			34.19	-118.47	5	119		77			
9184					34.18	-118.59	1	97?		79			
9185	6345	Balboa Blvd		91316	34.19	-118.50	3	56		80			
9186	21800	Burbank Blvd		91367	34.17	-118.60	3	146		81			
9187	18801	Ventura Blvd	Tarzana	91356	34.17	-118.54	3	60		82		60	
9188					34.17	-118.59	1			83			
9189	10780	Santa Monica Blvd			35.05	-118.43	4	23		85			
9190	1640	S Sepulveda Blvd			34.05	-118.44	5	61		86			
9191					34.06	-118.46	17			87			
9192	16861	Ventura Blvd			34.16	-118.50	3	219		89			
9193	3838	Lankershim Blvd			34.14	-118.36	24	166.2		90			
9194					34.05	-118.44	3	180?		91			
9195	11100	Santa Monica Blvd			34.05	-118.44	16	58		92			
none					34.27	-118.47	4			93			

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
9197	11766	Wilshire Blvd			34.05	-118.46	17	178		94			
9198	3838	Lankershim Blvd			34.14	-118.36	4	166.3		96			
9199	11500	Olympic Blvd			34.04	-118.44	6	125		97			
9200					34.18	-118.59	1	99?		98			
9201					34.05	-118.43	3	71?		99			
9202	5900	N Sepulveda Blvd	Van Nuys	91411	34.18	-118.47	5	142		100		142	
none	11550	Indian Hills Rd					3	2					
none	15503	Ventura Blvd					3	7					
none	11400	Olympic Blvd	Los Angeles	90064			16	12.1					
none	11400	Olympic Blvd	Los Angeles	90064			4	12.2					
none	18370	Burbank Blvd		91356			7	13					
none	5921	Owensmouth Ave					1	15					
none	6301	Owensmouth Ave	Woodland Hills	91367	34.18	-118.60	12	17				17	
none	6300	Canoga Ave					17	18					
none	10877	Wilshire Blvd					22	25					
none	20955	Warner Center Ln	Woodland Hills	91367	34.18	-118.59	1	26		53?		26	
none	10920	Wilshire Blvd					20	28					
none	11080	Olympic Blvd					4	29					
none	10960	Wilshire Blvd					24	32					
none	1460	Westwood Blvd					3	34					
none	10866	Wilshire Blvd					14	35					
none	7345	Medical Center Dr					6	40					

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
none	9055	Reseda Blvd					1	43					
none	1801	Century Park East					25	46					
none	10900	Wilshire Blvd	Los Angeles	90024	34.06	-118.44	16	49			61		
none	5000	Van Nuys Blvd					?	50					
none	13248	Roscoe Blvd					3	53					
none	13400	Riverside Dr					3	55					
none	6060	Sepulveda Blvd					3	66					
none	11999	San Vicente Blvd					4	68					
none	21054	Sherman Way					3	74					
none	16461	Sherman Way					3	75					
none	12400	Wilshire Blvd					15	76					
none	1849	Sawtelle					7	78					
none	20935	Warner Center Ln					1	85					
none	22144	Clarendon St					3	87					
none	10585	Santa Monica Blvd					3	92					
none	10635	Santa Monica Blvd					3	93					
none	20971	Burbank Blvd		91367			1	96					
none	20970	Warner Center Ln	Woodland Hills	91367	34.18	-118.59	1	97		79?		97	
none	20920	Warner Center Ln	Woodland Hills	91367	34.17	-118.59	1	99		7?98?		99	
none	20931	Burbank Blvd		91367			1	100					
none	640	S Sepulveda Blvd					3	103					

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
none	5805	Sepulveda Blvd					8	105					
none	6815	Noble Ave.	Van Nuys	91405	34.19	-118.46	5	108				108	
none	21031	Warner Center Ln	Woodland Hills	91367	34.18	-118.59	1	109		53?		109	
none	1440	S Sepulveda Blvd					3	111					
none	13425	Ventura Blvd					3	112					
none	11633	San Vicente Blvd					3	113					
none	13245	Riverside Dr	Sherman Oaks	91423	34.16	-118.42	6	115	BAK?				
none	12100	Wilshire Blvd					20	116					
none	11990	San Vicente Blvd					3	117					
none	3838	Lankershim Blvd					36	120					
none	10800	Wilshire Blvd					25	121					
none	14140	Ventura Blvd	Sherman Oaks	91423	34.15	-118.44	3	122				122	
none	10840	Wilshire Blvd					13	123					
none	16530	Ventura Blvd	Encino	91436	34.16	-118.49	6	127			46	127	
none	9375	San Fernando Rd					6	129					
none	20350	Ventura Blvd	Woodland Hills	91364	34.17	-118.58	2	130				130	
none	6400	Laurel Canyon Blvd	North Hollywood	91606	34.19	-118.40	6	131				131	
none	10990	Wilshire Blvd					18	133					
none	7301	Medical Center Dr	West Hills	91307	34.20	-118.63	5	134				134	
none	15760	Ventura Blvd	Sherman Oaks	91436	34.16	-118.48	20	135				135	Naeim/CSMIP?
none	16500	Ventura Blvd	Sherman Oaks	91436	34.16	-118.49	4	136				136	

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
none	4312	Woodman Ave					3	137					
none	16603	Ventura Blvd	Sherman Oaks	91436	34.16	-118.49	5	141				141	
none	11726	San Vicente Blvd					6	143					
none	10474	Santa Monica Blvd					3	144					
none	6350	Laurel Canyon Blvd	North Hollywood	91606	34.19	-118.40	4	148				148	
none	11846	Ventura Blvd					3	155					
none	11677	San Vicente Blvd					3	157					
none	12925	Riverside Dr	Sherman Oaks	91423	34.16	-118.41	4	158				158	
none	5311	Topanga Cyn Blvd.	Woodland Hills	91364	34.17	-118.61	1	164				164	
none	16221	Mulholland Dr					1	165					
none	1101	Gayley Ave					3	167					
none	14546	Hamlin St	Van Nuys	91411	34.19	-118.45	3	169				169	
none	15500	S Stephen Wise Dr					2	170					
none	11601	Wilshire Blvd					24	175					
none	10936	Wilshire Blvd					22	176					
none	16600	Sherman Way	Van Nuys	91406	34.20	-118.49	2	177				177	
none	21731	Ventura Blvd					3	181					
none	13412	Ventura Blvd	Sherman Oaks	91423	34.15	-118.42	3	183		56	60		
none	22141	Ventura Blvd					3	184					
none	21300	Victory Blvd					12	185					
none	10515	Balboa Blvd	Granada Hills				3	187					
none	16542	Ventura Blvd	Encino	91436	34.16	-118.49	5	188				188	

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
none	3575	Cahuenga Blvd		90068			6	191					
none	12626	Riverside Dr					5	192					
none	6445	Sepulveda Blvd					3	195					
none	3330	Cahuenga Blvd		90068			5	196					
none	18321	Ventura Blvd					9	197					
none	11570	Indian Hills Rd					1	199					
none	17404	Ventura Blvd					3	200					
none	1828	Sawtelle					3	201					
none	15301	Ventura Blvd					3	209					
none	7230	Medical Center Dr	Canoga Park	91307	34.20	-118.63	6	211				211	
none	1990	Westwood Blvd					3	213					
none	22110	Roscoe Blvd					3	214					
none	16501	Ventura Blvd					6	215					
none	20750	Ventura Blvd					4	222					
none	17547	Ventura Blvd					3	223					
none	22025	Ventura Blvd					3	226					
none	15650	Devonshire St					3	227					
none	10800	W Pico Blvd					4	229					
none	11859	Wilshire Blvd					6	230					
none	15531	San Fernando Mission Blvd					3	231					
none	22801	Ventura Blvd	Woodland Hills	91364	34.17	-118.62	3	233				blank	
none	4705	Laurel Canyon Blvd					5	234					
none	1100	Glendon Ave					20	242					
none			Encino				20	135?					Naeim/CSMIP

Table A-1 Master List of Northridge WSMF Databases Sorted by SAC Building ID and L.A. Building ID (continued)

Bldg ID	Number	Street	City	Zip	Lat.	Long.	Sty	LA	Bonowitz	Durkin	Dames	Paret	Case Study
none			North Hollywood				8						Naeim/CSMIP
none			Tarzana				10						Naeim/CSMIP

APPENDIX B. NORTHRIDGE EARTHQUAKE WSMF BUILDING DAMAGE

Bruce F. Maison

B.1 Introduction

This appendix presents the results from a database of WSMF buildings in the immediate region affected by the 1994 Northridge earthquake. The purpose is to summarize the actual past performance of WSMF buildings based on interpretation of actual damage inspection reports. The building database is described and several statistics summarized.

B.2 Connection Component Damage

Typical components of pre-Northridge moment connections are depicted in Figure B-1. Building surveys revealed different numbers of damage incidents associated with each component, and Figure B-2 illustrates the distribution found in 29 Northridge damaged buildings (Dames and Moore, 1998). A connection can suffer multiple incidents so that the number of damage incidents is greater than the number of damaged connections. Damage to girder groove welds and column flanges are the predominant damaged components.

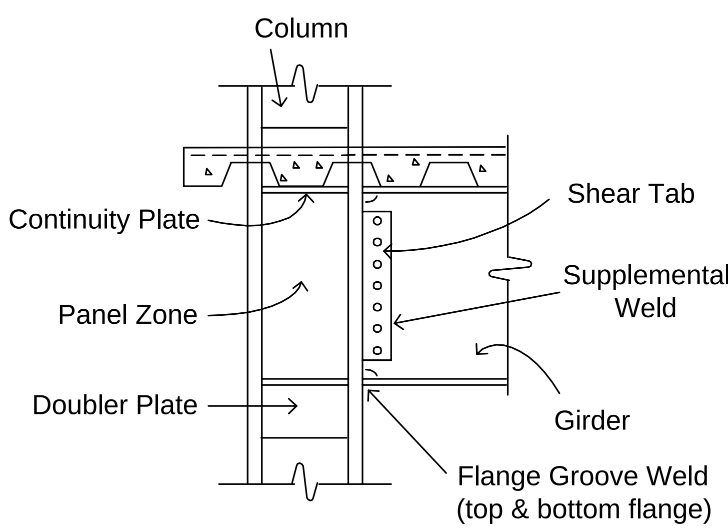


Figure B-1 Typical Components of Pre-Northridge Moment Connections

FEMA 267 (1995) further differentiates damage types into specific categories and assigns damage indices to each: groove weld (W1 to W5), column (C1 to C7), girder (G1 to G8), panel zone (P1 to P9), and shear tab (S1 to S6). The indices (Table 4-3a in FEMA 267) represent SAC judgmental estimates of the relative damage severity, and the most severe have indices of 8 or greater, where 10 represents a total loss. These typically involve fracture initiating at the flange groove weld root with the crack propagating into the weld or column flange. The most frequent types are fracture through the weld metal thickness (W2) and full or partial flange crack in the column HAZ (C4).

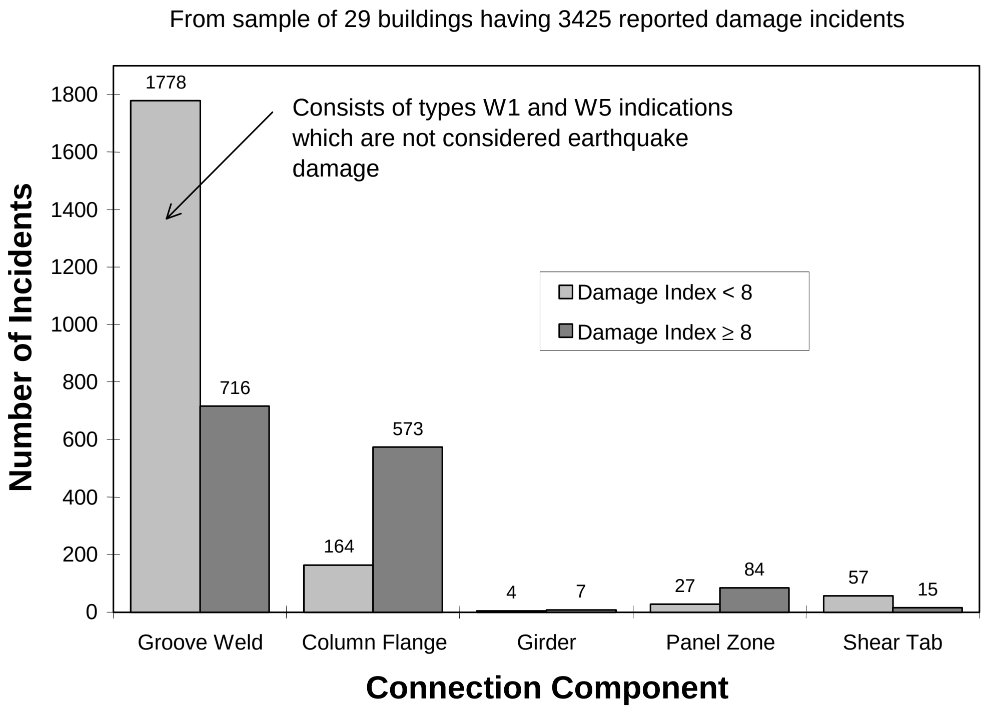


Figure B-2 Distribution of Component Damage

Note that fully one-half of the total (1778 out of 3425 reported incidents) are groove weld types having damage indices < 8. These are weld root (W1) indications and non-rejectable UT detectable indications (W5) which were classified as possible damage by FEMA 267, but are now considered by SAC as pre-earthquake existing defects (SAC 90% Draft, 1999, Paret and Attalla, 1998). This reclassification sharply reduces the amount of connection damage attributed to the Northridge earthquake.

This appendix uses the latter SAC interpretation, and does not count W1 and W5 incidents as damage. A connection is considered damaged if it has damage at the bottom flange, top flange or both locations. No differentiation is made here between the severity and numbers of the differing damage types affecting a connection. The SAC definition of a connection is also used, i.e., one connection is the attachment of one girder to one column.

B.3 Sources of Building Survey Data

Prior SAC studies involved the compilation of results from postearthquake building damage surveys, and these data were used here. This data set represents virtually all known buildings with available damage survey results in the area affected by the Northridge earthquake. The three main sources are as follows.

SAC Phase 2 Subtask 3.1.1

This effort was the collection of data on buildings inspected under the Los Angeles Inspection Ordinance (Michael F. Durkin and Associates, 1999). Following the 1994 Northridge earthquake and the finding of fractured WSMF connections, the City of Los Angeles passed Ordinance 170406 on February 22, 1995 requiring inspection and repair of certain buildings (those within particular geographical area affected by Northridge with exemptions given to residential buildings). The building owners were required to submit the inspection report to the City and these were the subject of the Subtask.

Data on 242 buildings were obtained directly from the submittals, and more detailed information on a sample of 100 buildings was obtained via contact with building owners. In general, the data compilation from City records did not specifically identify how may connections had W1 type damage and repairs. Since these are now not considered as damage, many of the buildings could not be used. The 100 building sample had segregation of damage into non-W1 and W1 damage types by use of SAC Phase 2 Subtask 3.1.3 results (Wiss, Janney, Elstner Associates, 1999). These 100 buildings were used here. The data was on a building-wide basis having no information on particular damage types or the amount of damage in each building principal direction. The coordinates were slightly altered to maintain anonymity of the building.

SAC Phase 2 Subtask 3.1.2.

This was the detailed data collection for selected WSMF buildings (Dames and Moore, 1998). Data on 49 buildings was compiled from project work performed by six engineering firms who agreed to participate on the Subtask. A subset of 29 buildings had very detailed damage information including the specific FEMA 267 damage types and locations within the building frame.

SAC Phase 1 Task 2.

This task was the initial SAC data collection, assessment and interpretation of damage caused by the Northridge event (SAC 95-06, 1995). Data on 89 buildings were compiled from canvassing local engineering firms and testing labs. The data compilation was fairly detailed but in a format that necessitated some review of the original survey reports to provide information suitable for use here.

B.4 Seismic Demands

Seismic demands at each building site were estimated by assigning actual recordings of the Northridge earthquake from a nearby recording station (Somerville, 1999). The vast majority of buildings had recording instruments within 3 km. The seismic demand quantities were: Modified Mercalli Intensity, peak ground acceleration, peak ground velocity, spectral acceleration (5% damping) at the building fundamental, 0.03 sec, 1 sec, 2 sec, and 3 sec periods. The spectral acceleration at the building fundamental period was computed by interpolation of the spectral values using an estimated building period. Due to earthquake directivity effects, the demands varied in the NS and EW directions, and the geometric mean (square root of the product) of the two directions was used to characterize the demand at the building site.

B.5 Building Database

The three sources of building survey data discussed above were compiled into a database. Each source had different formats, level of detail, data deficiencies, and errors. Hence, the compilation took considerable effort including: elimination of duplicate buildings, resolution of conflicting data from different sources, collection of missing and updated data, determination of longitude and latitude coordinates, removal of W1 and W5 types from damage counts, and data entry/formatting into a single product.

Judgment was used to resolve discrepancies and augment missing data where reasonable. Some buildings did not have sufficient or appropriate data for use here, and were simply excluded. Note that under this effort, only a few buildings had their data checked against the original inspection reports prepared by the responsible engineer, and hence the quality of data is largely dependent on that provided by the original sources. It is likely that errors exist in the database presented here but it is believed that any errors are of such a nature as not to affect overall trends/conclusions based on database analysis.

Definitions of field data contained in the database are listed in Table B-1. Tables B-2 and B-3 are large tables, and are moved to the end of this appendix. They show a portion of the building database, sorted by different fields.

Table B-1 Definitions Used in Northridge Database (Partial List)

Column Label	Description
ID	Building identifier number.
Lat	Latitude.
Long	Longitude.
Conn	Number of moment connections in building.
Insp	Number of moment connections inspected.
Bot	Number of connections having damage at bottom flange only.
Top	Number of connections having damage at top flange only.
B&T	Number of connections having damage at both top and bottom flange.
Shr	Number of connections having damage to shear connectors.
PZ	Number of connections having damage to column panel zone.
Total	Total number of damaged connections, Tdam = Bdmg + Tdmg + B&T. Note that some building surveys reported total number of damaged connections only, and for these cases, Bdmg, Tdmg, B&T are listed as 0.
Area	Building area (sf)
Sty	Number of stories in building.
MMI	Modified Mercalli Intensity per Wald et al. (1999)
Pga	Peak ground acceleration (g).
Pgv	Peak ground velocity (in/sec).

B.6 Summary Statistics

The database contained 185 WSMF buildings, but 18 were screened out because of either low connection inspection rates or site locations outside the vicinity of the sample region.

Buildings having low inspection rates were excluded because the resulting connection damage rates were considered not representative for the entire building. The inspection rate cut-off was taken as 5%. Several isolated buildings were located some distance away from the others (e.g., Ventura County, downtown L.A.) and these were excluded in order to make the remaining sample more representative of the building population in the region most affected by Northridge. About two-thirds of the screened database were L.A. Ordinance building.

Figure B-3 shows the spatial distribution of the screened 167 buildings (spectral acceleration contours taken from Northridge, 1996). The geographic area extends from Santa Clarita in the north to Santa Monica in the south, and from Woodland Hills in the west to Burbank in the east. The area is roughly 600 sq. miles. The building distribution generally corresponds to the built environment pursuant to population density. Most buildings were located south of the area that experienced the most intense ground shaking. Clusters of buildings are found in several specific areas: Woodland Hills/Canoga Park (southwest of epicenter), Santa Monica/West L.A. (south of epicenter), and Sherman Oaks/Burbank (Ventura Blvd corridor east of highway I-405). Damaged buildings (those having at least one damaged connection) were present throughout the region represented by the database sample (Figure B-4).

The distribution of building heights and areas are shown in Figures B-5 and B-6. The median height and area are respectively 4 stories and 70,000 sf. These are likely to be representative of the buildings covered by the sample region, but the degree to which they match the true population was not studied. Buildings having six stories or less constitute about 80% of the total area (Figure B-7).

Figure B-8 shows that most buildings experienced peak ground accelerations over a fairly limited range from 0.3g to 0.4g. The median PGA was 0.36g. Such PGAs were not extraordinary versus those implicit in building design codes (e.g., 1994 *UBC* seismic zone 4 has Z = 0.4 which implies 0.4g PGA shaking).

Figure B-9 shows the distribution of connection inspection rates. Inspection rate is defined as the number of inspected connections divided by the total number of moment connections in the building, expressed as a percentage. The rates have a bimodal distribution reflecting the idea that during the building survey process, once a percentage of connections are inspected and no damage found, then the survey is terminated. Finding damage triggers more complete inspections. Hence, the distribution has peaks at both lower and higher rates. Damage types W1 were considered as damage when many of the inspections were performed on these buildings.

Figure B-10 shows the distribution of connection damage rates. Damage rate is defined as the number of damaged connections discovered divided by the number of moment connections in the inspection sample, expressed as a percentage. A striking observation is that only about one-half (53%) of the buildings suffered damage, and of these only about 1 in 3 (27%) had rates greater than 20%. The median and 90th percentile rates are 1.7% and 31%, respectively. This suggests that initial post-Northridge impressions about the amount and severity of building damage were overstated in large part due to consideration of types W1 as damage. The damage picture changes significantly when these are excluded. Paret and Attalla (1998) have previously noted this as well.

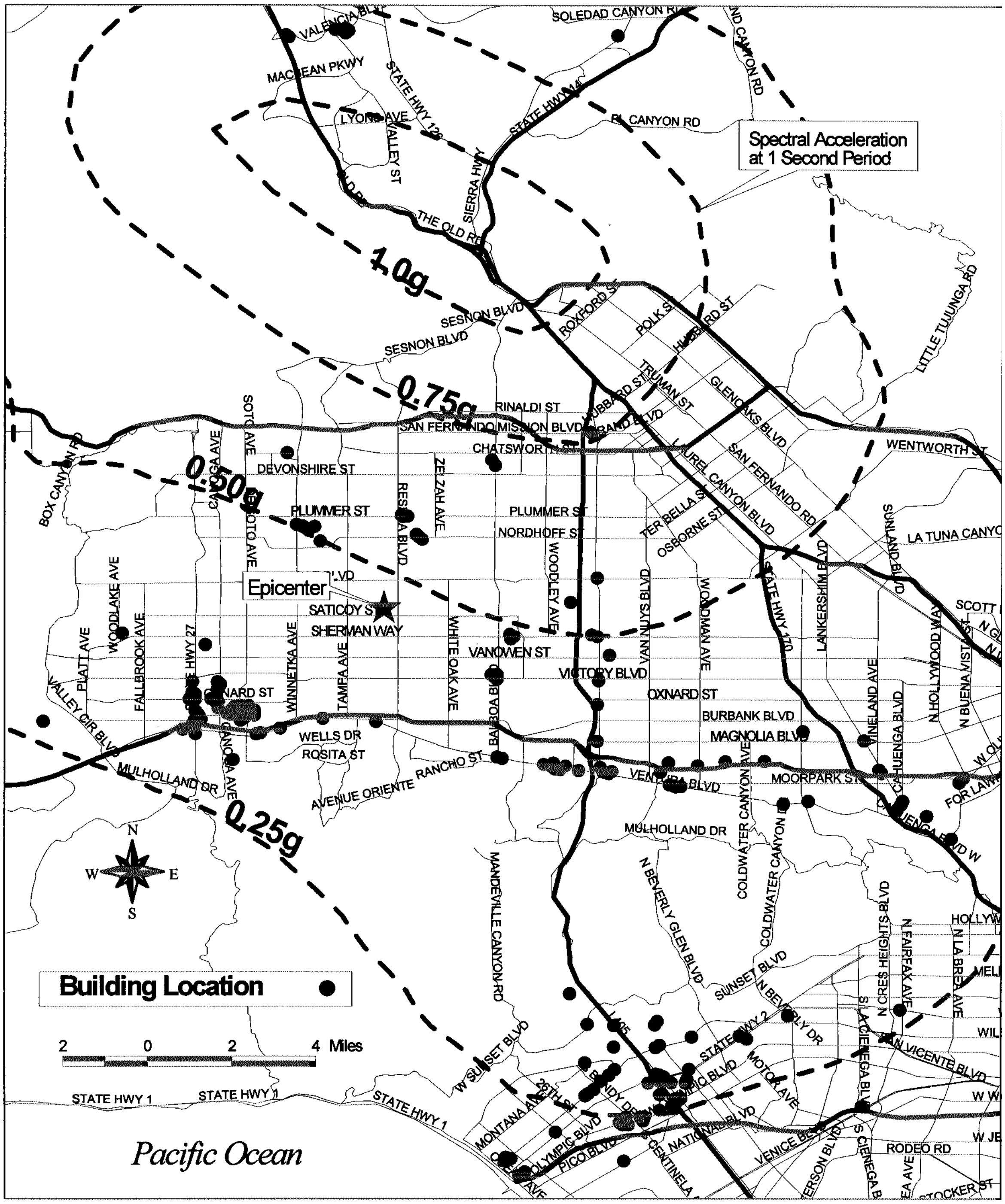


Figure B-3 Spatial Distribution of Screened Buildings

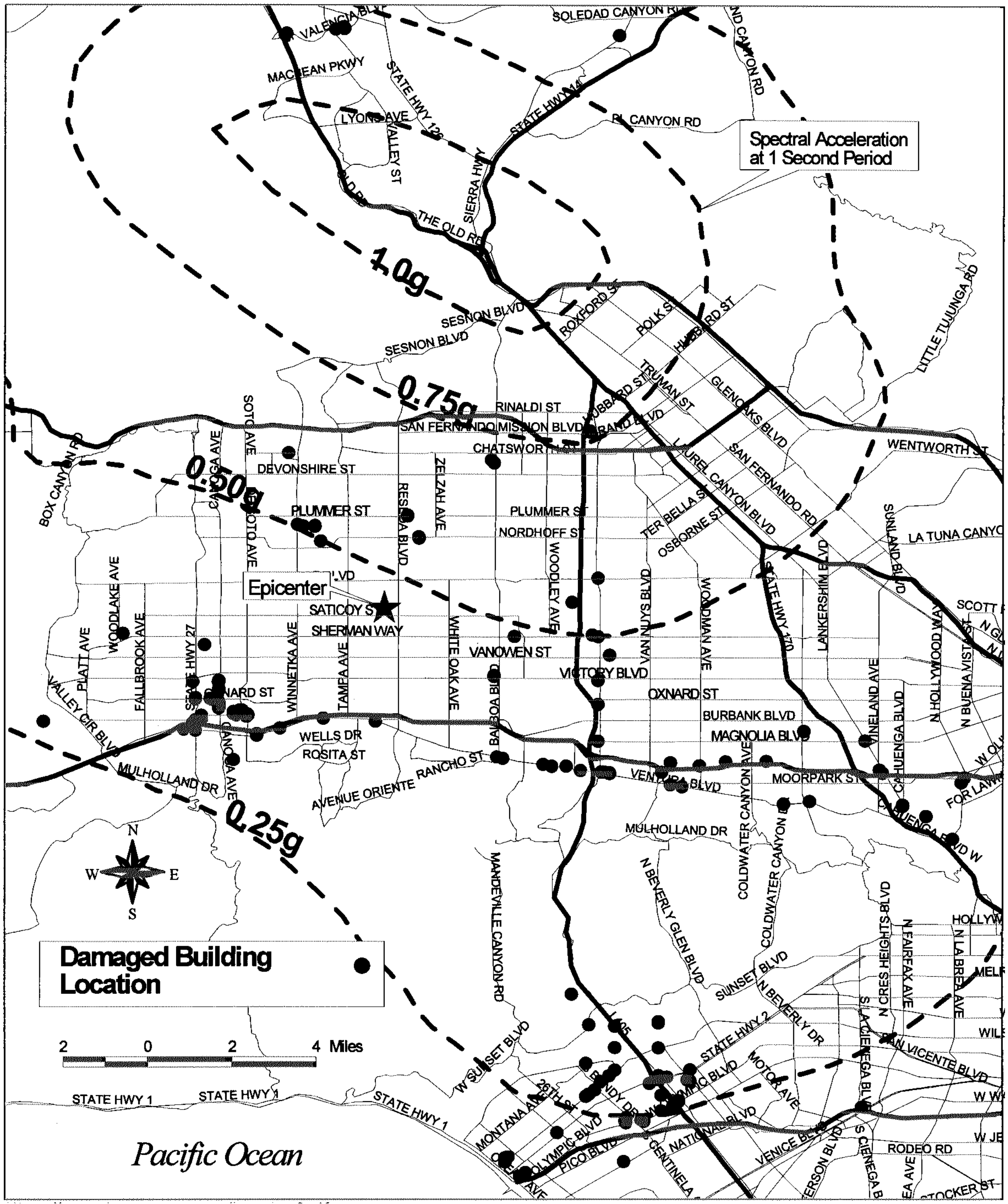


Figure B-4 Spatial Distribution of Damaged Buildings

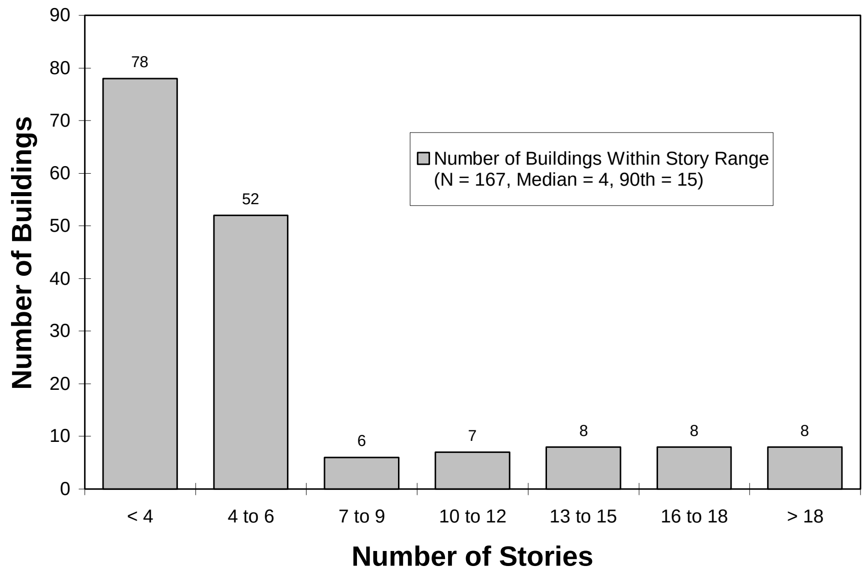


Figure B-5 Distribution of Building Heights

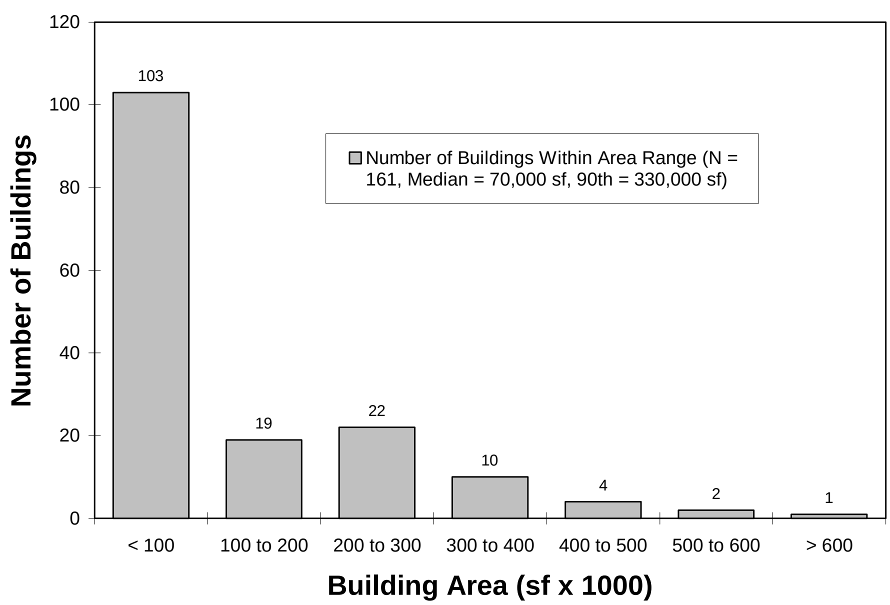


Figure B-6 Distribution of Building Areas

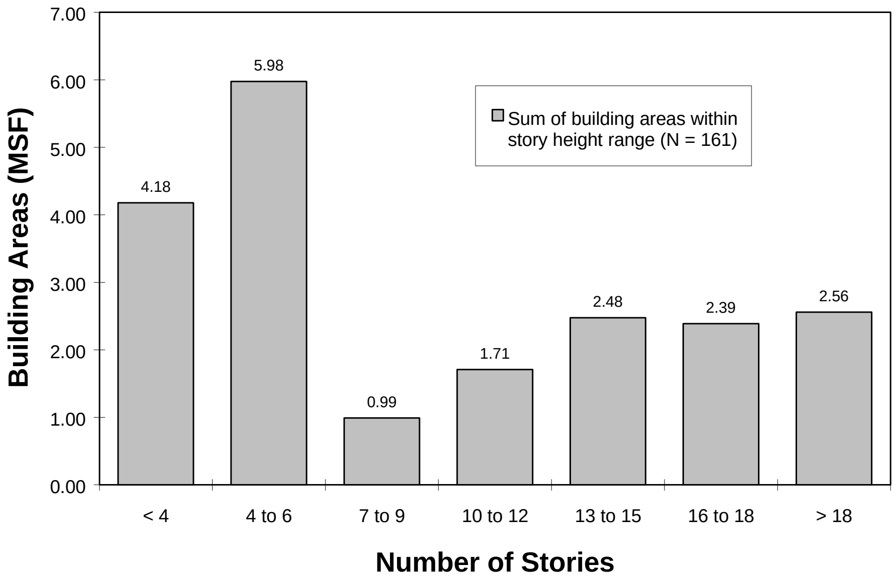


Figure B-7 Distribution of Total Areas

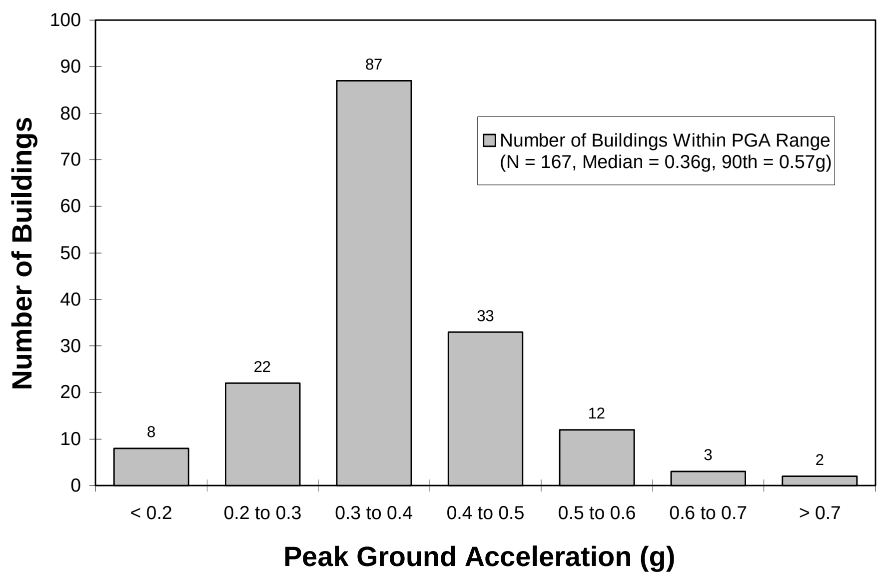


Figure B-8 Distribution of Peak Ground Accelerations

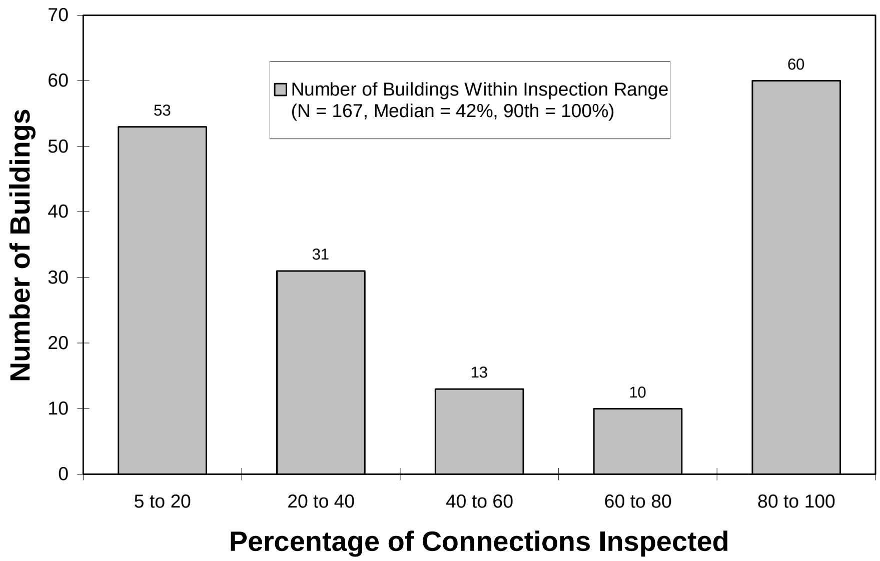


Figure B-9 Distribution of Connection Inspection Rates

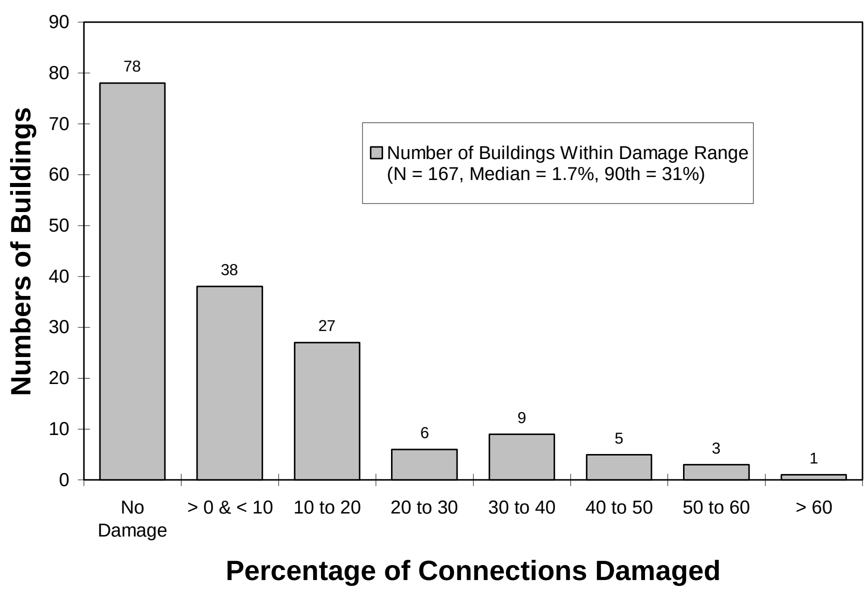


Figure B-10 Distribution of Connection Damage Rates

Figure B-11 shows a plot of damage rates versus PGA. There is large scatter but a trend of higher damage rates with increasing PGA is apparent. By inspection, at about 0.6g the median rate is about 50% whereas below about 0.45g there are so many buildings with no damage (cluster of points about 0% damage rate) that it causes the median rate to be very small in this range. In any event, the correlation between damage rate and PGA is weak. Further statistical analysis of the database was performed to create a methodology for rapid loss estimation. The results of this work can be found elsewhere (Maison and Bonowitz, 1999).

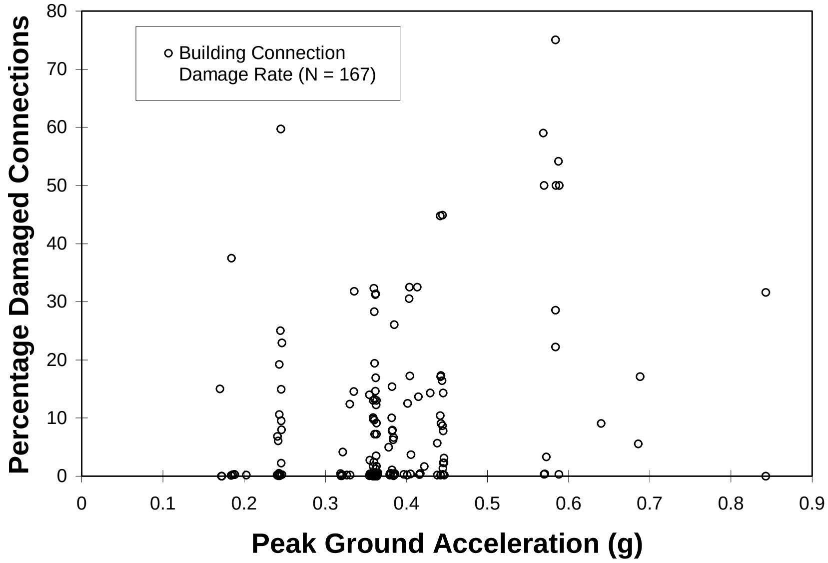


Figure B-11 Damage Rates Versus PGA

B.7 References

Dames and Moore, 1998, *Survey of Damaged Steel Moment Frame Buildings*, report for SAC Phase 2, Task 3.1.2, version 1.00.

FEMA 267, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel moment Frame Structures,* Federal Emergency Management Agency, Washington, D.C.

Goel, R.K., and Chopra, A.K., 1997, *Vibration Properties of Buildings Determined From Recorded Earthquake Motions*, Univ. of Calif. Earthquake Eng. Research Center Report No. UCB/EERC-97/14.

Maison, B.F., and Bonowitz, D., 1999, *Rapid Loss Estimation Methodology For Steel Moment Frame Buildings*, SAC Task 3.2.1, Report to SAC.

Michael F. Durkin and Associates, 1999, *Collection of Data on Buildings Inspected Under the Los Angeles Inspection Ordinance*, database for SAC Task 3.1.1.

Northridge Earthquake Reconnaissance Report, 1996, *Earthquake Spectra*, Supplement to Vol. II.

Paret, T.F., and Attalla, M.R., 1998, “Changing Perceptions of the Extent of Damage to Welded Steel Moment Frames in the Northridge Earthquake,” SEAOC 1998 Convention.

SAC 95-06, 1995, Survey and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994, SAC Joint Venture Technical Report SAC 95-06.

SAC 90%, 1999, Seismic Evaluation & Upgrade Criteria for Existing Welded Steel Moment-Resisting Frame Structures, Report No. SAC-2000-02, working draft at 90% level.

Somerville, P., 1999, Ground Motion Estimates for SAC Loss Estimation Task 3.2, report to SAC.

Wald, D.J., Quitoriano, V., Heaton, and T.H., Kanamori, H., 1999, “Relationships between Peak Ground Acceleration, Peak Ground Velocity, and Modified Mercalli Intensity in California”, *Earthquake Spectra*, Vol. 15, No. 3.

Wiss, Janney, Elstner Associates, Inc., 1999, Evaluation of Inspection Reliability, Clarification of the Origins of W1a and W1b and Distribution of W1 and non-W1 Conditions, report for SAC Task 3.1.3.

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9090	34.412	-118.554	1	7500	30	18	1	1	2	4	0.22	0	0	9.0	0.586	33.1
9149	34.066	-118.469	1	15000	12	8	0	0	0	0	0.00	0	0	7.6	0.202	10.4
9126	34.175	-118.589	1	16300	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9184	34.175	-118.589	1	17700	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9188	34.174	-118.591	1	19130	8	6	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9137	34.179	-118.599	1	20000	12	12	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9134	34.174	-118.591	1	20710	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9200	34.175	-118.589	1	21500	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9130	34.175	-118.592	1	22400	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9133	34.175	-118.589	1	23400	12	9	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9163	34.175	-118.590	1	23500	8	6	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9178	34.174	-118.587	1	24670	10	10	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9069	34.410	-118.574	1	27000	20	20	10	0	0	10	0.50	0	9	9.1	0.586	33.1
9182	34.143	-118.402	2	11000	10	5	0	0	0	0	0.00	0	0	7.8	0.244	14.5
9114	34.412	-118.554	2	15400	16	16	8	0	0	8	0.50	0	4	9.0	0.586	33.1
9094	34.236	-118.528	2	17930	72	27	1	0	0	1	0.04	0	0	8.9	0.403	18.3
9089	34.412	-118.554	2	19400	40	14	4	0	0	4	0.29	0	0	9.0	0.586	33.1
9051	34.286	-118.744	2	22600	16	6	0	0	0	0	0.00	0	0	7.7	0.328	12.9
9096	34.236	-118.528	2	31800	68	68	0	0	0	0	0.00	0	0	8.9	0.403	18.3
9031	34.239	-118.568	2	34600	50	50	0	0	0	0	0.00	0	0	8.9	0.403	18.3
9049	34.410	-118.459	2	36000	68	59	1	0	0	1	0.02	0	0	8.6	0.425	16.7
9160	34.158	-118.592	2	48000	5	5	0	0	0	0	0.00	0	0	7.9	0.362	19.1
9006	34.201	-118.495	2	49000	64	64	4	0	1	5	0.08	0	0	8.2	0.384	15.2
9091	34.412	-118.554	2	57600	480	77	0	0	0	0	0.00	0	0	9.0	0.586	33.1
9007	34.175	-118.590	2	62800	100	97	7	0	0	7	0.07	0	5	8.3	0.362	19.1
9026	34.239	-118.564	2	98444	84	82	22	0	3	25	0.30	2	5	8.9	0.403	18.3
9110	34.019	-118.457	2	112000	172	15	0	0	0	0	0.00	0	0	8.0	0.395	11.7
9152	34.046	-118.448	3	nr	28	13	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9167	34.167	-118.584	3	nr	76	8	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9201	34.047	-118.434	3	nr	96	23	0	0	0	0	0.00	0	0	8.0	0.444	19.4
9131	34.157	-118.441	3	nr	480	33	0	0	0	0	0.00	0	0	8.1	0.244	14.5
9092	34.198	-118.602	3	11100	24	12	0	0	0	0	0.00	0	0	8.5	0.353	16.7
9103	34.032	-118.456	3	13500	72	32	1	0	0	1	0.03	0	0	8.1	0.444	19.4

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9143	34.149	-118.437	3	16000	34	20	0	0	0	5	0.25	0	0	8.2	0.244	14.5
9102	34.032	-118.456	3	16800	102	44	4	0	0	4	0.09	0	0	8.1	0.444	19.4
9040	34.234	-118.562	3	17700	46	9	0	0	0	0	0.00	0	0	8.9	0.384	15.2
9194	34.048	-118.442	3	20000	36	5	0	0	0	0	0.00	0	0	8.1	0.357	10.8
9169	34.169	-118.576	3	20400	54	10	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9104	34.032	-118.456	3	21000	80	32	0	0	0	0	0.00	0	0	8.1	0.444	19.4
9157	34.048	-118.443	3	22000	24	8	0	0	0	0	0.00	0	0	8.1	0.357	10.8
9155	34.265	-118.573	3	22000	114	17	0	0	0	0	0.00	0	0	9.0	0.384	15.2
9111	34.015	-118.491	3	23100	48	6	0	0	0	0	0.00	0	0	7.9	0.571	12.7
9039	34.173	-118.561	3	25000	22	7	0	0	0	0	0.00	0	0	8.2	0.417	12.9
9175	34.047	-118.465	3	25000	46	10	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9170	34.170	-118.606	3	25600	62	16	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9164	34.170	-118.606	3	30000	50	13	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9100	34.032	-118.456	3	33600	152	76	11	0	2	13	0.17	0	0	8.1	0.444	19.4
9192	34.159	-118.501	3	35000	54	16	0	0	0	0	0.00	0	0	7.4	0.328	12.9
9153	34.201	-118.495	3	40000	64	64	0	0	0	4	0.06	0	0	8.2	0.384	15.2
9179	34.261	-118.502	3	42000	90	19	0	0	0	6	0.32	0	0	9.4	0.841	27.6
9177	34.155	-118.472	3	43000	96	20	0	0	0	1	0.05	0	0	8.0	0.381	16.5
9076	34.149	-118.437	3	45000	36	26	5	0	0	5	0.19	2	0	8.2	0.244	14.5
9097	34.243	-118.532	3	45900	172	168	26	0	3	29	0.17	0	3	9.1	0.403	18.3
9056	34.174	-118.587	3	48000	84	16	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9057	34.174	-118.587	3	48000	84	79	1	0	0	1	0.01	0	0	8.3	0.362	19.1
9129	34.187	-118.502	3	50000	57	15	0	0	0	1	0.07	0	0	8.1	0.384	15.2
9159	34.187	-118.502	3	50000	57	13	0	0	0	2	0.15	0	0	8.1	0.384	15.2
9125	34.187	-118.502	3	50000	69	20	0	0	0	2	0.10	0	0	8.1	0.384	15.2
9185	34.187	-118.502	3	50000	75	14	0	0	0	0	0.00	0	0	8.1	0.384	15.2
9022	34.042	-118.469	3	50240	100	100	13	0	1	14	0.14	0	0	8.2	0.357	10.8
9101	34.032	-118.456	3	51000	312	154	12	0	0	12	0.08	0	0	8.1	0.444	19.4
9004	34.150	-118.441	3	52000	114	114	17	0	0	17	0.15	0	9	8.2	0.244	14.5
9187	34.171	-118.543	3	53000	57	9	0	0	0	0	0.00	0	0	8.1	0.418	13.0
9047	34.042	-118.444	3	54000	146	38	16	0	1	17	0.45	0	0	8.2	0.444	19.4
9005	34.171	-118.657	3	59700	36	36	7	0	0	7	0.19	0	0	8.3	0.362	19.1

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9186	34.173	-118.603	3	60000	80	14	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9063	34.171	-118.606	3	63000	68	68	9	0	0	9	0.13	0	0	8.2	0.362	19.1
9158	34.262	-118.503	3	67000	28	9	0	0	0	0	0.00	0	0	9.4	0.841	27.6
9172	34.213	-118.475	3	68000	162	16	0	0	0	0	0.00	0	0	8.1	0.439	13.7
9142	34.201	-118.495	3	70000	64	11	0	0	0	0	0.00	0	0	8.2	0.384	15.2
9141	34.174	-118.587	3	75000	114	22	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9098	34.412	-118.557	3	81600	nr	48	0	0	0	0	0.00	0	0	9.0	0.586	33.1
9018	34.167	-118.584	3	82000	108	65	11	0	0	11	0.17	0	6	8.1	0.362	19.1
9106	34.172	-118.605	3	84000	132	33	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9162	34.053	-118.470	3	100000	28	4	0	0	0	0	0.00	0	0	8.1	0.243	9.7
9061	34.069	-118.400	3	108000	202	24	5	1	0	6	0.25	0	0	7.5	0.238	9.1
9166	34.172	-118.604	3	120000	140	124	0	0	0	3	0.02	0	0	8.2	0.362	19.1
9025	34.172	-118.603	3	120000	216	106	30	0	0	30	0.28	0	6	8.2	0.362	19.1
9054	34.139	-118.353	3	261000	135	120	33	2	10	45	0.38	6	8	7.2	0.187	8.0
9171	34.143	-118.361	3	865000	220	39	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9198	34.142	-118.361	4	nr	240	21	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9013	34.131	-118.344	4	26200	136	133	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9095	34.236	-118.528	4	27880	96	96	11	0	1	12	0.13	3	0	8.9	0.403	18.3
9119	34.064	-118.197	4	36800	240	30	0	0	0	0	0.00	0	0	7.1	0.336	6.9
9055	34.156	-118.431	4	42400	74	73	5	0	0	5	0.07	0	0	8.1	0.244	14.5
9032	34.177	-118.597	4	46375	112	112	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9065	34.020	-118.497	4	52000	78	49	0	0	0	0	0.00	0	0	7.9	0.571	12.7
9053	34.029	-118.480	4	54200	132	110	9	1	0	10	0.09	5	0	8.1	0.642	21.1
9140	34.047	-118.435	4	58000	298	43	0	0	0	1	0.02	0	0	8.0	0.444	19.4
9021	34.153	-118.462	4	61000	88	88	9	1	2	12	0.14	1	2	8.1	0.414	14.5
9020	34.153	-118.462	4	63640	40	40	10	0	3	13	0.33	6	1	8.1	0.414	14.5
9086	34.272	-118.469	4	64000	76	72	4	0	0	4	0.06	0	2	9.9	0.686	47.5
9085	34.272	-118.469	4	64000	76	70	12	0	0	12	0.17	1	4	9.9	0.686	47.5
9066	34.168	-118.395	4	68000	170	25	1	0	1	2	0.08	0	0	7.7	0.244	14.5
9117	34.152	-118.340	4	72000	48	48	10	0	1	11	0.23	0	8	7.4	0.244	14.5
9116	34.072	-118.362	4	73600	96	17	0	0	0	0	0.00	0	0	7.6	0.185	7.4
9064	34.038	-118.375	4	80000	228	21	3	0	0	3	0.14	0	0	7.6	0.428	14.1
9036	34.194	-118.900	4	80500	86	41	0	0	0	0	0.00	0	0	6.5	0.237	8.0

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9068	34.410	-118.574	4	86000	112	112	84	0	0	84	0.75	41	21	9.1	0.586	33.1
9189	34.051	-118.434	4	93502	120	18	0	0	0	0	0.00	0	0	7.9	0.357	10.8
9016	34.165	-118.466	4	94000	304	154	49	0	0	49	0.32	0	0	7.9	0.333	17.3
9087	34.183	-118.597	4	106800	136	102	32	0	0	32	0.31	0	18	8.4	0.362	19.1
9093	34.243	-118.532	4	124200	240	240	59	3	16	78	0.33	0	4	9.0	0.403	18.3
9035	34.201	-118.466	4	150000	208	37	0	0	0	0	0.00	0	0	7.9	0.32	12.7
9147	34.158	-118.408	5	50000	70	8	0	0	0	0	0.00	0	0	7.8	0.244	14.5
9128	34.174	-118.592	5	58500	100	97	0	0	0	7	0.07	0	0	8.3	0.362	19.1
9118	34.064	-118.197	5	68000	480	32	0	0	0	0	0.00	0	0	7.1	0.336	6.9
9070	34.077	-118.475	5	70800	100	100	13	0	2	15	0.15	4	0	7.3	0.171	8.8
9202	34.177	-118.466	5	77000	210	16	0	0	0	0	0.00	0	0	7.9	0.32	12.7
9168	34.185	-118.606	5	79000	210	57	0	0	0	1	0.02	0	0	8.4	0.362	19.1
9183	34.186	-118.466	5	79000	220	24	0	0	0	1	0.04	0	0	7.8	0.32	12.7
9127	34.195	-118.462	5	92000	144	21	0	0	0	0	0.00	0	0	7.8	0.32	12.7
9017	34.013	-118.493	5	102400	96	96	17	16	15	48	0.50	16	10	7.8	0.572	12.7
9030	34.154	-118.465	5	126020	1014	96	0	0	0	14	0.15	0	0	8.1	0.333	17.3
9060	34.174	-118.587	5	130000	288	284	23	3	2	28	0.10	1	1	8.3	0.362	19.1
9190	34.049	-118.445	5	134600	462	73	0	0	0	2	0.03	0	0	8.2	0.357	10.8
9037	34.202	-118.468	5	151000	268	58	0	0	0	0	0.00	0	0	7.9	0.32	12.7
9059	34.174	-118.587	5	250000	372	35	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9024	34.168	-118.605	5	376000	960	621	72	2	7	81	0.13	3	0	8.1	0.362	19.1
9165	34.066	-118.460	6	45000	156	23	0	0	0	0	0.00	0	0	7.5	0.171	8.8
9132	34.202	-118.630	6	50000	108	20	0	0	0	0	0.00	0	0	8.2	0.416	19.7
9138	34.144	-118.393	6	61200	168	56	0	0	0	0	0.00	0	0	7.6	0.244	14.5
9135	34.155	-118.369	6	70000	180	42	0	0	0	4	0.10	0	0	7.5	0.244	14.5
9161	34.221	-118.466	6	75000	190	35	0	0	0	2	0.06	0	0	8.1	0.439	13.7
9075	34.019	-118.498	6	90000	100	100	51	1	7	59	0.59	0	0	7.9	0.571	12.7
9012	34.176	-118.597	6	93514	192	192	59	0	1	60	0.31	6	0	8.3	0.362	19.1
9011	34.178	-118.597	6	93514	192	192	55	0	7	62	0.32	1	0	8.3	0.362	19.1
9050	34.158	-118.422	6	120000	228	72	43	0	0	43	0.60	0	0	8.0	0.244	14.5
9071	34.067	-118.445	6	120000	415	33	0	0	0	0	0.00	0	0	7.4	0.363	8.6
9082	34.156	-118.482	6	126000	378	8	0	0	0	0	0.00	0	0	7.7	0.381	16.5
9038	34.156	-118.482	6	150386	204	32	0	0	0	0	0.00	0	0	7.7	0.381	16.5

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9015	33.976	-118.393	6	172000	180	137	15	1	0	16	0.12	0	0	7.4	0.366	8.2
9029	34.159	-118.499	6	223000	252	207	0	0	0	0	0.00	0	0	7.4	0.328	12.9
9084	34.279	-118.737	6	239400	568	560	104	0	0	104	0.19	0	36	7.7	0.328	12.9
9077	34.067	-118.445	6	240000	360	36	0	0	0	0	0.00	0	0	7.4	0.363	8.6
9136	34.240	-118.570	6	250000	50	50	0	0	0	0	0.00	0	0	8.9	0.384	15.2
9199	34.036	-118.444	6	250000	700	87	0	0	0	2	0.02	0	0	8.1	0.444	19.4
9115	34.151	-118.341	6	267600	312	312	33	0	0	33	0.11	0	0	7.4	0.244	14.5
9014	34.156	-118.477	6	567000	964	189	14	0	1	15	0.08	0	0	7.8	0.381	16.5
9124	34.154	-118.368	8	116000	92	17	0	0	0	0	0.00	0	0	7.4	0.244	14.5
9048	34.157	-118.255	8	152000	224	8	0	0	0	0	0.00	0	0	7.0	0.22	6.9
9019	34.033	-118.450	8	230000	188	163	17	0	0	17	0.10	4	0	8.1	0.444	19.4
9034	34.165	-118.374	8	232000	208	66	0	0	0	4	0.06	0	0	7.5	0.244	14.5
9139	34.047	-118.446	9	70000	440	44	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9146	34.034	-118.456	9	164311	320	73	0	0	0	1	0.01	0	0	8.2	0.444	19.4
9033	34.038	-118.440	9	180000	278	40	0	0	0	0	0.00	0	0	8.1	0.444	19.4
9045	34.040	-118.438	10	183600	440	369	0	0	0	32	0.09	0	0	8.0	0.444	19.4
9044	34.040	-118.438	10	222400	503	232	0	0	0	38	0.16	0	0	8.0	0.444	19.4
9008	34.239	-118.564	10	275000	688	626	156	5	2	163	0.26	0	23	8.9	0.384	15.2
9009	34.179	-118.605	11	216640	342	342	25	6	2	33	0.10	9	6	8.3	0.362	19.1
9010	34.179	-118.605	11	216640	342	342	45	4	1	50	0.15	3	4	8.3	0.362	19.1
9028	34.053	-118.242	11	230560	528	521	53	0	2	55	0.11	0	7	6.7	0.152	6.5
9023	34.040	-118.438	11	409300	920	917	123	5	3	131	0.14	3	19	8.0	0.444	19.4
9080	34.156	-118.482	12	186000	392	94	1	0	0	1	0.01	0	0	7.8	0.381	16.5
9062	34.020	-118.498	13	221000	1042	299	10	0	0	10	0.03	0	0	7.9	0.571	12.7
9043	34.041	-118.470	13	244000	378	113	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9081	34.157	-118.485	13	247000	572	63	0	0	0	0	0.00	0	0	7.7	0.381	16.5
9046	34.033	-118.450	13	330000	823	750	0	0	0	130	0.17	0	0	8.1	0.444	19.4
9107	34.179	-118.600	13	332800	520	507	46	4	1	51	0.10	6	0	8.3	0.362	19.1
9105	34.033	-118.451	13	486720	764	107	34	2	12	48	0.45	1	2	8.1	0.444	19.4
9112	34.156	-118.255	14	262440	1344	19	0	0	0	0	0.00	0	0	7.1	0.22	6.9
9041	34.044	-118.467	14	344400	420	60	0	0	0	1	0.02	0	0	8.2	0.357	10.8
9148	34.154	-118.444	15	270000	576	133	0	0	0	3	0.02	0	0	8.2	0.244	14.5
9067	34.916	-118.391	15	315000	720	13	0	0	0	0	0.00	0	0	6.9	0.152	3.2

Table B-2 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted By Height (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9154	34.047	-118.445	16	225500	640	32	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9088	34.156	-118.480	16	233600	558	121	19	0	0	19	0.16	3	7	7.8	0.381	16.5
9195	34.048	-118.444	16	290600	962	50	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9144	34.058	-118.445	17	252000	480	28	0	0	0	0	0.00	0	0	7.8	0.243	9.7
9191	34.058	-118.460	17	295600	432	42	0	0	0	0	0.00	0	0	7.9	0.171	8.8
9078	34.058	-118.446	17	391000	704	12	0	0	0	0	0.00	0	0	7.8	0.243	9.7
9197	34.049	-118.462	17	400000	416	61	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9122	34.185	-118.597	18	345600	274	114	14	0	0	14	0.12	0	0	8.4	0.362	19.1
9121	34.185	-118.597	18	345600	278	251	23	0	0	23	0.09	0	0	8.4	0.362	19.1
9027	34.179	-118.605	20	385387	880	867	5	0	0	5	0.01	5	0	8.3	0.362	19.1
9120	34.157	-118.255	20	470000	488	8	0	0	0	0	0.00	0	0	7.0	0.22	6.9
9109	34.061	-118.414	20	510300	1040	10	0	0	0	0	0.00	0	0	7.5	0.238	9.1
9123	34.048	-118.445	21	300966	672	34	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9003	34.154	-118.464	21	369715	612	388	43	1	4	48	0.12	0	2	8.1	0.333	17.3
9156	34.169	-118.609	22	17500	34	10	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9001	34.179	-118.605	22	450000	624	603	21	0	0	21	0.03	0	2	8.3	0.362	19.1
9193	34.143	-118.361	24	nr	1104	67	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9079	34.051	-118.460	24	448800	1004	106	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9002	34.179	-118.605	26	586000	900	874	41	61	12	114	0.13	0	0	8.3	0.362	19.1
9074	34.062	-118.433	27	364500	766	22	0	0	9	9	0.41	0	0	7.6	0.363	8.6
9108	34.062	-118.417	28	672000	1232	18	0	0	0	0	0.00	0	0	7.5	0.238	9.1

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9028	34.053	-118.242	11	230560	528	521	53	0	2	55	0.11	0	7	6.7	0.152	6.5
9067	34.916	-118.391	15	315000	720	13	0	0	0	0	0.00	0	0	6.9	0.152	3.2
9070	34.077	-118.475	5	70800	100	100	13	0	2	15	0.15	4	0	7.3	0.171	8.8
9165	34.066	-118.460	6	45000	156	23	0	0	0	0	0.00	0	0	7.5	0.171	8.8
9191	34.058	-118.460	17	295600	432	42	0	0	0	0	0.00	0	0	7.9	0.171	8.8
9116	34.072	-118.362	4	73600	96	17	0	0	0	0	0.00	0	0	7.6	0.185	7.4
9054	34.139	-118.353	3	261000	135	120	33	2	10	45	0.38	6	8	7.2	0.187	8.0
9171	34.143	-118.361	3	865000	220	39	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9198	34.142	-118.361	4	nr	240	21	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9013	34.131	-118.344	4	26200	136	133	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9193	34.143	-118.361	24	nr	1104	67	0	0	0	0	0.00	0	0	7.2	0.187	8.0
9149	34.066	-118.469	1	15000	12	8	0	0	0	0	0.00	0	0	7.6	0.202	10.4
9048	34.157	-118.255	8	152000	224	8	0	0	0	0	0.00	0	0	7.0	0.220	6.9
9120	34.157	-118.255	20	470000	488	8	0	0	0	0	0.00	0	0	7.0	0.220	6.9
9112	34.156	-118.255	14	262440	1344	19	0	0	0	0	0.00	0	0	7.1	0.220	6.9
9036	34.194	-118.900	4	80500	86	41	0	0	0	0	0.00	0	0	6.5	0.237	8.0
9061	34.069	-118.400	3	108000	202	24	5	1	0	6	0.25	0	0	7.5	0.238	9.1
9109	34.061	-118.414	20	510300	1040	10	0	0	0	0	0.00	0	0	7.5	0.238	9.1
9108	34.062	-118.417	28	672000	1232	18	0	0	0	0	0.00	0	0	7.5	0.238	9.1
9144	34.058	-118.445	17	252000	480	28	0	0	0	0	0.00	0	0	7.8	0.243	9.7
9078	34.058	-118.446	17	391000	704	12	0	0	0	0	0.00	0	0	7.8	0.243	9.7
9162	34.053	-118.470	3	100000	28	4	0	0	0	0	0.00	0	0	8.1	0.243	9.7
9175	34.047	-118.465	3	25000	46	10	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9197	34.049	-118.462	17	400000	416	61	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9079	34.051	-118.460	24	448800	1004	106	0	0	0	0	0.00	0	0	8.2	0.243	9.7
9117	34.152	-118.340	4	72000	48	48	10	0	1	11	0.23	0	8	7.4	0.244	14.5
9115	34.151	-118.341	6	267600	312	312	33	0	0	33	0.11	0	0	7.4	0.244	14.5
9124	34.154	-118.368	8	116000	92	17	0	0	0	0	0.00	0	0	7.4	0.244	14.5
9135	34.155	-118.369	6	70000	180	42	0	0	0	4	0.10	0	0	7.5	0.244	14.5
9034	34.165	-118.374	8	232000	208	66	0	0	0	4	0.06	0	0	7.5	0.244	14.5
9138	34.144	-118.393	6	61200	168	56	0	0	0	0	0.00	0	0	7.6	0.244	14.5
9066	34.168	-118.395	4	68000	170	25	1	0	1	2	0.08	0	0	7.7	0.244	14.5
9182	34.143	-118.402	2	11000	10	5	0	0	0	0	0.00	0	0	7.8	0.244	14.5

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9147	34.158	-118.408	5	50000	70	8	0	0	0	0	0.00	0	0	7.8	0.244	14.5
9050	34.158	-118.422	6	120000	228	72	43	0	0	43	0.60	0	0	8.0	0.244	14.5
9131	34.157	-118.441	3	nr	480	33	0	0	0	0	0.00	0	0	8.1	0.244	14.5
9055	34.156	-118.431	4	42400	74	73	5	0	0	5	0.07	0	0	8.1	0.244	14.5
9143	34.149	-118.437	3	16000	34	20	0	0	0	5	0.25	0	0	8.2	0.244	14.5
9076	34.149	-118.437	3	45000	36	26	5	0	0	5	0.19	2	0	8.2	0.244	14.5
9004	34.150	-118.441	3	52000	114	114	17	0	0	17	0.15	0	9	8.2	0.244	14.5
9148	34.154	-118.444	15	270000	576	133	0	0	0	3	0.02	0	0	8.2	0.244	14.5
9183	34.186	-118.466	5	79000	220	24	0	0	0	1	0.04	0	0	7.8	0.320	12.7
9127	34.195	-118.462	5	92000	144	21	0	0	0	0	0.00	0	0	7.8	0.320	12.7
9035	34.201	-118.466	4	150000	208	37	0	0	0	0	0.00	0	0	7.9	0.320	12.7
9202	34.177	-118.466	5	77000	210	16	0	0	0	0	0.00	0	0	7.9	0.320	12.7
9037	34.202	-118.468	5	151000	268	58	0	0	0	0	0.00	0	0	7.9	0.320	12.7
9192	34.159	-118.501	3	35000	54	16	0	0	0	0	0.00	0	0	7.4	0.328	12.9
9029	34.159	-118.499	6	223000	252	207	0	0	0	0	0.00	0	0	7.4	0.328	12.9
9051	34.286	-118.744	2	22600	16	6	0	0	0	0	0.00	0	0	7.7	0.328	12.9
9084	34.279	-118.737	6	239400	568	560	104	0	0	104	0.19	0	36	7.7	0.328	12.9
9016	34.165	-118.466	4	94000	304	154	49	0	0	49	0.32	0	0	7.9	0.333	17.3
9030	34.154	-118.465	5	126020	1014	96	0	0	0	14	0.15	0	0	8.1	0.333	17.3
9003	34.154	-118.464	21	369715	612	388	43	1	4	48	0.12	0	2	8.1	0.333	17.3
9119	34.064	-118.197	4	36800	240	30	0	0	0	0	0.00	0	0	7.1	0.336	6.9
9118	34.064	-118.197	5	68000	480	32	0	0	0	0	0.00	0	0	7.1	0.336	6.9
9092	34.198	-118.602	3	11100	24	12	0	0	0	0	0.00	0	0	8.5	0.353	16.7
9189	34.051	-118.434	4	93502	120	18	0	0	0	0	0.00	0	0	7.9	0.357	10.8
9194	34.048	-118.442	3	20000	36	5	0	0	0	0	0.00	0	0	8.1	0.357	10.8
9157	34.048	-118.443	3	22000	24	8	0	0	0	0	0.00	0	0	8.1	0.357	10.8
9152	34.046	-118.448	3	nr	28	13	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9022	34.042	-118.469	3	50240	100	100	13	0	1	14	0.14	0	0	8.2	0.357	10.8
9190	34.049	-118.445	5	134600	462	73	0	0	0	2	0.03	0	0	8.2	0.357	10.8
9139	34.047	-118.446	9	70000	440	44	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9043	34.041	-118.470	13	244000	378	113	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9041	34.044	-118.467	14	344400	420	60	0	0	0	1	0.02	0	0	8.2	0.357	10.8
9154	34.047	-118.445	16	225500	640	32	0	0	0	0	0.00	0	0	8.2	0.357	10.8

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9195	34.048	-118.444	16	290600	962	50	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9123	34.048	-118.445	21	300966	672	34	0	0	0	0	0.00	0	0	8.2	0.357	10.8
9160	34.158	-118.592	2	48000	5	5	0	0	0	0	0.00	0	0	7.9	0.362	19.1
9167	34.167	-118.584	3	nr	76	8	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9170	34.170	-118.606	3	25600	62	16	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9164	34.170	-118.606	3	30000	50	13	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9018	34.167	-118.584	3	82000	108	65	11	0	0	11	0.17	0	6	8.1	0.362	19.1
9024	34.168	-118.605	5	376000	960	621	72	2	7	81	0.13	3	0	8.1	0.362	19.1
9156	34.169	-118.609	22	17500	34	10	0	0	0	0	0.00	0	0	8.1	0.362	19.1
9169	34.169	-118.576	3	20400	54	10	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9186	34.173	-118.603	3	60000	80	14	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9063	34.171	-118.606	3	63000	68	68	9	0	0	9	0.13	0	0	8.2	0.362	19.1
9106	34.172	-118.605	3	84000	132	33	0	0	0	0	0.00	0	0	8.2	0.362	19.1
9166	34.172	-118.604	3	120000	140	124	0	0	0	3	0.02	0	0	8.2	0.362	19.1
9025	34.172	-118.603	3	120000	216	106	30	0	0	30	0.28	0	6	8.2	0.362	19.1
9126	34.175	-118.589	1	16300	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9184	34.175	-118.589	1	17700	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9188	34.174	-118.591	1	19130	8	6	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9137	34.179	-118.599	1	20000	12	12	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9134	34.174	-118.591	1	20710	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9200	34.175	-118.589	1	21500	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9130	34.175	-118.592	1	22400	8	8	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9133	34.175	-118.589	1	23400	12	9	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9163	34.175	-118.590	1	23500	8	6	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9178	34.174	-118.587	1	24670	10	10	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9007	34.175	-118.590	2	62800	100	97	7	0	0	7	0.07	0	5	8.3	0.362	19.1
9056	34.174	-118.587	3	48000	84	16	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9057	34.174	-118.587	3	48000	84	79	1	0	0	1	0.01	0	0	8.3	0.362	19.1
9005	34.171	-118.657	3	59700	36	36	7	0	0	7	0.19	0	0	8.3	0.362	19.1
9141	34.174	-118.587	3	75000	114	22	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9032	34.177	-118.597	4	46375	112	112	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9128	34.174	-118.592	5	58500	100	97	0	0	0	7	0.07	0	0	8.3	0.362	19.1
9060	34.174	-118.587	5	130000	288	284	23	3	2	28	0.10	1	1	8.3	0.362	19.1

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9059	34.174	-118.587	5	250000	372	35	0	0	0	0	0.00	0	0	8.3	0.362	19.1
9012	34.176	-118.597	6	93514	192	192	59	0	1	60	0.31	6	0	8.3	0.362	19.1
9011	34.178	-118.597	6	93514	192	192	55	0	7	62	0.32	1	0	8.3	0.362	19.1
9009	34.179	-118.605	11	216640	342	342	25	6	2	33	0.10	9	6	8.3	0.362	19.1
9010	34.179	-118.605	11	216640	342	342	45	4	1	50	0.15	3	4	8.3	0.362	19.1
9107	34.179	-118.600	13	332800	520	507	46	4	1	51	0.10	6	0	8.3	0.362	19.1
9027	34.179	-118.605	20	385387	880	867	5	0	0	5	0.01	5	0	8.3	0.362	19.1
9001	34.179	-118.605	22	450000	624	603	21	0	0	21	0.03	0	2	8.3	0.362	19.1
9002	34.179	-118.605	26	586000	900	874	41	61	12	114	0.13	0	0	8.3	0.362	19.1
9087	34.183	-118.597	4	106800	136	102	32	0	0	32	0.31	0	18	8.4	0.362	19.1
9168	34.185	-118.606	5	79000	210	57	0	0	0	1	0.02	0	0	8.4	0.362	19.1
9122	34.185	-118.597	18	345600	274	114	14	0	0	14	0.12	0	0	8.4	0.362	19.1
9121	34.185	-118.597	18	345600	278	251	23	0	0	23	0.09	0	0	8.4	0.362	19.1
9071	34.067	-118.445	6	120000	415	33	0	0	0	0	0.00	0	0	7.4	0.363	8.6
9077	34.067	-118.445	6	240000	360	36	0	0	0	0	0.00	0	0	7.4	0.363	8.6
9074	34.062	-118.433	27	364500	766	22	0	0	9	9	0.41	0	0	7.6	0.363	8.6
9015	33.976	-118.393	6	172000	180	137	15	1	0	16	0.12	0	0	7.4	0.366	8.2
9082	34.156	-118.482	6	126000	378	8	0	0	0	0	0.00	0	0	7.7	0.381	16.5
9038	34.156	-118.482	6	150386	204	32	0	0	0	0	0.00	0	0	7.7	0.381	16.5
9081	34.157	-118.485	13	247000	572	63	0	0	0	0	0.00	0	0	7.7	0.381	16.5
9014	34.156	-118.477	6	567000	964	189	14	0	1	15	0.08	0	0	7.8	0.381	16.5
9080	34.156	-118.482	12	186000	392	94	1	0	0	1	0.01	0	0	7.8	0.381	16.5
9088	34.156	-118.480	16	233600	558	121	19	0	0	19	0.16	3	7	7.8	0.381	16.5
9177	34.155	-118.472	3	43000	96	20	0	0	0	1	0.05	0	0	8.0	0.381	16.5
9129	34.187	-118.502	3	50000	57	15	0	0	0	1	0.07	0	0	8.1	0.384	15.2
9159	34.187	-118.502	3	50000	57	13	0	0	0	2	0.15	0	0	8.1	0.384	15.2
9125	34.187	-118.502	3	50000	69	20	0	0	0	2	0.10	0	0	8.1	0.384	15.2
9185	34.187	-118.502	3	50000	75	14	0	0	0	0	0.00	0	0	8.1	0.384	15.2
9006	34.201	-118.495	2	49000	64	64	4	0	1	5	0.08	0	0	8.2	0.384	15.2
9153	34.201	-118.495	3	40000	64	64	0	0	0	4	0.06	0	0	8.2	0.384	15.2
9142	34.201	-118.495	3	70000	64	11	0	0	0	0	0.00	0	0	8.2	0.384	15.2
9040	34.234	-118.562	3	17700	46	9	0	0	0	0	0.00	0	0	8.9	0.384	15.2
9136	34.240	-118.570	6	250000	50	50	0	0	0	0	0.00	0	0	8.9	0.384	15.2

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9008	34.239	-118.564	10	275000	688	626	156	5	2	163	0.26	0	23	8.9	0.384	15.2
9155	34.265	-118.573	3	22000	114	17	0	0	0	0	0.00	0	0	9.0	0.384	15.2
9110	34.019	-118.457	2	112000	172	15	0	0	0	0	0.00	0	0	8.0	0.395	11.7
9094	34.236	-118.528	2	17930	72	27	1	0	0	1	0.04	0	0	8.9	0.403	18.3
9096	34.236	-118.528	2	31800	68	68	0	0	0	0	0.00	0	0	8.9	0.403	18.3
9031	34.239	-118.568	2	34600	50	50	0	0	0	0	0.00	0	0	8.9	0.403	18.3
9026	34.239	-118.564	2	98444	84	82	22	0	3	25	0.30	2	5	8.9	0.403	18.3
9095	34.236	-118.528	4	27880	96	96	11	0	1	12	0.13	3	0	8.9	0.403	18.3
9093	34.243	-118.532	4	124200	240	240	59	3	16	78	0.33	0	4	9.0	0.403	18.3
9097	34.243	-118.532	3	45900	172	168	26	0	3	29	0.17	0	3	9.1	0.403	18.3
9021	34.153	-118.462	4	61000	88	88	9	1	2	12	0.14	1	2	8.1	0.414	14.5
9020	34.153	-118.462	4	63640	40	40	10	0	3	13	0.33	6	1	8.1	0.414	14.5
9132	34.202	-118.630	6	50000	108	20	0	0	0	0	0.00	0	0	8.2	0.416	19.7
9039	34.173	-118.561	3	25000	22	7	0	0	0	0	0.00	0	0	8.2	0.417	12.9
9187	34.171	-118.543	3	53000	57	9	0	0	0	0	0.00	0	0	8.1	0.418	13.0
9049	34.410	-118.459	2	36000	68	59	1	0	0	1	0.02	0	0	8.6	0.425	16.7
9064	34.038	-118.375	4	80000	228	21	3	0	0	3	0.14	0	0	7.6	0.428	14.1
9172	34.213	-118.475	3	68000	162	16	0	0	0	0	0.00	0	0	8.1	0.439	13.7
9161	34.221	-118.466	6	75000	190	35	0	0	0	2	0.06	0	0	8.1	0.439	13.7
9201	34.047	-118.434	3	nr	96	23	0	0	0	0	0.00	0	0	8.0	0.444	19.4
9140	34.047	-118.435	4	58000	298	43	0	0	0	1	0.02	0	0	8.0	0.444	19.4
9045	34.040	-118.438	10	183600	440	369	0	0	0	32	0.09	0	0	8.0	0.444	19.4
9044	34.040	-118.438	10	222400	503	232	0	0	0	38	0.16	0	0	8.0	0.444	19.4
9023	34.040	-118.438	11	409300	920	917	123	5	3	131	0.14	3	19	8.0	0.444	19.4
9103	34.032	-118.456	3	13500	72	32	1	0	0	1	0.03	0	0	8.1	0.444	19.4
9102	34.032	-118.456	3	16800	102	44	4	0	0	4	0.09	0	0	8.1	0.444	19.4
9104	34.032	-118.456	3	21000	80	32	0	0	0	0	0.00	0	0	8.1	0.444	19.4
9100	34.032	-118.456	3	33600	152	76	11	0	2	13	0.17	0	0	8.1	0.444	19.4
9101	34.032	-118.456	3	51000	312	154	12	0	0	12	0.08	0	0	8.1	0.444	19.4
9199	34.036	-118.444	6	250000	700	87	0	0	0	2	0.02	0	0	8.1	0.444	19.4
9019	34.033	-118.450	8	230000	188	163	17	0	0	17	0.10	4	0	8.1	0.444	19.4
9033	34.038	-118.440	9	180000	278	40	0	0	0	0	0.00	0	0	8.1	0.444	19.4
9046	34.033	-118.450	13	330000	823	750	0	0	0	130	0.17	0	0	8.1	0.444	19.4

Table B-3 185 Inspected WSMF Buildings Affected by the 1994 Northridge Earthquake Sorted by Peak Ground Acceleration (As of 11/99) (continued)

ID	Lat	Long	Sty	Area	Conn	Insp	Bot	Top	B&T	Total	DR	Shr	PZ	MMI	Pga	Pgv
9105	34.033	-118.451	13	486720	764	107	34	2	12	48	0.45	1	2	8.1	0.444	19.4
9047	34.042	-118.444	3	54000	146	38	16	0	1	17	0.45	0	0	8.2	0.444	19.4
9146	34.034	-118.456	9	164311	320	73	0	0	0	1	0.01	0	0	8.2	0.444	19.4
9111	34.015	-118.491	3	23100	48	6	0	0	0	0	0.00	0	0	7.9	0.571	12.7
9065	34.020	-118.497	4	52000	78	49	0	0	0	0	0.00	0	0	7.9	0.571	12.7
9075	34.019	-118.498	6	90000	100	100	51	1	7	59	0.59	0	0	7.9	0.571	12.7
9062	34.020	-118.498	13	221000	1042	299	10	0	0	10	0.03	0	0	7.9	0.571	12.7
9017	34.013	-118.493	5	102400	96	96	17	16	15	48	0.50	16	10	7.8	0.572	12.7
9090	34.412	-118.554	1	7500	30	18	1	1	2	4	0.22	0	0	9.0	0.586	33.1
9114	34.412	-118.554	2	15400	16	16	8	0	0	8	0.50	0	4	9.0	0.586	33.1
9089	34.412	-118.554	2	19400	40	14	4	0	0	4	0.29	0	0	9.0	0.586	33.1
9091	34.412	-118.554	2	57600	480	77	0	0	0	0	0.00	0	0	9.0	0.586	33.1
9098	34.412	-118.557	3	81600	nr	48	0	0	0	0	0.00	0	0	9.0	0.586	33.1
9069	34.410	-118.574	1	27000	20	20	10	0	0	10	0.50	0	9	9.1	0.586	33.1
9068	34.410	-118.574	4	86000	112	112	84	0	0	84	0.75	41	21	9.1	0.586	33.1
9053	34.029	-118.480	4	54200	132	110	9	1	0	10	0.09	5	0	8.1	0.642	21.1
9086	34.272	-118.469	4	64000	76	72	4	0	0	4	0.06	0	2	9.9	0.686	47.5
9085	34.272	-118.469	4	64000	76	70	12	0	0	12	0.17	1	4	9.9	0.686	47.5
9179	34.261	-118.502	3	42000	90	19	0	0	0	6	0.32	0	0	9.4	0.841	27.6
9158	34.262	-118.503	3	67000	28	9	0	0	0	0	0.00	0	0	9.4	0.841	27.6

APPENDIX C. OVERVIEW OF DAMAGE TO STEEL BUILDING STRUCTURES OBSERVED IN THE 1995 KOBE EARTHQUAKE

THE 1995 HYOGOKEN-NANBU (KOBE) EARTHQUAKE

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C.1 Summary

This appendix presents an overview of damage to steel building structures observed following the 1995 Hyogoken-Nanbu (Kobe) earthquake. Damage statistics are presented with respect to the number of stories, type of structural framing, location of damaged elements and severity of damage. Standard practices exercised in Japan before the earthquake and causes of damage discussed immediately after the earthquake are introduced in terms of materials, welding, beam-to-column connection details and seismic design forces. Efforts are made to compare these with corresponding U.S. practices. A partial summary of post-Kobe research activities in Japan on steel structures is also presented.

C.2 Introduction

The Hyogoken-Nanbu (Kobe) earthquake shook Kobe and surrounding areas on January 17, 1995, exactly one year after the Northridge earthquake. More than 5,000 people were killed, 35,000 people were injured and 300,000 more were rendered homeless by this earthquake. The direct cost of structural damages caused by the earthquake exceeded $150 billion. Substantial damage was experienced by reinforced concrete and steel highway structures, as well as by wood, concrete and steel buildings.

In the long history of large Japanese earthquakes, the Kobe earthquake was the first to cause widespread and serious damage to modern steel buildings. Numerous steel buildings had been shaken strongly by other recent earthquakes, such as the 1978 Miyagiken-oki earthquake that struck urban Sendai City. However, these recent earthquakes have caused only minimal damage to steel buildings.

Why did the Kobe earthquake damage modern steel buildings so badly? Many suggest that ground motions in Kobe were much larger than those experienced during previous earthquakes in Japan. Moreover, the Kobe area is one of the earliest urban developments in Japan and, consequently, contained a large inventory of older steel buildings designed when design criteria were not as advanced as today. Whatever reasons may be given, the fact remains that modern steel buildings experienced significant damage, refuting the popular myth in Japan that steel buildings are immune to strong earthquakes.

To help put this situation into perspective, this section presents (1) an overview of damage to steel buildings observed following the Kobe earthquake, (2) plausible causes of this damage as discussed by the Japanese engineering community immediately after the earthquake, (3) some comparisons between the types of damage observed and the post-earthquake actions taken in the U.S. and Japan, and (4) a partial summary of the efforts being conducted in Japan with respect to the improvement of seismic safety of modern steel buildings.

C.3 Damage to Steel Buildings

Steel Building Construction in Japan

Steel is a very popular structural material in Japanese building construction. Figure C-1(a) compares the total floor area of steel buildings constructed each year with that employed in construction using other structural materials. Wood has ranked first for years, but it is used almost exclusively for residential houses. Steel ranks second, followed by reinforced concrete (RC) and steel-encased reinforced concrete (designated SRC in Japan). Figure C-1(b) shows the total floor area of steel buildings constructed each year with respect to the number of stories in each building. This figure suggests that the vast majority of steel buildings are shorter than five stories in height. In fact, most of steel buildings in Japan are low-rise, used for offices, shops and mixed occupancies, as well as for industrial and manufacturing structures.

Damage to Older Steel Buildings

The Architectural Institute of Japan (AIJ) conducted a preliminary field reconnaissance of Kobe from January 24 to 26, 1995, and identified 4,530 engineered buildings that were damaged, including 1,067 that collapsed or were damaged beyond repair (AIJ 1995a, 1997a). The Kobe area contains many engineered buildings constructed before the major economic growth of the post-war era. As a consequence, a large stock of steel buildings more than 35 years old were subjected to the effects of the ground shaking.

Figure C-2 shows examples of damage to such older steel buildings. As shown in Figure C-2(b), these buildings generally were constructed with bundled light-gauged sections for columns; beams were typically fabricated using shallow trusses consisting of light-gauge rolled sections and round bars. Unfortunately, these old buildings lacked significant earthquake resistance, in comparison with modern seismic design codes. Many of these buildings also suffered considerable corrosion and other material deterioration, as shown in Figure C-2(c). According to a preliminary estimate, over 70 % of the damaged steel buildings located in Kobe City were of this older construction type.

Damage to Newer Steel Buildings

From mid-February to mid-March, 1995, the Steel Committee of the Kinki Branch of the Architectural Institute of Japan conducted a detailed survey of the damage to modern steel buildings and located 988 damaged steel buildings (AIJ 1995b). Among those buildings, 90 were rated as having collapsed, 332 as being severely damaged, 266 as moderately damaged, and 300 had minor damage. Figure C-3 shows the number of buildings with respect to the damage level, indicating that most of the collapsed buildings were 2 to 5 stories tall. No building with seven or more stories collapsed.

Damaged buildings were classified as having rigid moment-resisting (R) frames or braced (B) frames. Thus, considering the two principal framing orientations of a building results in the following framing designations: R-R (an unbraced moment resisting frame in two horizontal directions), R-B (an unbraced frame in one horizontal direction and a braced frame in the other direction), and B-B (a braced frame in both horizontal directions). Considering the 988 damaged steel buildings, 432 were R-R, 134 were R-B and 34 were B-B, with 388 with an unidentified framing system. These statistics indicate that the majority of damaged buildings had moment-resisting frames. Although these statistics were for damaged buildings only, they are also believed to reflect the general distribution among framing systems used in modern Japanese steel buildings.

Table C-1 indicates cross-sectional types used for columns, beams, and braces. Beams consisted almost exclusively of wide-flange sections, either rolled or built-up. For columns, wide-flange (H) sections were used most extensively, followed by square-tube sections. During the past 25 years, it is notable that square-tube (commonly cold-formed for low- to moderate-rise construction) sections have been used more frequently for columns than have wide flange sections. In braced frames, rods, angles, flat bars, round-tubes, wide-flanges, square-tubes, and channels were used for the braces.

Table C-2 indicates the type of connection details found in the damaged buildings. In moment-resisting frames, short stubs of wide flange beams are generally shop welded to the columns in a so-called Christmas tree arrangement. The beam stubs were then field bolted (using high-tension bolts) to the central portion of the beam. Column splices were mostly accomplished by welding. In braced frames, braces were connected mostly by bolting, except for small rod and flat bar braces, which are generally welded.

Figure C-4 shows two typical types of Japanese beam-to-column connections, namely the through-diaphragm connection and the interior diaphragm connection. Of these connections, the through-diaphragm connection is by far most popular, as indicated in Table C-2(c). In the through-diaphragm connection, the square-tube used for the column is cut longitudinally into three pieces: one used for the column of the lower story, one for the connection's panel zone (a short piece, often called a ‘dice’ in Japan), and one for the column of the upper story. Two diaphragm plates are inserted between the three separated pieces and shop-welded all around. Then, short segments of beams are welded to the panel zone: the beam flanges are welded directly to the diaphragms and the web to the side of the dice. The entire piece (often called a Christmas tree) is transported to the construction site and connected to the mid-portion of the beam by high-tension bolts.

Figure C-5 shows three typical types of column base connections, including the standard base plate connection, the concrete encased column base connection, and the embedded column base connection. As evidenced in Table C-2(d), standard base plate connections were most commonly used.

C.4 General Damage Statistics for Modern Steel Buildings

Figure C-6 shows the correlation between damage level and structural framing type, which is further sub-divided according to the type of column used. This figure indicates that no significant difference existed in damage level with respect to the structural framing type (R-R, R-B, and B-B). Interestingly, buildings with wide-flange columns suffered somewhat more serious damage in comparison with buildings having other column types. This may be attributed to building age, since square-tube sections are generally used in recent construction. Figure C-7 shows the location of damage (columns, beams, beam-to-column connections, braces, and column bases) as a function of frame type. Major observations from the data in this figure are as follows: (1) columns in unbraced frames suffered the most damage relative to other frame elements (in terms of the number of buildings), while braces in braced frames were the most frequently damaged structural element; (2) in unbraced frames, damage to beam-to-column connections and column bases was also significant; (3) damage to beam-to-column connections was most significant for unbraced frames having square-tube columns; and (4) damage to columns was most significant for unbraced frames having wide-flange columns. Unfortunately, these particular statistics are limited in that they do not include information on the building age or the types of connections and members used in non-damaged steel buildings.

The Building Research Institute (BRI) of the Ministry of Construction of Japan conducted a more detailed survey of about 630 steel buildings (not including old steel buildings made of light-gauged sections) located in a severely shaken area (Midorikawa et al. 1997). According to the BRI survey, the approximate distribution of damage severity was 17% collapse/severe, 17% moderate, 33% minor, and 33% no damage. The incidence of damage to columns, braces, and column bases were found to be significantly lower for buildings constructed after 1981 than for those constructed before. This may be a function of the major changes in Japan's Seismic Design Code that occurred in 1981, or other changes in construction practices that occurred over time. On the other hand, no significant difference was found in the BRI study between damage to beam-to-column connections in buildings constructed before and after 1981.

C.5 Damage to Members in Modern Buildings

Columns

Damage occurred in many columns. Most of this damage occurred near beam-to-column connections. Damage to columns included plastification, excessive distortion and local buckling near the column ends, as well as fractures in the base metals and at column splices. Many wide-flange columns sustained excessive bending about their weak-axis. Figure C-8 shows a fracture in the base metal of a cold-formed square tube having dimensions of 450 mm x 450 mm x 25 mm. The fracture surface was mostly brittle, but significant local buckling appeared above the fractured section, suggesting that the column had been loaded well beyond its yield stress.

A cluster of modern high-rise residential steel buildings, located near the seashore in Ashiyahama, exhibited fractures in over 50 large-sized columns and braces The fractured columns were made of square sections (not cold-formed sections), with an outer dimension of 500-550 mm and a wall thickness of 50-55 mm. Fracture occurred in the base metals (Figure C-

9(a)), at welded column splices, and at beam-to-brace connections (Figure C-9(b)). In these fractures, surfaces were rather rough, exhibiting shear lips and tear ridges, which confirmed that the fracture was brittle involving only limited plastification.

Braces

Damage to braces was found to be more severe in relatively smaller cross-sections (rods, angles, and flat plates). Although this was not quantified, the size of cross-sections appears to correlate strongly with the building age, with small cross-sections used more frequently in older buildings. Damage to braces with larger cross-sections was concentrated mostly at their connections with the adjoining beams or columns. Figure C-10(a) shows an example of such damage, in which connection bolts were broken. Another example, in which a beam connected to a pair of braces sustained significant web-plate buckling and out-of-plane distortion, is shown in Figure C-10(b). On the other hand, it was not unusual for braced frames located in severely shaken areas to have suffered only minimal damage; in these cases, the braces were firmly connected to the adjoining beams and columns. In summary, damage to braces having large cross-sections was strongly correlated with the connection details, and poor connection details suffered more severe damage.

Column Bases

Damage to column bases was relatively common. Most of the damage to column bases was observed for the standard base plate connections. Figure C-11 shows the damage level with respect to the location of damage in the standard base plate connections, indicating that the majority of damage occurred to the anchor bolts. In present seismic design practice, standard base plate connections are commonly designed assuming them to be pin supports (meaning that no moment transfer is considered for design at the column base). Regardless of such assumptions, column bases must securely withstand shear forces under load reversals. The design profession in Japan has come to recognize that some of the practices used for the design of these connections before the Kobe earthquake were inadequate.

Beam-to-Column Connections

As stated earlier, many fractures were observed in beam-to-column welded connections. Fractures to beam-to-column connections were essentially divided into two types. Figure C-12 shows the first type of fracture, which occurred in beam-to-column (shop-welded) connections where columns, beam, and connection panels were fillet-welded using rather small sized welds. At a glance, it was understood that such small welds could hardly transfer the stresses exerted on the connections. In fact, they generally fractured without any observed plastification in the neighboring columns and beams.

The second type of fracture was observed in many beam-to-column connections that employed full-penetration welding. Fractures were mostly brittle and occurred in the weld deposit, heat-affected zones, base metals (initiating from the toe of the weld access holes), and diaphragm plates (Figure C-13). From instances where such fractures occurred, the following observations could be made: (1) residual story drifts were not significant; (2) damage to interior and exterior finishes were minimal; (3) fractures occurred mostly at beam bottom flanges only; (4) significant yielding, plastification and local buckling of beam bottom flanges were observed, indicating that the beams dissipated some energy before fracture; and (5) such plastification occurred only in the beams while the adjoining columns remained almost elastic. The reason for observation (5) may be that many designers adopted strong column – weak beam proportioning concepts, as well as the fact that the real yield strength of a square tube column was usually significantly higher than the nominal value due to the cold-forming of the steel during the manufacture of square tube sections.

Figure C-14 shows a set of results of tests conducted for the base metal of a fractured beam (Nakashima et al. 1998). The Charpy V-notch test shows that the material could absorb more than 50 Joules of energy prior to fracture at zero degree centigrade. The base metal near the fractured surface was significantly hardened, which suggests the base metal in the fractured connection might have sustained significant plastic strain before fracture.

C.6 Design and Construction Practices Before Kobe Earthquake Damage

Among various types of the damage described above, the damage to welded beam-to-column connections of modern steel buildings has posed one of the most serious problems in the steel building community. The following summarizes pre-Kobe design and fabrication practices for Japanese steel buildings. Also noted are some relations between these practices and damage as discussed by the Japanese design community immediately after the Kobe earthquake.

Materials

Japan uses both the integrated casting and continuous casting (mini-mill) steels for beam members. The market share of continuous casting steels is higher for smaller, thinner sections (commonly less than 450 mm deep and thinner than 40 mm). Immediately after the Kobe earthquake, some concerns were raised about the fracture toughness of such steels, particularly at the flange-web junction. However, it was confirmed that the steels in recent continuous casting production are equivalent in general quality to corresponding steels from integrated casting mills. Further, it is notable that large construction companies establish their own in-house regulations for quality assurance when using steels from continuous casting mills.

At the time of the Kobe earthquake, Japan was in a process of introducing new types of structural steels, called the SN steels. The development of SN steel initiated long before the Kobe earthquake. Both upper and lower limits are specified for the yield and maximum strengths of this steel, and an upper bound of 0.8 is specified for the ratio of the yield to the maximum strength in order to ensure good ductility. These steels have been commercially available and, particularly after the Kobe earthquake, designers have been more interested in the use of these steels.

Connection Details

As previously discussed, the design and fabrication of recent Japanese beam-to-column connections are characterized as follows: (1) square tube (often cold-formed) sections are used for columns; (2) the through-diaphragm connection (Figure C-4(a)) is by far most popular means for accomplishing the beam-to-column connection; and (3) welding is accomplished in the shop, where automatic welding robots are increasingly used in some shops.

A constructor conducted a detailed survey on several damaged steel buildings in which shop-welded through-diaphragm connections were used (1997a). Out of 2,396 connections surveyed, 79 were found to have damage including complete fractures and partial cracks. The damage percentage is 3.3%. Figure C-15 shows the fracture and crack distribution with respect to the damage location, indicating that 20.5 % of the damaged connections fractured in the base metal. In these instances, the fracture initiated from the toe of the weld access hole. It was speculated that this type of fracture was attributed primarily to a combined effect of stress and strain concentration at the toe and low fracture toughness of the material at the flange-web junction. After the Kobe earthquake, extensive studies have been undertaken to determine how to avoid this type of fracture, and various modified connection details have been proposed, as will be explained later. Most of the modified details reduce the size of the weld access holes, aiming to mitigate the stress concentration at the toe.

In the through-diaphragm connection, the beam web is shop-welded directly to the column flange, but normally the column is supplied without any vertical stiffener or diaphragm at the back of the beam web (Figure C-4(a)). Because of the flexibility of the column flange in out-of-plane deformation, the moment resisted by the beam web is reduced compared to that developed for an ideal rigid support, causing stress concentrations in the beam flanges. Concerns were raised about this issue after the earthquake.

As stated earlier, most of the fractures in beam-to-column connections occurred at beam bottom flanges only, which is very similar to what has been observed from the 1994 Northridge earthquake. In the U.S., the coincidence of the weld root, backing bar, and most stressed fiber due to local deformations was said to be a source of bottom flange fractures. On the other hand, Japanese shop welding enables welding of the beam bottom flange from the bottom side, as shown in Figure C-4(a), where the weld root is located on the interior side of the bottom flange. The observation that many of Japanese shop welded beam-to-column connections also fractured only at bottom flanges indicates that the root location is not the sole cause of bottom flange fractures. Composite action with floor slabs, another source being addressed, is a strong suspect because in Kobe some beam-to-column connections fractured in both the top and bottom flanges when floor slabs were not present.

Cold-formed tubes are limited to thicknesses less than 40 mm and dimensions less than 1000 mm. Therefore, columns for tall buildings and columns with large axial loads are generally constructed as heavy built-up columns with the connection shown in Figure C-4(b). The built-up box sections require extensive fabrication labor because of the longitudinal seam and internal diaphragm welds. This welding is completed in the shop. When such large, heavy box columns are used, connections with shop-attached beam stubs projecting from the column (Figure C-4(a)) are no longer feasible, because of difficulties in transportation and the size of the required bolted beam splice. Thus, connections similar to the U.S. practice (i.e. field welded beam flange to column joints and bolted web attachments) are employed. No serious damage was reported for this type of connection (AIJ 1995b) during the Kobe earthquake.

Welding

In recent Japanese construction, semi-automatic CO^2- (or sometimes argon-mixed) shielded metal arc welding has been used almost exclusively for the welding of beam flanges. This practice is common not only in shop welding, but in field welding as well. Self-shielded flux-cored electrodes were introduced in the late 1960s and used in the early 1970s primarily for field welding, with electrodes developed in Japan. It lost favor, however, and gas shielded metal arc welding has been used almost exclusively in recent years.

As shown in Figure C-15, 24.4 % of the damaged connections had complete fracture along the weld metal, 10.3 % had cracks at craters, and 37.2 % had cracks initiating from run-off tabs. This large percentage of damage associated with welding was striking, and serious concerns were raised about the present welding practice. Higher voltages and larger deposition rates than those stipulated in regulations and excessive weaving were thought to be the likely causes.

Seismic Design Forces

The present Japanese seismic design code, adopted in 1981, provides two levels (Levels-I and -II) of design earthquake forces. Level-I is for small to medium earthquakes with maximum ground accelerations ranging between 0.8 and 1.0 m/sec^2. To ensure serviceability, structural systems are required to remain elastic during such earthquakes. The Level-II design earthquake represents a large earthquake with the maximum ground acceleration ranging approximately from 3.0 to 4.0 m/sec^2. For such large earthquakes, collapse prevention is a typical design consideration, and some damage to structures (meaning plastic deformation in members and connections) is permitted. Based upon these maximum ground accelerations, Japan's Seismic Design Code stipulates 0.2g and 1.0g (g is the acceleration of gravity) as the maximum design base shear for Levels-I and -II, respectively. For Level-II, a trade-off between strength and ductility capacities is taken into account, and a reduction factor of 4.0 is adopted for most ductile steel moment frames. Figure C-16 shows the Level-II design base shear unreduced for ductility corresponding to medium soil conditions. Also drawn in this figure are pseudo-acceleration spectra (with 2% damping) for a dozen large ground motions recorded in the Kobe earthquake. This plot clearly indicates that some of response accelerations are significantly larger than code specified values. Considering such large recorded ground motions as well as Japan's seismic design philosophy, it was understood that steel buildings located in severely shaken regions were expected to sustain significant structural damage even if they were built in conformance with the present design and construction practices.

C.7 Comparison of Building Damage in the U.S. and Japan

Damage Similarities

In both the Northridge and Kobe earthquakes, steel buildings sustained significant damage, and many similarities in damage patterns were disclosed. Some notable similarities are summarized as follows.

1. Steel buildings in Japan and the U.S. had not experienced much damage in previous earthquakes. These two earthquakes exposed for the first time in each country the potential significant damage in welded steel moment resisting frame buildings.
2. Many modern building structures designed and constructed with present practices were damaged. Thus, damage was not exclusively associated with old technologies and design practices.
3. Much damage was found. However, no building constructed using the most recent design and construction practices collapsed.
4. Many welded beam-to-column connections failed by fracturing, indicating that welded connections were one of the weakest locations in steel moment frames.

Differences in Damage, Design and Construction

Differences in damage patterns and sources were also observed. Notable differences are summarized as follows.

1. Beam-to-column connections fractured, but in many instances fractures in Japanese structures were preceded by significant plastification and local buckling, meaning that the beams dissipated some energy before fracture. The vast majority of fractures in the U.S. involved virtually no plastification in either beams or columns. Thus, the degree of plastic rotation capacity of steel beams-to-column connections constructed using pre-Northridge and pre-Kobe practices may have been significantly different.
2. Steel materials used may also be different. It appears that Japan has placed relatively more attention to the importance of material strength and strain hardening for securing beam plastic rotation capacity. Development of SN steels before the Kobe earthquake may be an indirect indication of this. The use of so-called dual-certified steels in the U.S prior to the Northridge earthquake suggests less concern in this regard.
3. Welding processes and procedures are significantly different between the two countries. Japan almost exclusively employs gas-shielded metal arc welding with solid wires, whereas self-shielded flux-cored welding is commonly used in the U.S. Japanese welding is often conducted in the shop as shown in Figure C-4(a), whereas the critical welding of beams to columns in the U.S. is commonly done in the field.
4. Connection details are also different. Japan construction typically uses square tube (box) columns, whereas wide-flange columns are usually employed in the U.S. This difference is accompanied by many differences in local connection details, such as through-diaphragm connections in Japan versus through-column connections in the U.S., and welded web to column joints in Japan versus bolted web to column in pre-Northridge connections in the U.S.
5. The redundancy of the moment frame system is not the same in the two countries. All beam-to-column connections are rigidly connected in Japan, whereas in the U.S. rigid connections are commonly assigned only to selected locations. In addition to the degree of redundancy, this difference affects member sizes, the importance of gravity loads relative to seismic loads, and the stress condition (bi- versus uni-directional bending) in columns.

C.8 Partial Summary of Post-Kobe Japanese Research

Research Efforts

As a natural consequence of the damage disclosed in Kobe by the Kobe earthquake, major research and development programs have been undertaken in Japan. Japanese post-Kobe steel research efforts aim at (1) reevaluation and upgrading of plastic rotation capacity of welded steel connections and (2) characterization of plastic rotation demanded of these connections. The latter effort is closely associated with a revision of the Japan's Building Law. The new law passed the Japanese parliament in July 1998, and a detailed design code that supports the implementation of the revised law will become available in the summer of 2000.

The complete body of the research leading to these code changes, conducted in universities and by government and industry organizations, is too extensive to summarize herein. A partial description of large coordinated research/development efforts conducted after the Kobe earthquake follows. The Japanese Ministry of Education provided a four-year grant-in-aid (1996-1999) for studying urban disaster mitigation measures. The principal investigator of this project is Prof. Kenzo Toki of Kyoto University. This project is broad based, ranging from ground motion research to studies of societal impacts and risk management. Part of this project deals with steel structures. The Japanese Ministry of Construction (MOC) undertook a three-year comprehensive project (1996-1998) for improving Japanese steel construction, in which issues related to materials and welding, beam-to-column connections, and plastic rotation demands were investigated. Prof. Koichi Takanashi of Chiba University chaired the oversight committee of this project. Three volumes of the project report were released in the spring of 1999 (MOC 1999), and efforts still continue to develop guidelines for design and fabrication of steel moment frames. The Steel Committee of the Kinki Branch of the Architectural Institute of Japan conducted a two-year study (1996-1997) on welded beam-to-column connections. The project leader was Prof. Kazuo Inoue of Osaka University. In that study, 86 full-scale beam-column sub-assemblages were tested considering various parameters including connection details (weld access holes, run-off tabs, etc.), welding procedures, rate of loading, and temperature (AIJ 1997b). The Japan Society of Steel Construction (JSSC) conducted a two-year study (1995-1996), lead by Prof. Ben Kato of Toyo University, on improvement of welded connections, and published a guideline for design of steel moment frames (JSSC 1997). The Japan Welding Engineering Society (JWES) has been conducting a comprehensive research project on both demand and capacity issues for steel moment frames and their welded connections. Prof. Koichi Takanashi of Chiba University heads the project. In 1996, they released an interim report consisting of three volumes (JWES 1997). From 1995 to 1997, a research group lead by Prof. Hiroshi Akiyama of University of Tokyo conducted a series of dynamic loading tests on full-scale steel connections using a large shaking table (15 m by 15 m). The results have been published in several professional journals (for example, see Akiyama et al. 1998). A book that completely describes this project is planned. In 1997, the Science and Technology Agency (STA) started constructing a larger multi-axis shaking table having a dimension of 20 m by 15 m. Completion of the table and associated facilities is expected in the year of 2005, and various structures and structural components are planned for testing.

Code Changes

In response to the urgent need to upgrade design and fabrication practices for steel building structures, changes have already been made in a few Japanese design codes and guidelines.

A steel fabrication specification, called "JASS-6," published by the Architectural Institute of Japan (AIJ 1996), was revised in 1996. It contains new recommendations with respect to the shapes and sizes of weld access holes. Many of the newly recommended details utilize smaller hole sizes so that stress and strain concentrations would be mitigated at the toe of the weld access hole. Whether or not backing bars and run-off tabs should be removed has remained a subject of continuing debate in Japan, but the revised specification does not require their removal.

The Japan seismic design code is reviewed regularly, and minor revisions are made every few years. The last revision was made in 1997 (BCJ 1997), in which many sections related to steel buildings were amended to reflect the damage observed in the Kobe earthquake. Notable changes include among other items: (1) introduction of SN steels, (2) new design procedures for cold-formed steel tubes, (3) description of required material properties, and (4) detailed design procedures for column bases.

Japan's Building Law was revised in July 1998, and the Ministry of Construction is undertaking efforts to establish a detailed design code that supports the implementation of the revised law. In the new code, deformation demand and capacity are to be considered more explicitly. The code is to be enforced beginning in the summer of 2000.

Differences in Post-Earthquake Actions in U.S. and Japan

Damage to steel buildings in Northridge and Kobe was believed to have occurred because of a mixture of various sources related to design, materials, welding, connection details, and structural systems. This understanding is shared between the U.S. and Japan. However, solutions being provided after few years of postearthquake efforts appear to be significantly different in many aspects between the two countries. Some examples of differences, particularly related to beam-to-column connections, are summarized below.

Materials

Use of materials with larger ductility can be a solution toward higher seismic performance of steel buildings. Japan developed a new type of steels having a good margin between the yield and ultimate stresses, a smaller variation in these specified stresses, and larger fracture toughness. Use of the new steels is still optional at the present time, but their use has been increasing significantly. The U.S. also introduced a new steel grade and revised specifications for testing of materials to be used in seismic details. It appears that the utilization of special high toughness steels is gaining acceptance faster in Japan than in the U.S.

Welding

Fractures at weld metals were very serious in the U.S., and use of different electrodes having a larger toughness and controlled deposition rate is now mandatory. In Japan, fractures at weld metals were also disclosed in many instances, and welding with stringer bead placement to avoid too large heat input is strongly recommended. Efforts to develop tougher electrodes are also underway in Japan (for example, see JWES 1996). In general, however, the U.S. is more explicit as to the changes in welding and inspection practices.

Connection Details

Regarding connection details, the U.S. has pursued three courses: moving the plastic hinge away from the beam end, improving the local details and *in situ* material properties for conventional unreinforced connections, and substitution of welded connections by bolted connections. Many believe that moving the plastic hinge region is the most secure way to improve the ductility capacity of beam-to-column connections. Many new details have been developed along this line, such as strengthening of beam ends by cover plates, haunches, ribs, etc. or trimming beam flanges at a location away from the column face (named the Reduced Beam Section (RBS) connection). Such strengthening is considered as a possible solution also in Japan, but the general sentiment is that sufficient ductility capacity can be ensured by modifying connection details combined with good welding. Many efforts have been made to modify details by changing the size and shape of weld access holes, etc (Figure C-17). After five years of extensive studies, it has been felt that connection details without any weld access holes (shown in Figure C-17(c)) can ensure the most ductile performance among the various post-Kobe connection details considered.

Of immediate interest is whether a U.S.-style RBS connection or a Japanese-style connection without weld access holes (designated the no-hole connection) has larger deformation capacity. To provide some quantitative information on this issue, an experimental study was conducted as part of a U.S.-Japan Cooperative Research Project on Urban Disaster Mitigation (Suita et al. 1999). In the tests, all conditions including material properties, sectional properties, fabricator, welder, and loading history were identical, except for the connection details as shown in Figure C-18. The design procedure proposed by Engelhardt (1999) was adopted for trimming the beam flanges in the RBS connection. The no-hole connection is one of the recommended connections adopted in the AIJ's steel fabrication specification (AIJ 1996). Examples of test results in terms of the beam end moment versus rotation relationship are shown in Figure C-19. In both the no-hole and RBS connections (Figure C-19(a), (b)), no early fracture occurred at or in the vicinity of welds, and no strength deterioration was observed up to the plastic rotation of about 0.03 to 0.04 radians. The strength reduction during cycles with larger rotation amplitudes was gradual, caused primarily by the progress of local buckling to beam flanges. Ductility capacity of the two connections was judged to be nearly the same in this test program. For comparison purposes, an additional specimen was fabricated with the pre-Kobe conventional detail having the then standard weld access holes (Figure C-17(a)). The specimen failed in fracture initiated from the toe of the weld access hole, with the rotation capacity significantly smaller than the specimens having other two connections (Figure C-19(c)).

Structural Systems

As to the structural system employed, it is not likely that Japan will abandon square tube (box) columns and switch to wide flange columns at least for the foreseeable future. Similarly, the practice of using rigid connections for all beam-to-column connections will also likely remain unchanged. By the same token, the U.S. practices of using wide-flange shapes for columns and providing moment resisting connections in a small portion of the structure will likely remain unchanged as well.

C.9 Conclusions

This section has provided an overview of damage to steel building structures observed in the 1995 Hyogoken-Nanbu (Kobe) earthquake, and postearthquake activities being conducted in Japan. Although the U.S. and Japan experienced similar damage, a closer look indicates significant differences in the causes of damage. These differences most likely stem from the variation in design, detailing, fabrication, and construction practices between the two countries. It also appears that postearthquake approaches to resolve problems encountered in Kobe and Northridge also appear to be different in many aspects. After the Northridge and Kobe earthquakes, extensive technical exchange has been conducted between the U.S. and Japan. Japanese researchers and professional engineers have learned much from this exchange, and have benefited a great deal from U.S. efforts by the FEMA/SAC Steel Project and others. Nevertheless, Japanese approaches differ in many aspects from U.S. approaches, primarily as a result of differences in construction culture and philosophy as a whole. The writer wishes that U.S. readers recognize these differences when referring to Japanese literature.

Table C-1 Cross-Sections Used In Damaged Steel Buildings; (a) Columns; (b) Beams; (c) Braces

(a)

	Cross Section	Total
Column	□ (Cold-Formed)	235 (212)
	○	8
	H	409
	⊟-Shaped	70
	Built-up	55
	unknown	228

(b)

	Type of Connection	Total
Beam Splice	Weld	12
	Bolt	397
	unknown	457

(c)

	Cross Section	Total
Brace	Rod	77
	Angle	44
	Flat Plate	44
	○	42
	H	8
	□	6
	Channel	4
	unknown	227

**Table C-2 Types Of Connections Used In Damaged Buildings;
(a) Columns; (b) Beams; (c) Beam-To-Column Connections;
(d) Column Bases**

(a)

Column	Type of Connection	Total
	Weld	186
	Bolt	19
	unknown	514

(b)

Brace	Type of Connection	Total
	Weld	43
	Bolt	135
	unknown	283

(c)

Beam-to-Column Connection	Type of Connection	Total
	Field Welding	40
	Shop Welding	271
	Though Diaphragm	144
	Exterior Diaphragm	6
	Interior Diaphragm	8
	Stiffener Plate	161
	unknwon	516

(d)

Column Base	Type of Connection	Total
	Standard	270
	Concrete Encased	70
	Embedded	86
	unknown	569

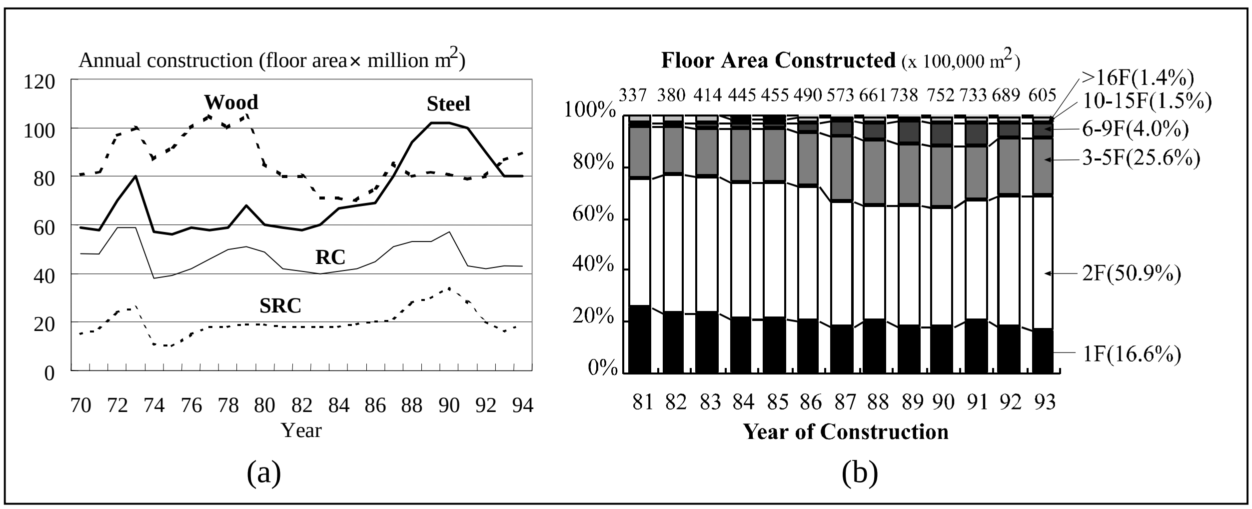
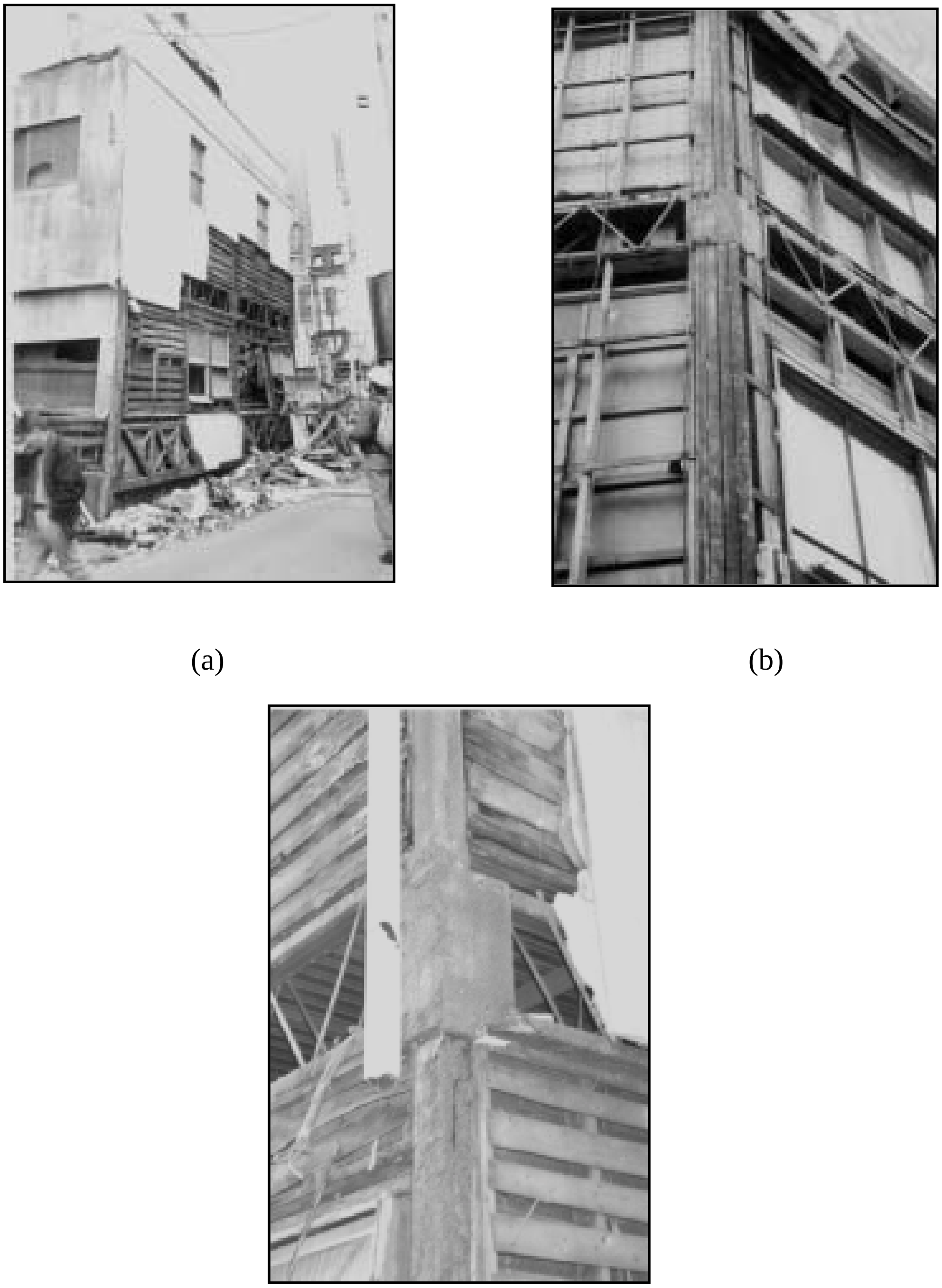


Figure C-1 Market Share For Japanese Steel Building Construction; (a) Floor Areas with Respect to Structural Material; (b) Floor Areas with Respect to Number of Stories



(a) (b)

(c)

Figure C-2 Damage to Old Steel Buildings; (a) Collapse; (b) Construction with Light-Gauged Sections; (c) Corroded Sections

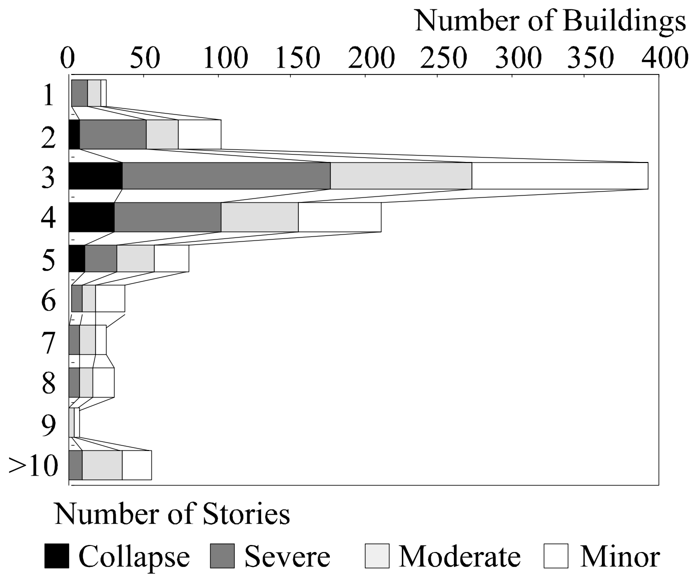


Figure C-3 Damage Level with Respect to Number of Stories

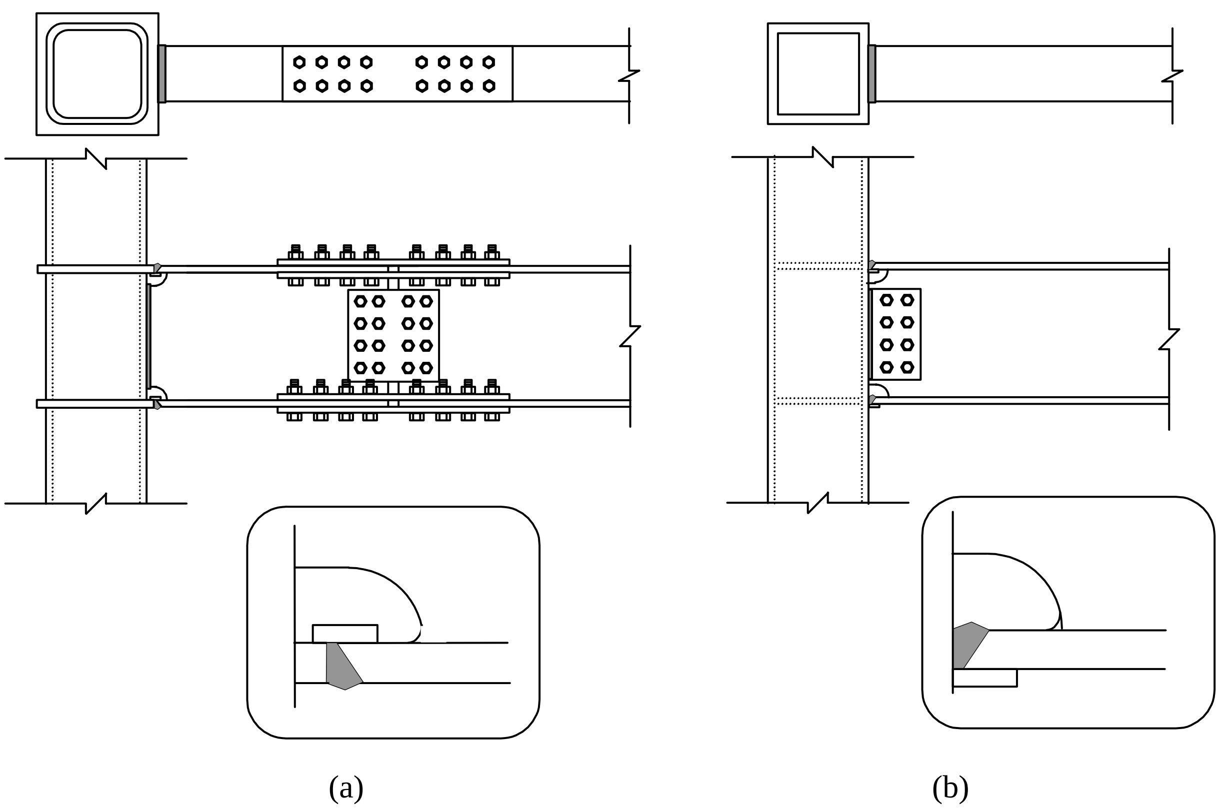


Figure C-4 Types of Beam-to-Column Connections Popular in Japan; (a) Through-Diaphragm Connection; (b) Interior-Diaphragm Connection

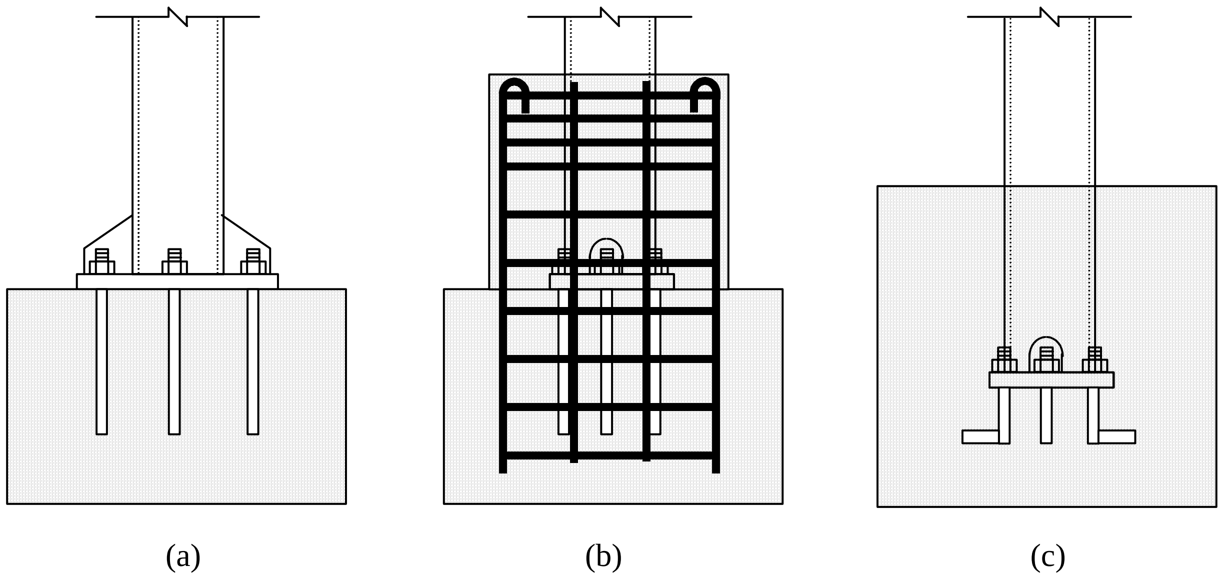


Figure C-5 Types of Column Bases Popular in Japan; (a) Base Plate Connection; (b) Concrete Encased Column Base; (c) Embedded Column Base

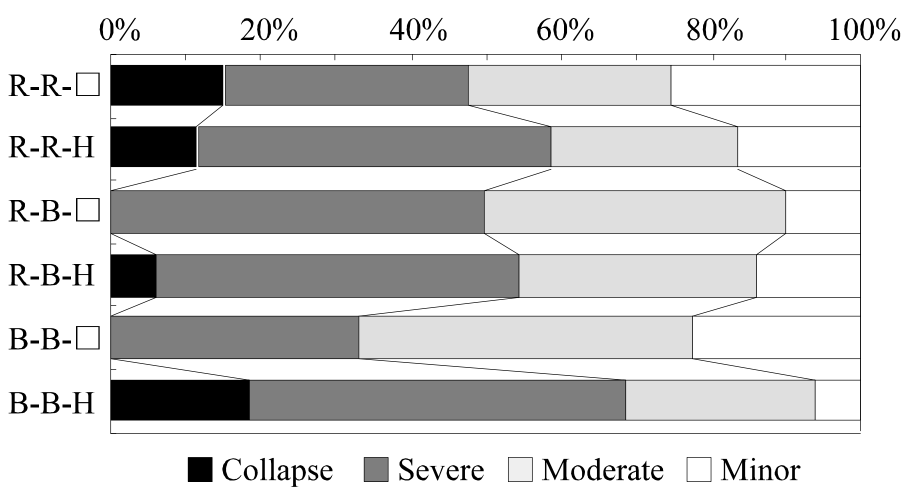


Figure C-6 Distribution of Damage Level with Respect to Structural Type

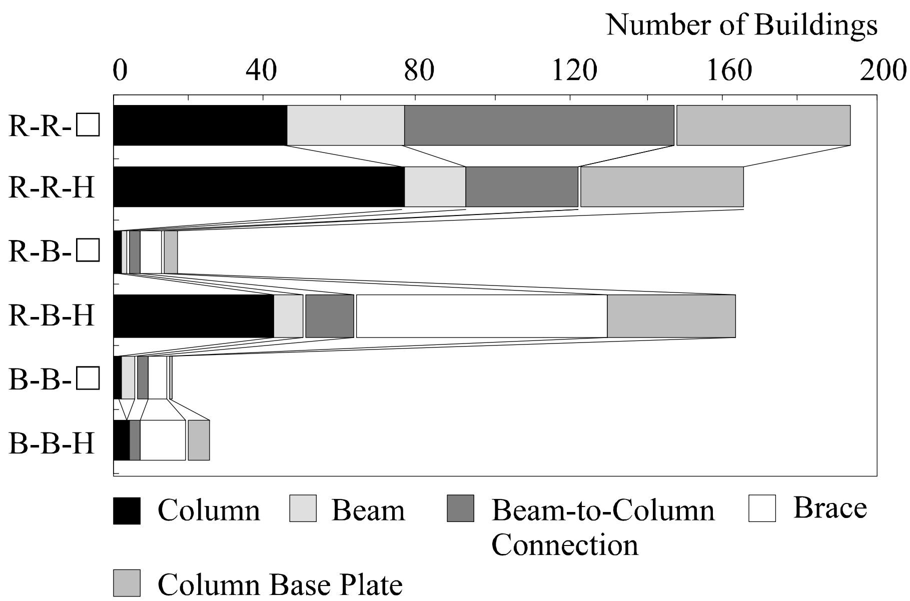


Figure C-7 Damage to Structural Members with Respect to Structural Type

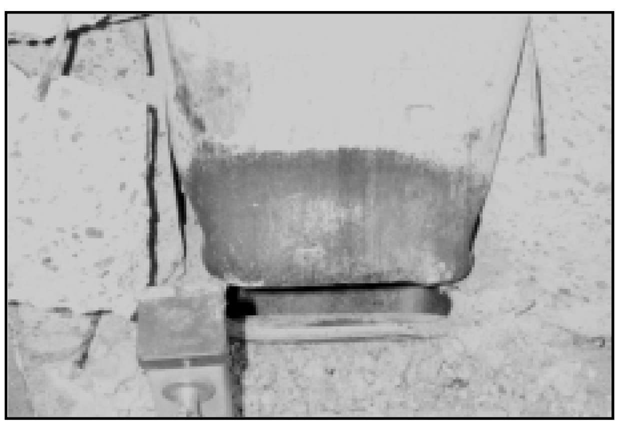
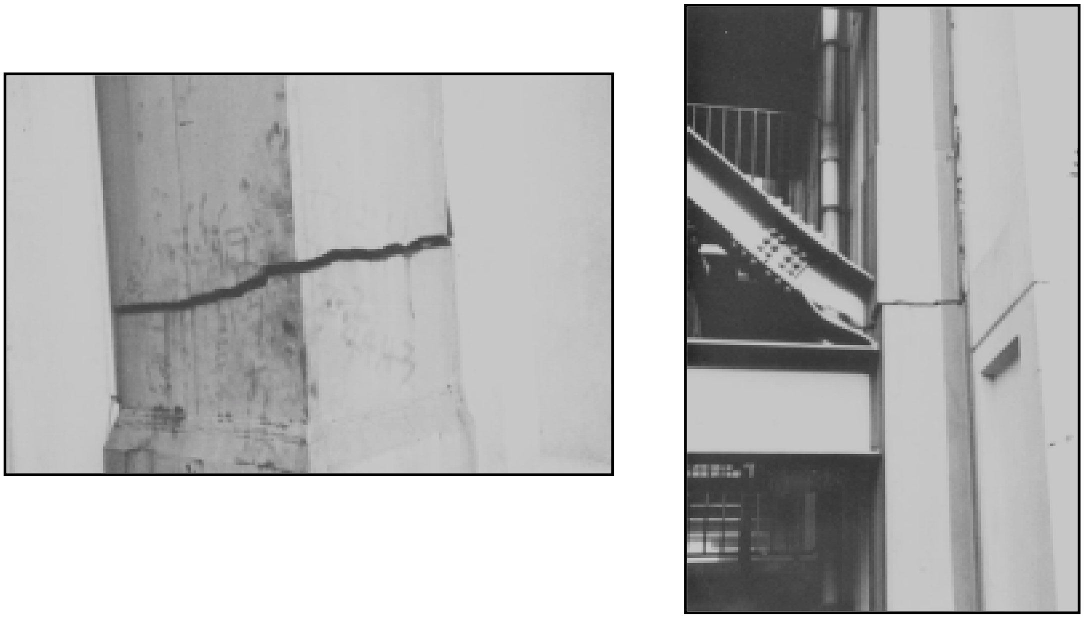
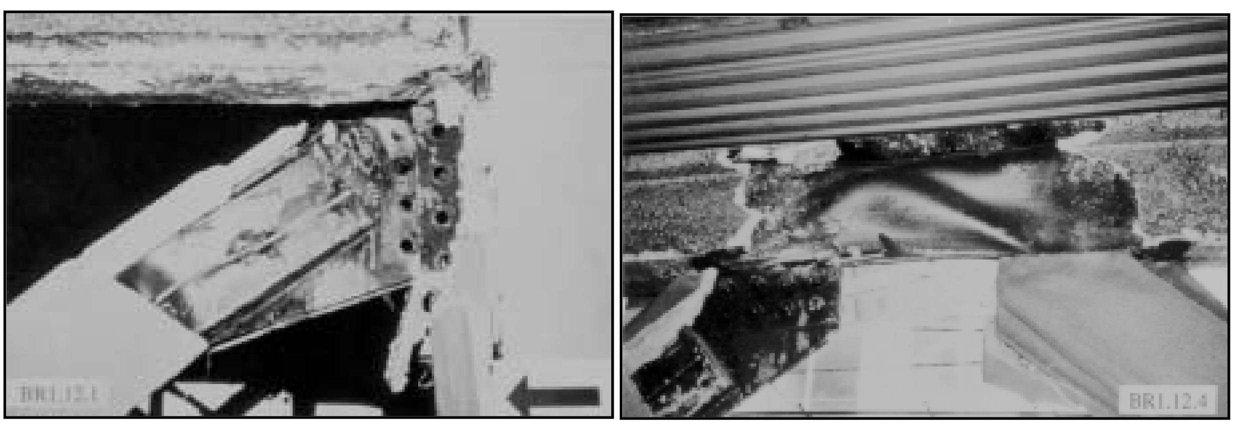


Figure C-8 Fracture at Cold-Formed Square Tube Column



(a) (b)

Figure C-9 Fracture of Square Tube Jumbo Columns; (a) Fracture at Base Metal; (b) Fracture at Brace-To-Column Connection



(a) (b)

Figure C-10 Damage To Brace Connections; (a) Breakage of Bolts; (b) Beam Web Buckling and Out-of-Plane Deformation

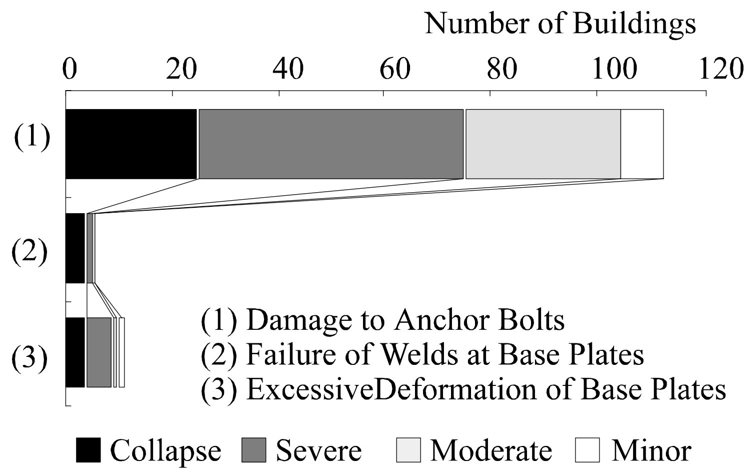
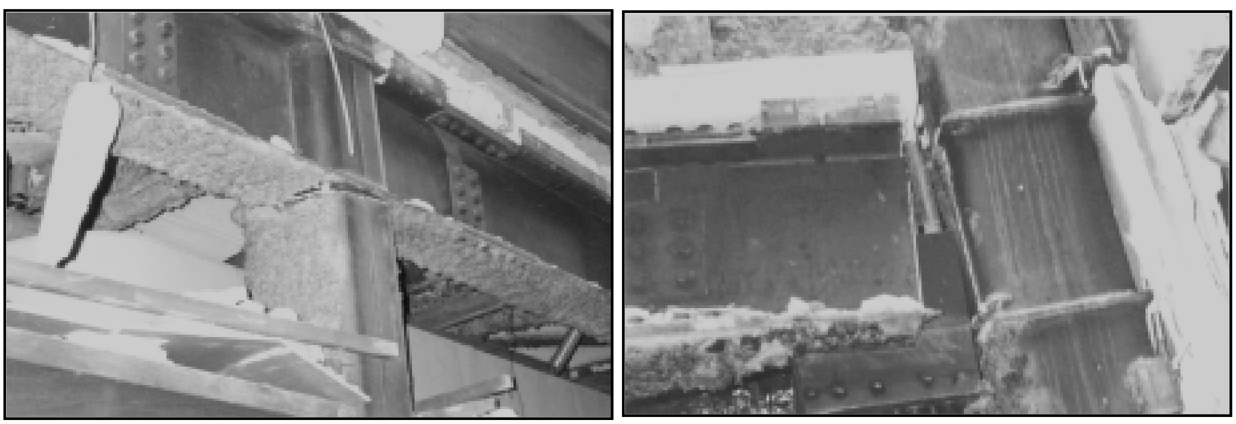


Figure C-11 Damage Location and Level of Column Base Connections

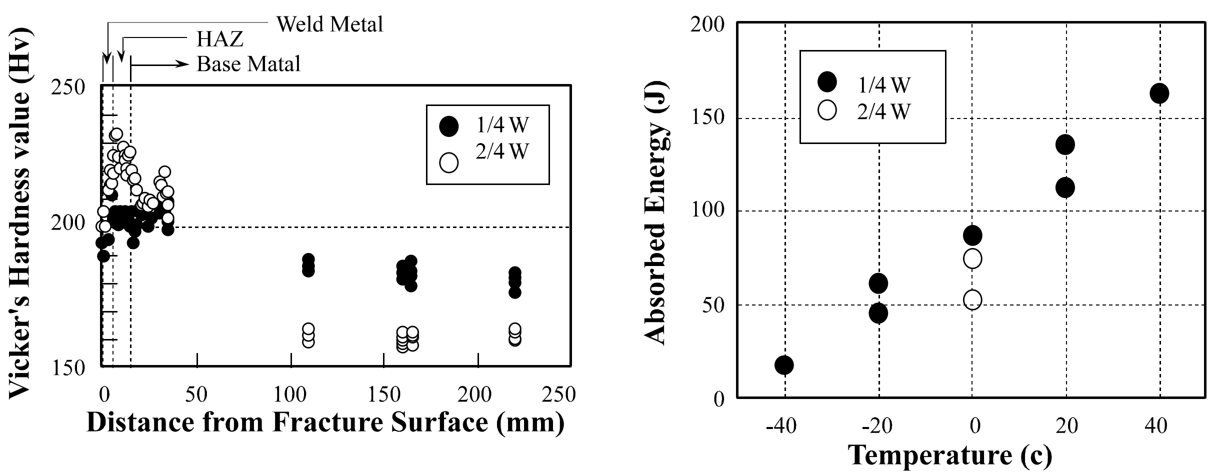


(a) (b)

Figure C-12 Fracture at Beam-To-Column Connections with Fillet Welding of Small Sizes; (a) Fracture at Column Top; (b) Fracture at Beam End

(a) (b)

Figure C-13 Fracture at Beam-To-Column Connections with Full Penetration Groove Welding; (a) Fracture at Base Metal Initiating from Toe of Weld Access Hole; (b) Fracture Involving Yielding and Local Buckling



(a) (b)

Figure C-14 Material Properties of Base Metal Near Fractured Surface; (a) Charpy V-Notch Test; (b) Hardness Test

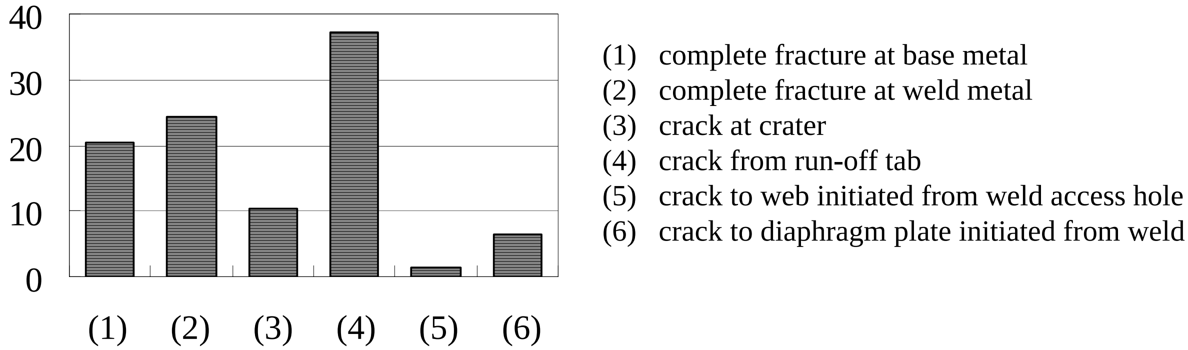


Figure C-15 Distribution of Damage To Beam-To-Column Connections with Respect to Type and Location

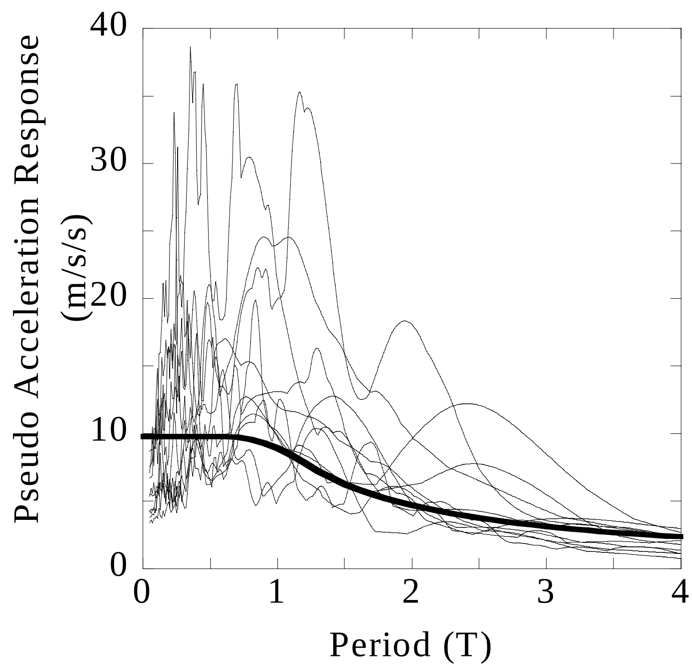
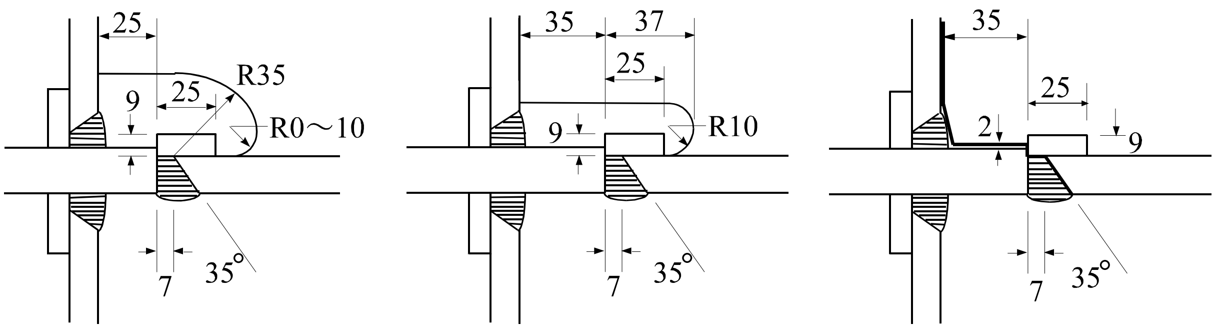
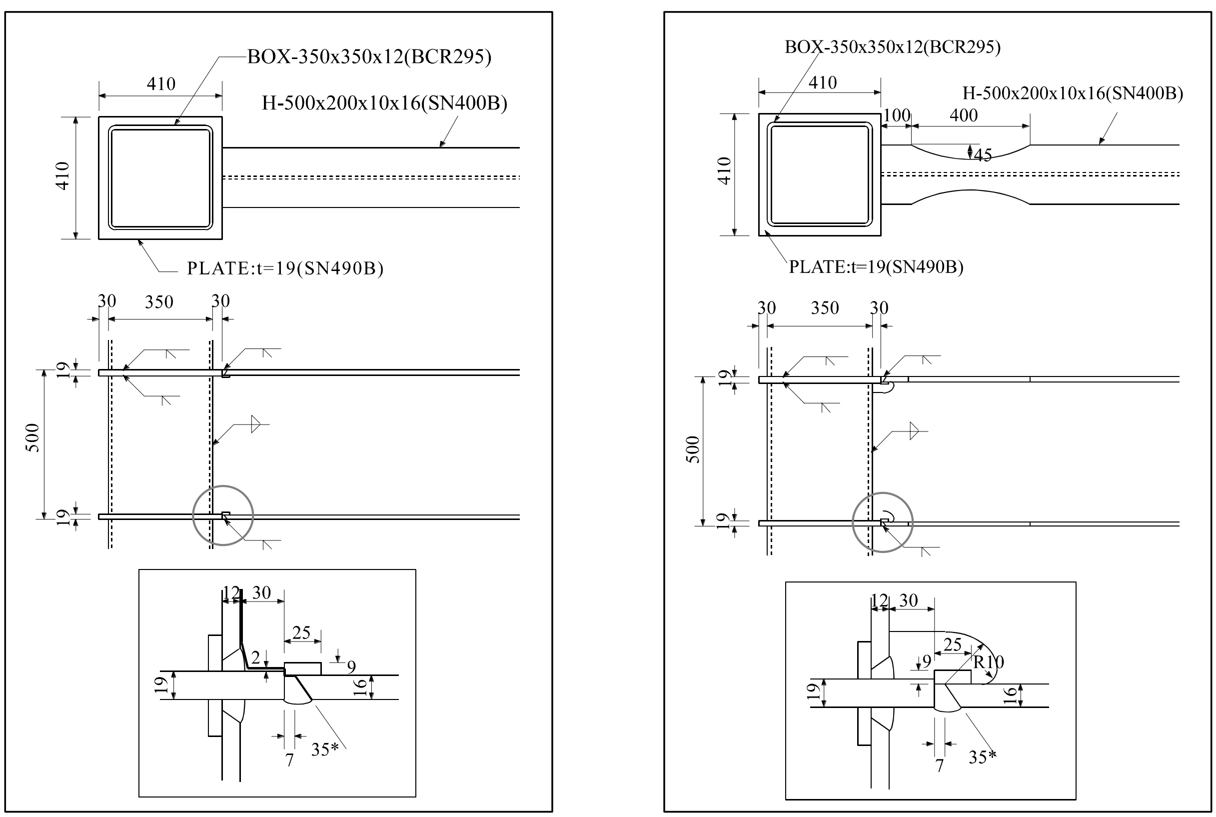


Figure C-16 Design Base Shear of Level-I Japanese Seismic Design Code and Pseudo Acceleration Response Spectra of Large Ground Motions Recorded in Kobe Earthquake



(a) (b) (c)

Figure C-17 Weld Access Hole Details Proposed after Kobe Earthquake; (a) Pre-Kobe Standard Detail; (b) Modified Detail with Smaller Hole; (c) No-Hole Detail



(a) (b)

Figure C-18 Comparison Between U.S. RBS Connection and Japanese No-Hole Connection; (a) No-Hole Connection Specimen; (b) RBS Connection Specimen

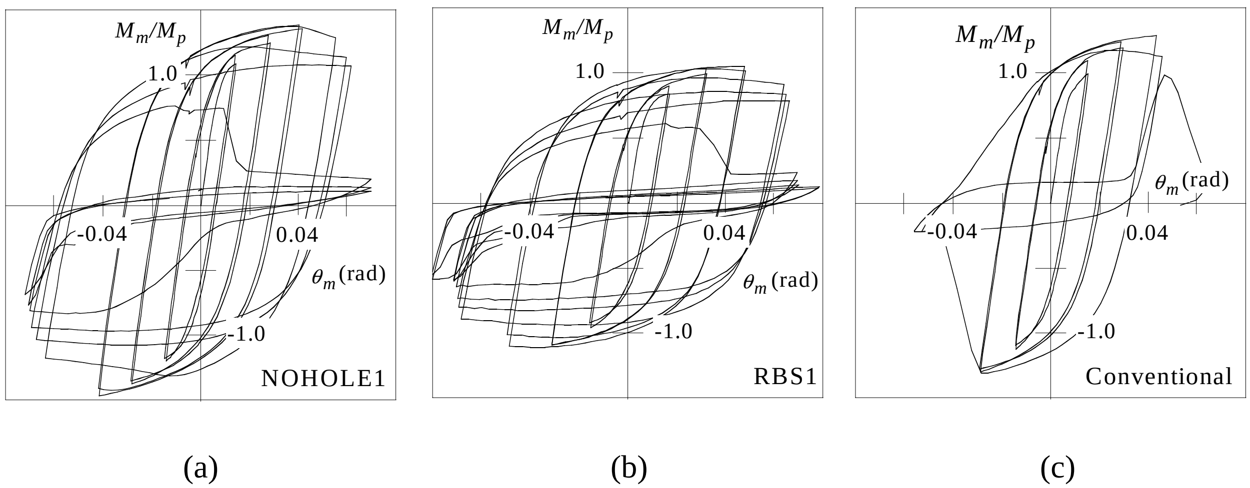


Figure C-19 Beam End Moment Versus Beam Rotation Relationships Obtained from Test; (a) No-Hole Connection; (b) RBS Connection; (c) Japanese Pre-Kobe Connection

APPENDIX D. DAMAGE TO STEEL BUILDINGS DUE TO THE SEPTEMBER 21, 1999 JI JI, TAIWAN EARTHQUAKE

(Compiled by Steve Mahin with contributions from K.C. Tsai, National Center for Earthquake Engineering Research, National Taiwan University; S.J. Chen, National Taiwan University of Science and Technology; C.M. Uang, University of California, San Diego.)

A series of damaging earthquakes struck central Taiwan starting on September 21, 1999. The largest shock was assigned a 7.6 Richter magnitude by USGS. More than 9,000 aftershocks followed the initial event. The epicenter was located in Nantou County, about 150 km south of Taipei. Many structures were heavily shaken along the 60-km long path of the fault that ruptured through or near several major cities in Taichung and Nantou Counties. Permanent fault offsets of up to nine meters were measured following the earthquake in both the vertical and horizontal directions. Prior to the earthquake, this area was thought to be one of moderate seismic hazard, and design forces used for building structures were lower than in other areas of Taiwan thought to have higher seismic risk.

Reinforced concrete is the most prevalent building material used in Taiwan for both buildings and bridges. Most commercial and residential structures are made from reinforced masonry, or reinforced concrete framing infilled with masonry or architectural concrete. Nonetheless, numerous mid- to high-rise steel structures have been constructed in the capital, Taipei (up to nearly 100 stories tall) as well as in other major cities. In Taichung, the largest city in the heavily shaken region, several high-rise dual systems and welded moment frame buildings have been constructed. These include several steel buildings in the 20 to 50-story range that were under construction at the time of the earthquake.

As indicated in Table D-1, damage was most prevalent in reinforced concrete and masonry buildings. Although damage to architectural features and localized yielding has been noted in steel structures, no significant fracture-related damage has been reported to date in welded steel moment-resisting frames.

Little information is available about the specifics of damage to steel buildings. However, some steel structures suffered localized yielding. In a few cases, some steel frames exhibited considerable ductility and lateral deformations that caused significant nonstructural damage, Figure D-1. It is expected that many steel frame buildings saw severe ground shaking in Taichung, Dali, Nantou and other large cities near the fault rupture. The only significant fracture-related damage reported was apparently detected in a couple of high-rise steel braced frames located in Taichung. Reports indicated brittle fractures occurred in some of the brace connections. Precise details of this damage are not currently available.

In Taiwan the most common grade of steel used for beams is A36 (or SM400) and A572 Gr. 50 (or SM490) for columns. Recently, higher strength steel plates (A572 Gr. 50, Gr.60 and Gr. 65, SM570) are also being used for both girders and columns.

Multistory moment frame structures generally have welded moment connections provided at all beam-to-column connections, in both framing directions, thereby giving the structures a high degree of redundancy. Columns are thus frequently built-up box sections. Beams are usually rolled sections, but built-up plate girders are used more frequently than in the U.S. In some cases, reduced beam section configurations are employed. The reduction generally differs from that used in the US profile (Chen, et al, 1997). Because of the high degree of redundancy, and the smaller seismic design forces used relative to the highest seismic hazard regions in the U.S., member sizes for comparable height buildings are smaller than would be typical in California. A typical steel frame building under construction is shown in Figure D-2.

Beam flanges are generally field welded to the columns utilizing complete joint penetration welds with E7016 using the shielded metal arc welding (SMAW) process. Interestingly, backing bars and run-off tabs are usually left in place following construction. A variety of bolted or welded connections of the beam web to shear tab/column flange is utilized. A typical connection detail of wide flange beams to box column is shown in Figure D-3. Shop welding is often done with ER70S or E70XX flux core electrode.

Low-rise steel structures up to three stories in height were used throughout the shaken area for commercial and residential construction. Welded and bolted (end plate) connections in light framing members are used. Reportedly, such structures are not designed by engineers, but rather by local, and often non-certified, fabricators and contractors. Some evidence of local working and yielding in bolted connections was observed on occasion, see Figure D-4. These light buildings generally performed well and continued to function following the earthquake, unless they were subjected to differential settlement, pounding damage or fault rupturing.

Table D-1 Statistics on Damage Due to September 21, 1999 Taiwan Earthquake, by Type of Building Material (NCREE, 1999)

Location	Reinforced Concrete/SRC	Masonry	Wooden	Steel/Light Steel	Other	Total
Nantou County	2291/9	1069	67	25 / 16	954	4431
Taichung County	1337/4	688	43	16 / 49	658	2785

Figure D-1 Permanent Lateral Displacement in Small Steel Frame

Figure D-2 Typical Taiwanese High-Rise Structure under Construction in Tiachung

Figure D-3 Welded Connection Detail of Beams to Box Column

Figure D-4 Bolted End Plate Connection in Light Steel Frame Building Showing Evidence of Local Yielding

References

"http://921., 1999, ncree.gov.tw," National Center for Research on Earthquake Engineering, Taipei, Taiwan.

Chen, S.J., Chu, J.M. and Chou, Z.L., 1997, *Dynamic Behavior of Steel Frames with Beam Flanges Shaved Around Connection,* J. Constructional Steel Research, 42 (1) 49-69.

REFERENCES, FEMA REPORTS, SAC REPORTS, AND ACRONYMS

References.

AISC, 1994, “Earthquake Safety,” *Modern Steel Construction*. (Preliminary report prepared by the Technology and Research Department of AISC), American Institute of Steel Construction, Chicago, Illinois.

AISC, 1997, *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, Illinois

Albert C. Martin & Associates, 1973, “Union Bank Square,” in Murphy, Leonard M, editor, *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Part B, National Oceanic and Atmospheric Administration, Washington, D.C.

Anderson, J. C. and Bertero, V.V., 1997, *Implications of the Landers and Big Bear Earthquakes on Earthquake Resistant Design of Structures*, EERC-97/08.

Anderson, J.C., Johnston, R.G., and Partridge, J.E., 1995, *Post Earthquake Studies of a Damaged Low Rise Office Building* (CE95-07), Department of Civil Engineering, University of Southern California.

Anderson, J.C. and Linderman, R.R., 1991, *Post Earthquake Repair of Welded Moment Connections* (CE 91-04), Department of Civil Engineering, University of Southern California.

Astaneh-Asl, A. and Liu, J., 1999, “Tests on Simple Connection Including Slab Effects (SAC Subtask 7.04),” unpublished progress report. (To be published as Liu and Astaneh-Asl, “Cyclic Tests on Simple Connections Including Effects of the Slab,” SAC/BD-00/03.)

AWS, 1995, “Structural Welding Committee Position Statement on Northridge Earthquake Welding Issues,” American Welding Society.

Bear, J., 1999, City of Santa Clarita Building Official. Personal correspondence.

Becker, R., Naeim, F., and Teal, E.J., 1988, “Seismic Design Practice for Steel Buildings,” *Steel Tips*, Structural Steel Education Council.

Beedle, L.S., 1963, “Tests of Steel Moment Connections,” *Proceedings*, Structural Engineers Association of California.

Beedle, L.S., 1965, “Reversed Loading of Steel Frames – Preliminary Tests,” *Proceedings*, Structural Engineers Association of California.

Berg, G.V., Bolt, B.A., Sozen, M.A., and Rojahn, C., 1980, *Earthquake in Romania, March 4, 1977*, National Academy Press.

Berg, G.V., and Stratta, J. L., 1964, *Anchorage and the Alaska Earthquake of March 27, 1964,*), American Institute of Steel Construction, Chicago, Illinois.

Bertero, V.V., Anderson, J.C., Krawinkler, H., 1994, “Performance of Steel Building Structures during the Northridge Earthquake,” EERC, UCB/EERC-94/09.

Bertero, V.V. and Miranda, E., 1989, “Performance of Low-Rise Buildings in Mexico City,” *Earthquake Spectra*.

Bertero, V.V., and Popov, E.P., 1965, “Effects of Large Alternating Strains of Steel Beams,” *Journal of the Structural Division*, ASCE.

Bertero, V.V., Popov, E.P., and Krawinkler, H., 1972, "Beam-Column Subassemblages Under Repeated Loading," *Journal of the Structural Division,* ASCE.

Blodgett, O.W., 1963, *Design of Weldments*, The James F. Lincoln Arc Welding Foundation.

Bonowitz, D, 1998, "The SAC Survey: Understanding and Using Steel Frame Damage Data," in *Proceedings of the NEHRP Conference and Workshop On Research on the Northridge, California Earthquake of January 17, 1994*, Volume III-B, California Universities for Research in Earthquake Engineering.

Bonowitz, D., 1999(a), "SAC Database of WSMF Connection Tests." Contractor's report submitted to the SAC Joint Venture.

Bonowitz, D., 1999(b), "The SAC Database: Understanding and Using Steel Frame Test Results," in *Proceedings of the 1999 Structures Congress,* ASCE, New Orleans.

Bonowitz, D. and Youssef, N., 1995, "SAC Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake," in SAC 95-06.

Bouwkamp, J.G., and Clough, R.W., 1965, "Dynamic Properties of a Steel Frame Building," *SEAOC Proceedings*.

Building Standards, 1994, "ICBO Board Approves Emergency Structural Design Provision," p. 26.

California Institute of Steel Construction, Southern California Division, *Why Take a Chance?* (collection of news articles), California Institute of Steel Construction.

Cary, H.B., 1970, "Flux-cored Arc Welding – Advances and Applications in the U.S.A.," Welding and Metal Fabrication.

Carydis, P. G., Tilford, N., Brandow, G., and Jirsa, J., 1982, *The Central Greece Earthquakes of February-March 1981*, National Academy Press.

Castro, J.B., 1997, "Cosmetically Repaired Welded Steel Moment Frames: A Clear Risk of Disaster," in *California Construction Law Reporter,* pp 52-62 and 67-68.

City of Burbank, Municipal Code, 1999, "Sec. 7-140. Earthquake Damage Repair of Welded Steel Moment Frame Buildings."

City of Los Angeles Building Bureau, 1994a, "Welding Procedures for Moment Ductile Frames—Both New and Repair Work," Interdepartmental Correspondence.

City of Los Angeles Building Bureau, 1994b, "Repair of Cracked Moment Connections in Steel Frame Structures and Requirements for Connections in New Buildings," Interdepartmental Correspondence.

City of Los Angeles Building Bureau, 1994c, "In New Construction the Use of Welded Connections in Buildings with Special Moment-Resisting Steel Frames (SMRF)," Interdepartmental Correspondence.

City of Los Angeles Building Bureau, 1994d, "Repair of Cracked Moment Frame Connections in Welded Steel Frame Structures and Requirements for SMRF-Welded Connections in New Buildings," Interdepartmental Correspondence.

City of Santa Monica, Municipal Code, "Chapter 8.76 Seismic Strengthening Provisions for Existing Welded Steel Moment Frame Structures," undated. Copy received December 13, 1999.

Coffman, J.L. and von Hake, C.A., 1973, editors. *Earthquake History of the United States* (Publication 41-1, Revised Edition through 1970), Washington: U.S. Department of Commerce.

Collin, A.L., 1992, "Letter to the Research Committee," Research Bulletin Board, SEAONC Research Committee.

Cornell, C.A. and Luco, N., 1999, "The Effect of Connection Fractures on Steel Moment Resisting Frame Seismic Demands and Safety: A Report on SAC Phase II Task 5.4.6," Final Draft (To be published as SAC/BD-99/03.)

Couch, M.F. and Olsson, D.A., 1965, "Weld Quality and Sonic Inspection," in *Proceedings: 34th Annual Convention*, Structural Engineers Association of California.

County of Los Angeles, 1994, "Emergency Regulations for Welded Special Moment Frame Connections and Concentrically Braced Frames (Building Code Manual No. 27.06)," County of Los Angeles Department of Public Works.

Crosby, P., 1999, personal communication.

Dames and Moore., 1998, *Survey of Damaged Steel Moment Frame Buildings*. Contractor's report to the SAC Joint Venture, Phase 2, Task 3.1.2.

Daniels, L. H. and Collin, A. L., 1972, "Design for Material Performance," *Proceedings*, Structural Engineers Association of California.

Deierlein, G., 1995, "Summary of Building Analysis Studies," in SAC 95-04, Part I.

Deppe, K., 1994, "Northridge Earthquake Overall Damage Review," in *Proceedings of the 63rd Annual Convention*, Structural Engineers Association of California.

DSA, 1995, "DSA Emergency Code Change For Steel Moment Frame Girder-to-Column Connection (IR 27-8)," California Division of the State Architect.

Durkin, M., 1995, "Inspection, Damage, and Repair of Steel Frame Buildings Following the Northridge Earthquake," in SAC 95-06.

Durkin, M., 1999, *SAC Task 3.1 Contractor's Report*, in progress.

Earthquake Spectra, May, 1985; November, 1985; February 1986; February 1988; August 1989; October 1991; April 1995 (Supplements A and B to Volume 11).

EERI, 1970, *Peru Earthquake of May 31, 1970: Preliminary Report*, Earthquake Engineering Research Institute.

EERI, 1976, "The Guatemala Earthquake of February 4, 1976 (A Preliminary Report), EERI Newsletter, v10, n2B, Earthquake Engineering Research Institute.

EERI, 1978, *Miyagi-Ken-Oki, Japan Earthquake, June 12, 1978*, Peter I Yanev, editor, Earthquake Engineering Research Institute.

EERI, 1980, *Montenegro, Yugoslavia Earthquake, April 15, 1979*, Arline Leeds, editor, Earthquake Engineering Research Institute.

EERI, 1983, *El-Asnam, Algeria Earthquake of October 10, 1980*, Earthquake Engineering Research Institute

EERI, 1996, *Public Policy and Building Safety*, Earthquake Engineering Research Institute, Earthquake Engineering Research Institute

El-Tawil, S., Vidarsson, E., Mikesell, T., and Kunnath, S., 1999, "Inelastic Behavior and Design of Steel Panel Zones," *Journal of Structural Engineering*, pp. 183-193.

Engelhardt, M.D. and Husain, A.S., 1993, "Cyclic Loading Performance of Welded Flange-Bolted Web Connections," *Journal of Structural Engineering*, ASCE.

Engelhardt, M.D., Kim, K., Sabol, T., Ho, L., Kim, H.-I., Uzarski, J., and Husain, A., 1995, "Analysis of a Six Story Steel Moment Frame Building in Santa Monica," in SAC 95-04, Part 1.

Engelhardt, M.D. and Sabol, T.A., 1994, "Testing of Welded Steel Moment Connections in Response to the Northridge Earthquake," in *Northridge Steel Update I*, American Institute of Steel Construction.

ENR, 1997a, "Suit Fuels Debate Over West's Once Popular Weld Metal."

ENR, 1997b, "Unusual Bonds to Fund Quake Reconstruction."

ENR, 1997c, "Researcher Says No One Heeded His Warnings About Connection."

ENR, 1997d, "Lincoln Settles First Lawsuit."

ENR, 1998, "Parties Settle in Lawsuit Over Cracking of Welded Connections."

EQE, 1990, *The July 16, 1990 Philippines Earthquake: A Quick Look Report*, EQE.

FEMA-267, 1995, *Interim Guidelines: Evaluation, Repair, Modification, and Design of Welded Steel Frame Moment Frame Structures*, Federal Emergency Management Agency, Washington, DC.

FEMA-267A, 1997, *Interim Guidelines Advisory No. 1 (Supplement to FEMA 267)*, Federal Emergency Management Agency, Washington, DC.

FEMA-273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington, DC.

FEMA-274, 1997, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, Federal Emergency Management Agency, Washington, DC.

FEMA-288, 1997, *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior*, Federal Emergency Management Agency, Washington, DC.

FEMA-302 and FEMA-303, 1998, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (Part 1–Provisions) and (Part 2–Commentary)*, Federal Emergency Management Agency, Washington, DC.

Ferch, R., 2000, unpublished correspondence with SAC.

Forell, N.F. and Nicoletti, J.P., 1980, *Mexico Earthquakes*, Earthquake Engineering Research Institute.

Foutch, D.A., Housner, G.W., and Jennings, P.C. 1975, *Dynamic Responses of Six Multistory Buildings During the San Fernando Earthquake* (Report No. EERL 75-02). California Institute of Technology, Earthquake Engineering Research Laboratory.

Freeman, J.R., 1932, *Earthquake Damage and Earthquake Insurance*, McGraw-Hill Book Company, New York.

Garreau, J., 1991, *Edge City*, Doubleday, New York.

Gates, W.E., 1973a, "Kajima International Building," in Murphy, Leonard M, editor, *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Part B, National Oceanic and Atmospheric Administration, Washington, D.C.

Gates, W.E., 1973b, “KB Valley Center,” in Murphy, L.M, editor, *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Part B, National Oceanic and Atmospheric Administration, Washington, D.C.

Gates, W.E. and Morden, M., 1995, “Lessons From Inspection, Evaluation, Repair and Construction of Welded Steel Moment Frames Following the Northridge Earthquake,” in SAC 95-06.

Goel, S.C., 1968, “Response of Multistory Steel Frames to Earthquake Forces,” in *Steel Research for Construction* (Bulletin No. 12), American Iron and Steel Institute.

Goltz, J.D. and Weinberg, P., 1998, “The Impact of Earthquakes on Welded Steel Moment Frame Buildings: Experience in Past Earthquakes,” draft report for the SAC Steel Project. (To be published as Weinberg and Goltz, SAC/BD-99/11.)

Graham, J.D., Sherbourne, A. N., and Khabbaz, R. N., 1959, *Welded Interior Beam-to-Column Connections*, American Institute of Steel Construction, Inc., New York.

H. J. Degenkolb Associates, Engineers, *The Whittier Narrows Earthquake, October 1, 1987*, H. J. Degenkolb Associates, Engineers, not dated.

Habibullah, A., 1984, “Personal Computers and the Structural Engineer,” *Proceedings: 1984 Convention*, Structural Engineers Association of California.

Hadley, J. M. *How Structures Withstood the Japanese Earthquake and Fire*, Portland Cement Association.

Hanson, R.D. and Degenkolb, H.J., 1969, *The Venezuela Earthquake, July 29, 1967*, American Iron and Steel Institute.

Hart, G.C., 1973, “1901 Avenue of the Stars Building,” in Murphy, Leonard M, editor, *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Part B, National Oceanic and Atmospheric Administration, Washington, D.C.

Hart, G.C., Huang, S., Lobo, R.F., Van Winkle, M., Jain, A., 1995, “Earthquake Response of Strengthened Steel Special Moment Resisting Frames,” in SAC 95-04, Part 1.

Holmes, W.T., 1998, “The History of U.S. Seismic Code Development,” *’98 EERI Annual Meeting*, Earthquake Engineering Research Institute.

ICBO, 1949 through 1997, *Uniform Building Code*.

ICBO, 1999, *1999 City of Los Angeles Building Code*, International Conference of Building Officials and California Building Standards Commission.

Islam, M.S., Youssef, N.F.G., and Lam, S.T., 1995, *Analysis of Two Steel Moment-Resisting Frame Buildings Damaged During the Northridge Earthquake*. Draft report to NIST.

J. H. Wiggins Company, 1971, “Arco Plaza Earthquake Risk Evaluation,” Unpublished report prepared for Arthur Young and Company.

Jacobsen, L.S. and Ayre, R.S., 1938, “Experimentally Determined Dynamic Shears in a Sixteen-Story Model,” *Bulletin of the Seismological Society of America*, V28.4.

The James F. Lincoln Arc Welding Foundation, 1997, *Fabricators’ and Erectors’ Guide to Welded Steel Construction*, the James F. Lincoln Arc Welding Foundation.

Jenks, K.B. and Ritts, G.J., 1997, “Welded Steel Moment Frames: In an Earthquake, We’d Rather Be in a Steel-Framed Building,” in *California Construction Law Reporter*, pp 62-66.

Jennings, P.C., 1971, editor. *Engineering Features of the San Fernando Earthquake of February 9, 1971* (EERL 71-02), Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena.

John A. Blume & Associates, 1973, “Bunker Hill Tower,” in Murphy, L.M., editor, *San Fernando, California, Earthquake of February 9, 1971*, Volume I, Part A, National Oceanic and Atmospheric Administration Washington, D.C.

Johnson, M.Q., “Evaluation of the Effect of Welding Procedure on the Mechanical Properties of FCAW-S and SMAW Weld Metal Used in the Construction of Seismic Moment Frames,” in-progress research to be published by SAC as SAC/BD-00/12.

Kariotis, J. and Eimani, T.J., 1995, “Analysis of a 16-Story Steel Frame Building at Site 5 for the Northridge Earthquake,” in SAC 95-04, Part 2.

Kaufmann, E.J., Fisher, J.W., Di Julio, R.M. Jr., and Gross, J.L., 1997, *Failure Analysis of Welded Steel Moment Frames Damaged in the Northridge Earthquake* (NISTIR 5944), National Institute of Science and Technology.

Kellam, H.S. “Recommended Lateral Force Requirements By SEAOC, 1966, Part I—Background of Paragraph (J),” *35th Annual Convention Proceedings*, Structural Engineers Association of California.

Krawinkler, H., 1985, “Tentative Lateral Force Requirements Background to Selected Steel Provisions,” *Proceedings: 54th Annual Convention*, Structural Engineers Association of California.

Krawinkler, H., 1997, “Systems Behavior of Structural Steel Frames Subjected to Earthquake Ground Motion,” in FEMA-288.

Krawinkler, H., 2000, *State of the Art Report: Systems Performance of Moment-Resisting Steel Frames Subject to Earthquake Ground Shaking* (FEMA-355C).

Krawinkler, H., Alali, A., Thiel, C.C., Dunlea, J.M., 1995, “Analysis of a Damaged Four-Story Building and an Undamaged Two-Story Building,” in SAC 95-04, Part 1.

LACOTAP (Los Angeles County Technical Advisory Panel), 1997, *SMRF Bulletin No. 3, Steel Moment Resisting Frame (SMRF) Connection Systems*.

Layne, H.M. (moderator), et al., 1963, “Panel Discussion: Section J of the SEA Code—Should It Be Retained, Modified, or Eliminated?” *Proceedings*, Structural Engineers Association of California.

Lee, K.-H., Stojadinovic, B., Goel, S., Margarian, A., Choi, J., Wongkaew, A., Reyher, B., and Lee, D.-Y., “Parametric Tests on Unreinforced Connections.” Contractors’ report to the SAC Joint Venture, to be published as SAC/BD-00/01.

Los Angeles Department of Building & Safety, “Status of Buildings Ordered to Comply with Ord. 170406 as of 5/4/98,” unpublished.

Mahin, S.A., Malley, J.O., Hamburger, R.O., 1996, “Phase 2 of the SAC Steel Project,” *Proceedings: 65th Annual Convention,* Structural Engineers Association of California.

Maison, B.F. and Bonowitz, D., 1999, “How Safe Are Pre-Northridge WSMFs? A Case Study of the SAC Los Angeles Nine-Story Building,” in *Earthquake Spectra* 15.

Maison, B.F. and Bonowitz, D., 2000, “Simplified Loss Estimation For Pre-Northridge WSMF Buildings,” in-progress report to SAC.

Maison, B.F. and Kasai, K., 1997, “Analysis of Northridge Damaged Thirteen-Story WSMF Building,” in *Earthquake Spectra* 13.

McDonald, G., 1999, City of Simi Valley Building Official. Personal correspondence.

McGuire, W., 1988, "Introduction," in *Journal of Constructional Steel Research*, volume 10 (special issue on steel beam-to-column building connections), Elsevier Applied Science.

Melnick, S., 1991, "Criticizing Steel," *Modern Steel Construction*.

Mendizabal, M., 2000, City of Santa Monica Building Official. Personal correspondence.

Mendoza, B., 1999, City of San Fernando Building Official. Personal correspondence.

Modern Steel Construction, 1994, "Localized Steel Damage," *Modern Steel Construction*.

Moon, S., 1999, City of Beverly Hills Building Official. Personal correspondence.

Naeim, F., DiJulio, R., Benuska, K., Reinhorn, A.M., and Chen, L., 1995, "Evaluation of Seismic Performance of an 11-Story Steel Moment Frame Building During the 1994 Northridge Earthquake," in SAC 95-04, Part 2.

Naeim, F., Lobo, R., Skliros, K., and Sgambelluri, M., 1999, *Seismic Performance of Four Instrumented Moment Resisting Steel Structures During the January 17, 1994 Northridge, California Earthquake*. Final Draft report to California Division of Mines and Geology, SMIP.

Osteraas, J., and Krawinkler, H., 1989, "The Mexico Earthquake of September 19, 1985: Behavior of Steel Buildings," *Earthquake Spectra*, v 5.1, Earthquake Engineering Research Institute.

Paret, T.F., 1999, "Studies Directed Towards Explaining the W1 Issue; Clarifying the Extent of Northridge Induced Weld Fracturing; and Examining the Related Issue of UT Reliability," Contractor's report to the SAC Joint Venture. (To be published as SAC/BD-99/10.)

Paret, T.F., *Nine Studies Aimed at Explaining the W1 Issue and the Related Issue of UT Inspection Reliability*, Contractor's report to the SAC Joint Venture, undated.

Paret, T. and Sasaki, K.K., 1995, "Analysis of a 17-Story Steel Frame Building Damaged by the Northridge Earthquake," in SAC 95-04, Part 2.

Phipps, M.T., 1998, "Task 3.1, Objective 5: Investigation of Damage to WSMF by Earthquakes other than Northridge," Contractor's report to the SAC Joint Venture, (to be published as SAC/BD-99/09.)

Popov, E.P., 1987, "Seismic Moment Connections for MRF's," *Journal of Constructional Steel Research* (U.K.). Republished in *Journal of Constructional Steel Research*, volume 10 (special issue on steel beam-to-column building connections), Elsevier Applied Science, 1988.

Popov, E.P., Amin, N.R., Louie, J.J.C., and Stephen, R.M., 1985, "Cyclic Behavior of Large Beam-Column Assemblies," *Earthquake Spectra*, Earthquake Engineering Research Institute.

Popov, E.P. and Bertero, V.V., 1970, "Research on Seismic Behavior of High-Rise Moment-Resisting Steel Frames," *Proceedings*, Structural Engineers Association of California.

Popov, E.P. and Bertero, V.V., 1973, "Cyclic Loading of Steel Beams and Connections," *Journal of the Structural Division*, ASCE.

Popov, E.P. and Franklin, H.A., 1965, "Steel Beam-to-Column Connections," *SEAOC Proceedings*.

Popov, E.P. and Pinkney, A.M., 1969, "Cyclic Yield Reversal in Steel Building Connections," *Journal of the Structural Division*, ASCE.

Popov, E.P. and Stephen, R.M., 1972, "Cyclic Loading of Full-Size Steel Connections," *Steel Research for Construction* (Bulletin No. 21), AISI.

Popov, E.P. and Tsai, K.C., 1987, "Performance of Large Seismic Steel Moment Connections Under Cyclic Loads," *SEAONC Proceedings, 56th Annual Convention*.

Popov, E.P., Tsai, K.C., and Engelhardt, M.D., 1988, "On Seismic Steel Joints and Connections," in *Proceedings: 57th Annual Convention*, Structural Engineers Association of California.

Popov, E.P., Yang, T.-S., and Grigorian, C.E., 1993, "New Directions in Structural Seismic Designs," *Earthquake Spectra*.

Preece, F.R., 1981, *Structural Steel in the 80s—Materials, Fastening and Testing*. The Steel Committee of California.

Preece, F.R. and Collin, A.L., 1991, *Structural Steel Construction in the '90s*. Structural Steel Education Council.

Procedure Handbook of Arc Welding, The, 1973, Twelfth Edition, The Lincoln Electric Company, Cleveland, Ohio.

Putkey, J.J., 1993, "Common Steel Erection Problems and Suggested Solutions," in *Steel Tips*, Structural Steel Education Council.

Reynolds, R.E., editor. *Landers: Earthquakes and Aftershocks*, Winter 1993 issue of *San Bernardino County Museum Association Quarterly*.

Roeder, C.W. and Foutch, D.A., 1996, "Experimental Results for Seismic Resistant Steel Moment Frame Connections," *Journal of Structural Engineering*.

SAC, 1998, "Observations of the Behavior of Steel Frame Structures in Historic Earthquakes," draft report to the SAC Joint Venture Partnership.

SAC 95-01, 1995, *Advisory No. 3*, SAC Joint Venture Partnership.

SAC 95-04, 1995, *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*, SAC Joint Venture Partnership.

SAC 95-06, 1995, *Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, SAC Joint Venture Partnership.

SAC 95-07, 1995, *Case Studies of Steel Moment Frame Building Performance in the Northridge Earthquake of January 17, 1994*, SAC Joint Venture Partnership.

SAC Steel Project, 1996, "Notice to Building Officials and Design Professionals in the San Francisco Bay Area."

Schneider, S.P., Roeder, C.W., 1993, Carpenter, James E., "Seismic Behavior of Moment Resisting Steel Frames: Experimental Study," *Journal of Structural Engineering*, ASCE.

SEAOC, 1959; 1968 (with revisions); 1974 (with 1975 and 1980 revisions); 1988; 1990; 1996, *Recommended Lateral Force Requirements and Commentary* (Blue Book), Structural Engineers Association of California.

SEAOC Plan Review, 1996, "SEAOC Represented in Legislative Process."

SEAONC, 1998, *SMRF Connection Qualification Commentary*, Structural Engineers Association of Northern California Seismology Committee.

Seligson, H. and Eguchi, R., 1999, “Normalizing Database of SMRF Structures within Los Angeles County for Use in SAC Damage Model Development Projects,” EQE Project Number 250825.01.

Sloan, T., 2000, City of Burbank Plan Check Engineer. Personal correspondence.

Smith, D., 1995, “Inspections, Repairs Lag for Steel-Frame Buildings,” *Los Angeles Times*.

Song, J. and Ellingwood, B.R., 1999, “Seismic Reliability of Special Moment Steel Frames With Welded Connections: I,” *Journal of Structural Engineering*.

SSDA, 1996, “Slotted Beam-to-Column Connection Designs,” Seismic Structural Design Associates, Inc.

Steel Committee of California, 1979 (revised); 1983; 1986, *Steel Connections/Details and Relative Costs*.

Steinbrugge, K.V. and Moran, D.F., 1954, “An Engineering Study of the Southern California Earthquake of July 21, 1952, and Its Aftershocks,” *Bulletin of the Seismological Society of America*.

Steinbrugge, K.V., Schader, E.E., Bigglestone, H.C., Weers, C.A., 1971, “San Fernando Earthquake, February 9, 1971,” Pacific Fire Rating Bureau.

Strand, D.R., 1984, “Code Development Between 1927 and 1980,” *Proceedings*, Structural Engineers Association of California.

Stratta, J.L., Escalante, L.E., Krinitzsky, Ellis L., and Morelli, U., 1981, *Earthquake in Campania-Basilicata, Italy, November 23, 1980*, National Academy Press.

Stratta, J.L. and Wyllie, L.A., 1979, *Friuli, Italy Earthquakes of 1976*, Earthquake Engineering Research Institute.

Thornton, W.A., 1992, “Designing for Cost Efficient Fabrication,” in *Steel Tips*, Structural Steel Education Council.

Tide, R., 2000, unpublished correspondence with SAC, January 28, 2000, and personal correspondence.

Tierney, K.J., 1985, *Report on the Coalinga Earthquake of May 2, 1983*. Seismic Safety Commission.

Tobriner, S., 1984, “The History of Building Codes to the 1920’s,” *Proceedings*, Structural Engineers Association of California.

Tom, S., 2000, City of Glendale Building Official. Personal correspondence.

Tsai, K.C. and Popov, E.P., 1988, *Steel Beam-Column Joints in Seismic Moment Resisting Frames* (UCB/EERC-88/19), Earthquake Engineering Research Center, Richmond, California.

Uang, C.M., Yu, Q.S., Sadre, A., Bonowitz, D., Youssef, N., 1995, *Performance of a 13-Story Steel Moment-Resisting Frame Building Damaged in the 1994 Northridge Earthquake*, SSRP-95/04, University of California, San Diego.

United States Geological Survey (USGS), 1907, *The San Francisco Earthquake and Fire of April 18, 1906*, Washington: Government Printing Office.

United States Steel Corporation, circa 1980, *Beam-to-Column Flange Connections—Restrained Members: A Design Aid*, United States Steel Corporation.

Wilson, E.L., 1984, "Selection of Microcomputer Hardware for Structural Analysis and Design," *Proceedings: 1984 Convention,* Structural Engineers Association of California.

Wood, F.J., editor, 1967, *The Prince William Sound, Alaska, Earthquake of 1964 and Aftershocks* (Coast and Geodetic Survey Publication 10-3), Washington D.C.: United States Government Printing Office.

Yanev, P.I., Gillengerten, J.D., and Hamburger, R.O., 1991, *The Performance of Steel Buildings in Past Earthquakes*, The American Iron and Steel Institute.

Youssef, N., Bonowitz, D., and Gross, J., 1995, *A Survey of Steel Moment-Resisting Frame Buildings Affected by the 1994 Northridge Earthquake* (NISTIR 5625), National Institute of Science and Technology.

Zeevaert, L., 1962, "The Development of Shear Displacement Meters and Accelerometers to Measure Earthquake Forces in Buildings," in *Proceedings: 31st Annual Convention*, Structural Engineers Association of California.

Zhang, J. and Dong, P., 2000, "Residual Stresses in Welded Moment Frames and Implications for Structural Performance," *Journal of Structural Engineering*.

Zsutty, T.C., 1989, "The 1989 Blue Book Commentary," in *Proceedings: 58th Annual Convention,* Structural Engineers Association of California.

FEMA Reports.

FEMA reports are listed by report number.

FEMA-178, 1992, *NEHRP Handbook for the Seismic Evaluation of Existing Buildings*, developed by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-267, 1995, *Interim Guidelines, Inspection, Evaluation, Repair, Upgrade and Design of Welded Moment Resisting Steel Structures*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC. Superseded by FEMA 350 to 353.

FEMA-267A, 1996, *Interim Guidelines Advisory No. 1,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC. Superseded by FEMA 350 to 353.

FEMA-267B, 1999, *Interim Guidelines Advisory No. 2,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC. Superseded by FEMA 350 to 353.

FEMA-273, 1997, *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-274, 1997, *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings*, prepared by the Applied Technology Council for the Building Seismic Safety Council, published by the Federal Emergency Management Agency, Washington, DC.

FEMA-302, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1 – Provisions,* prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-303, 1997, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 2 – Commentary*, prepared by the Building Seismic Safety Council for the Federal Emergency Management Agency, Washington, DC.

FEMA-310, 1998, *Handbook for the Seismic Evaluation of Buildings – A Prestandard*, prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, DC.

FEMA-350, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-351, 2000, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-352, 2000, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-353, 2000, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-354, 2000, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355A, 2000, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355B, 2000, *State of the Art Report on Welding and Inspection,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355C, 2000, *State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355D, 2000, *State of the Art Report on Connection Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355E, 2000, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

FEMA-355F, 2000, *State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC Joint Venture Reports.

SAC Joint Venture reports are listed by report number, except for SAC 2000a through 2000k; those entries that do not include a FEMA report number are published by the SAC Joint Venture.

SAC 94-01, 1994, *Proceedings of the Invitational Workshop on Steel Seismic Issues, Los Angeles,* September 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 94-01, 1994b, *Proceedings of the International Workshop on Steel Moment Frames, Sacramento,* December, 1994, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-01, 1995, *Steel Moment Frame Connection Advisory No. 3*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-02, 1995, *Interim Guidelines: Evaluation, Repair, Modification and Design of Welded Steel Moment Frame Structures*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267, Washington, DC.

SAC 95-03, 1995, *Characterization of Ground Motions During the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-04, 1995, *Analytical and Field Investigations of Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-05, 1995, *Parametric Analytic Investigations of Ground Motion and Structural Response, Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-06, 1995, *Technical Report: Surveys and Assessment of Damage to Buildings Affected by the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-07, 1995, *Technical Report: Case Studies of Steel Moment-Frame Building Performance in the Northridge Earthquake of January 17, 1994*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-08, 1995, *Experimental Investigations of Materials, Weldments and Nondestructive Examination Techniques*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 95-09, 1995, *Background Reports: Metallurgy, Fracture Mechanics, Welding, Moment Connections and Frame Systems Behavior*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-288, Washington, DC.

SAC 96-01, 1996, *Experimental Investigations of Beam-Column Subassemblages, Part 1 and 2,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 96-02, 1996, *Connection Test Summaries,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-289, Washington, DC.

SAC 96-03, 1997, *Interim Guidelines Advisory No. 1 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-267A, Washington, DC.

SAC 98-PG, *Update on the Seismic Safety of Steel Buildings – A Guide for Policy Makers,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Washington, DC.

SAC 99-01, 1999, *Interim Guidelines Advisory No. 2 Supplement to FEMA-267 Interim Guidelines*, prepared by the SAC Joint Venture, for the Federal Emergency Management Agency, Report No. FEMA-267B, Washington, DC.

SAC, 2000a, *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-350, Washington, DC.

SAC, 2000b, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-351, Washington, DC.

SAC, 2000c, *Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-352, Washington, DC.

SAC, 2000d, *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-353, Washington, DC.

SAC, 2000e, *A Policy Guide to Steel Moment-Frame Construction*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-354, Washington, DC.

SAC, 2000f, *State of the Art Report on Base Metals and Fracture*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355A, Washington, DC.

SAC, 2000g, *State of the Art Report on Welding and Inspection*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355B, Washington, DC.

SAC, 2000h, *State of the Art Report on Systems Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355C, Washington, DC.

SAC, 2000i, *State of the Art Report on Connection Performance*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355D, Washington, DC.

SAC, 2000j, *State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355E, Washington, DC.

SAC, 2000k, *State of the Art Report on Performance Prediction and Evaluation*, prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No. FEMA-355F, Washington, DC.

SAC/BD-96/01, *Selected Results from the SAC Phase 1 Beam-Column Connection Pre-Test Analyses*, submissions from B. Maison, K. Kasai, and R. Dexter; and A. Ingraffea and G. Deierlein.

SAC/BD-96/02, *Summary Report on SAC Phase 1 - Task 7 Experimental Studies*, by C. Roeder (a revised version of this document is published in Report No. SAC 96-01; the original is no longer available).

SAC/BD-96/03, *Selected Documents from the U.S.-Japan Workshop on Steel Fracture Issues*.

SAC/BD-96/04, *Survey of Computer Programs for the Nonlinear Analysis of Steel Moment Frame Structures*.

SAC/BD-97/01, *Through-Thickness Properties of Structural Steels*, by J. Barsom and S. Korvink.

SAC/BD-97/02, *Protocol for Fabrication, Inspection, Testing, and Documentation of Beam-Column Connection Tests and Other Experimental Specimens,* by P. Clark, K. Frank, H. Krawinkler, and R. Shaw.

SAC/BD-97/03, *Proposed Statistical and Reliability Framework for Comparing and Evaluating Predictive Models for Evaluation and Design*, by Y.-K. Wen.

SAC/BD-97/04, *Development of Ground Motion Time Histories for Phase 2 of the FEMA/SAC Steel Project*, by. P. Somerville, N. Smith, S. Punyamurthula, and J. Sun.

SAC/BD-97/05, *Finite Element Fracture Mechanics Investigation of Welded Beam-Column Connections,* by W.-M. Chi, G. Deierlein, and A. Ingraffea.

SAC/BD-98/01, *Strength and Ductility of FR Welded-Bolted Connections*, by S. El-Tawil, T. Mikesell, E. Vidarsson, and S. K. Kunnath.

SAC/BD-98/02, *Effects of Strain Hardening and Strain Aging on the K-Region of Structural Shapes*, by J. Barsom and S. Korvink

SAC/BD-98/03, *Implementation Issues for Improved Seismic Design Criteria: Report on the Social, Economic, Policy and Political Issues Workshop* by L. T. Tobin.

SAC/BD-99/01, *Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae.

SAC/BD-99/01A, *Appendix to: Parametric Study on the Effect of Ground Motion Intensity and Dynamic Characteristics on Seismic Demands in Steel Moment Resisting Frames* by G. A. MacRae.

SAC/BD-99/02, *Through-Thickness Strength and Ductility of Column Flange in Moment Connections*, by R. Dexter and M. Melendrez.

SAC/BD-99/03, *The Effects of Connection Fractures on Steel Moment Resisting Frame Seismic Demands and Safety*, by C. A. Cornell and N. Luco.

SAC/BD-99/04, *Effects of Strength/Toughness Mismatch on Structural and Fracture Behaviors in Weldments,* by P. Dong, T. Kilinski, J. Zhang, and F.W. Brust.

SAC/BD-99/05, *Assessment of the Reliability of Available NDE Methods for Welded Joint and the Development of Improved UT Procedures*, by G. Gruber and G. Light.

SAC/BD-99/06, *Prediction of Seismic Demands for SMRFs with Ductile Connections and Elements*, by A. Gupta and H. Krawinkler.

SAC/BD-99/07, *Characterization of the Material Properties of Rolled Sections,* by T. K. Jaquess and K. Frank.

SAC/BD-99/08, *Study of the Material Properties of the Web-Flange Intersection of Rolled Shapes,* by K. R. Miller and K. Frank.

SAC/BD-99/09, *Investigation of Damage to WSMF Earthquakes other than Northridge*, by M. Phipps.

SAC/BD-99/10, *Clarifying the Extent of Northridge Induced Weld Fracturing and Examining the Related Issue of UT Reliability,* by T. Paret.

SAC/BD-99/11, *The Impact of Earthquakes on Welded Steel Moment Frame Buildings: Experience in Past Earthquakes,* by P. Weinburg and J. Goltz.

SAC/BD-99/12, *Assessment of the Benefits of Implementing the New Seismic Design Criteria and Inspection Procedures,* by H. A. Seligson and R. Eguchi.

SAC/BD-99/13, *Earthquake Loss Estimation for WSMF Buildings*, by C. A. Kircher.

SAC/BD-99/14, *Simplified Loss Estimation for Pre-Northridge WSMF Buildings*, by B. F. Maison and D. Bonowitz.

SAC/BD-99/15, *Integrative Analytical Investigations on the Fracture Behavior of Welded Moment Resisting Connections*, by G. G. Deierlein and W.-M. Chi.

SAC/BD-99/16, *Seismic Performance of 3- and 9- Story Partially Restrained Moment Frame Buildings*, by B. F. Maison and K. Kasai.

SAC/BD-99/17, *Effects of Partially-Restrained Connection Stiffness and Strength on Frame Seismic Performance*, by K. Kasai, B. F. Maison, and A. Mayangarum.

SAC/BD-99/18, *Effects of Hysteretic Deterioration Characteristics on Seismic Response of Moment Resisting Steel Structures*, by F. Naeim, K. Skliros, A. M. Reinhorn, and M. V. Sivaselvan.

SAC/BD-99/19, *Cyclic Instability of Steel Moment Connections with Reduced Beam Section*, by C.-M. Uang and C.-C. Fan.

SAC/BD-99/20, *Local and Lateral-Torsion Buckling of Wide Flange Beams*, by L. Kwasniewski, B. Stojadinovic, and S. C. Goel.

SAC/BD-99/21, *Elastic Models for Predicting Building Performance*, by X. Duan and J. C. Anderson.

SAC/BD-99/22, *Reliability-Based Seismic Performance Evaluation of Steel Frame Buildings Using Nonlinear Static Analysis Methods*, by G. C. Hart and M. J. Skokan.

SAC/BD-99/23, *Failure Analysis of Welded Beam to Column Connections*, by J. M. Barsom and J. V. Pellegrino.

SAC/BD-99/24, *Weld Acceptance Criteria for Seismically-Loaded Welded Connections*, by W. Mohr.

SAC/BD-00/01, *Parametric Tests on Unreinforced Connections, Volume I – Final Report*, by K.-H. Lee, B. Stojadinovic, S. C. Goel, A. G. Margarian, J. Choi, A. Wongkaew, B. P. Reyher, and D.-Y, Lee.

SAC/BD-00/01A, *Parametric Tests on Unreinforced Connections, Volume II – Appendices*, by K.-H. Lee, B. Stojadinovic, S. C. Goel, A. G. Margarian, J. Choi, A. Wongkaew, B. P. Reyher, and D.-Y, Lee.

SAC/BD-00/02, *Parametric Tests on the Free Flange Connections*, by J. Choi, B. Stojadinovic, and S. C. Goel.

SAC/BD-00/03, *Cyclic Tests on Simple Connections Including Effects of the Slab*, by J. Liu and A. Astaneh-Asl.

SAC/BD-00/04, *Tests on Bolted Connections, Part I: Technical Report*, by J. Swanson, R. Leon, and J. Smallridge.

SAC/BD-00/04A, *Tests on Bolted Connections, Part II: Appendices*, by J. Swanson, R. Leon, and J. Smallridge.

SAC/BD-00/05, *Bolted Flange Plate Connections*, by S. P. Schneider and I. Teeraparbwong.

SAC/BD-00/06, *Round Robin Testing of Ultrasonic Testing Technicians*, by R. E. Shaw, Jr.

SAC/BD-00/07, *Dynamic Tension Tests of Simulated Welded Beam Flange Connections*, by J. M. Ricles, C. Mao, E. J. Kaufmann, L.-W. Lu, and J. W. Fisher.

SAC/BD-00/08, *Design of Steel Moment Frame Model Buildings in Los Angeles, Seattle and Boston*, by P. Clark.

SAC/BD-00/09, *Benchmarking of Analysis Programs for SMRF System Performance Studies*, by A. Gupta and H. Krawinkler.

SAC/BD-00/10, *Loading Histories for Seismic Performance Testing of SMRF Components and Assemblies*, by H. Krawinkler, A. Gupta, R. Medina, and N. Luco.

SAC/BD-00/11, *Development of Improved Post-Earthquake Inspection Procedures for Steel Moment Frame Buildings*, by P. Clark.

SAC/BD-00/12, *Evaluation of the Effect of Welding Procedure on the Mechanical Properties of FCAW-S and SMAW Weld Metal Used in the Construction of Seismic Moment Frames,* by M. Q. Johnson.

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Acronyms.

2-D, two-dimensional
3-D, three-dimensional
A, acceleration response, amps
A2LA, American Association for Laboratory Accreditation
ACAG, air carbon arc gouging
ACIL, American Council of Independent Laboratories
AE, acoustic emission (testing)
AISC, American Institute for Steel Construction
AISI, American Iron and Steel Institute
AL, aluminum
ANSI, American National Standards Institute
API, American Petroleum Institute
ARCO, Atlantic-Richfield Company
As, arsenic
ASD, allowable stress design
ASME, American Society of Mechanical Engineers
ASNT, American Society for Nondestructive Testing
ASTM, American Society for Testing and Materials
ATC, Applied Technology Council
AWS, American Welding Society
B, boron
BB, Bolted Bracket (connection)
BD, background document
BF, bias factor
BFO, bottom flange only (fracture)
BFP, Bolted Flange Plates (connection)
BM, base metal
BO, Boston, Massachusetts
BOCA, Building Officials and Code Administrators
BOF, basic oxygen furnace
BSEP, Bolted Stiffened End Plate (connection)
BSSC, Building Seismic Safety Council
BUEP, Bolted Unstiffened End Plate (connection)
C, carbon
CA, California
CAC-A, air carbon arc cutting
CAWI, Certified Associate Welding Inspector
CGHAZ, coarse-grained HAZ
CJP, complete joint penetration (weld)
CMU, concrete masonry unit, concrete block
COD, crack opening displacement
"COV," modified coefficient of variation, or dispersion
CP, Collapse Prevention (performance level)
Connection Performance (team)
Cr, chromium
CSM, Capacity Spectrum Method
CTOD, crack tip opening dimension or displacement
CTS, controlled thermal severity (test)
Cu, copper
CUREe, California Universities for Research in Earthquake Engineering
CVN, Charpy V-notch
CWI, Certified Welding Inspector
D, displacement response, dead load
DMRSF, ductile, moment-resisting, space frame
DNV, Det Norske Veritas
DRAIN-2DX, analysis program
DRAIN-3DX, analysis program
DRI, direct reduced iron
DST, Double Split Tee (connection)
DTI, Direct Tension Indicator
EAF, electric-arc furnace
EBT, eccentric bottom tapping
EE, electrode extension

EERC, Earthquake Engineering Research Center, UC Berkeley
EGW, electrogas welding
ELF, equivalent lateral force
EMS, electromagnetic stirring
ENR, Engineering News Record
ESW, electroslag welding
EWI, Edison Welding Institute
FATT, fracture appearance transition temperature
fb, fusion boundary
FCAW-G, flux-cored arc welding – gas-shielded
FCAW-S or FCAW-SS, flux-cored arc welding – self-shielded
FEMA, Federal Emergency Management Agency
FF, Free Flange (connection)
FGHAZ, fine-grained HAZ
FL, fusion line
FR, fully restrained (connection)
GBOP, gapped bead on plate (test)
gl, gage length
GMAW, gas metal arc welding
GTAW, gas tungsten arc welding
HAC, hydrogen-assisted cracking
HAZ, heat-affected zone
HBI, hot briquetted iron
HSLA, high strength, low alloy
IBC, *International Building Code*
ICBO, International Conference of Building Officials
ICC, International Code Council
ICCGHAZ, intercritically reheated CGHAZ
ICHAZ, intercritical HAZ
ID, identification
IDA, Incremental Dynamic Analysis
IMF, Intermediate Moment Frame
IO, Immediate Occupancy (performance level)
IOA, Incremental Dynamic Analysis
ISO, International Standardization Organization
IWURF, Improved Welded Unreinforced Flange (connection)
L, longitudinal, live load
LA, Los Angeles, California
LACOTAP, Los Angeles County Technical Advisory Panel
LAX, Los Angeles International Airport
LB, lower bound (building)
LBZ, local brittlezone
LDP, Linear Dynamic Procedure
LEC, Lincoln Electric Company
LMF, ladle metallurgy furnace
LRFD, load and resistance-factor design
LS, Life Safety (performance level)
LSP, Linear Static Procedure
LTH, linear time history (analysis)
LU, Lehigh University
M, moment
MAP, modal analysis procedure
MAR, microalloyed rutile (consumables)
MCE, Maximum Considered Earthquake
MDOF, multidegree of freedom
MMI, Modified Mercalli Intensity
Mn, manganese
Mo, molybdenum
MRF, steel moment frame
MRS, modal response spectrum
MRSF, steel moment frame
MT, magnetic particle testing
N, nitrogen
Nb, niobium
NBC, *National Building Code*
NDE, nondestructive examination
NDP, Nonlinear Dynamic Procedure
NDT, nondestructive testing
NEHRP, National Earthquake Hazards Reduction Program
NES, National Evaluation Services
NF, near-fault, near-field
Ni, nickel
NLP, nonlinear procedure
NLTH, nonlinear time history (analysis)
NS, north-south (direction)
NSP, Nonlinear Static Procedure
NTH, nonlinear time history (analysis)
NVLAP, National Volunteer Laboratory Accreditation Program
O, oxygen
OHF, open hearth furnace

OMF, Ordinary Moment Frame
OTM, overturning moment
P, axial load
P, axial load, phosphorus
Pb, lead
PGA, peak ground acceleration
PGV, peak ground velocity
PIDR, pseudo interstory drift ratio
PJP, partial joint penetration (weld)
PPE, Performance, Prediction, and Evaluation (team)
PQR, Performance Qualification Record
PR, partially restrained (connection)
PR-CC, partially restrained, composite connection
PT, liquid dye penetrant testing
PWHT, postweld heat treatment
PZ, panel zone
QA, quality assurance
QC, quality control
QCP, Quality Control Plan, Quality Certification Program
QST, Quenching and Self-Tempering (process)
RB, Rockwell B scale (of hardness)
RBS, Reduced Beam Section (connection)
RCSC, Research Council for Structural Connections
RT, radiographic testing
S, sulphur, shearwave (probe)
SAC, the SAC Joint Venture; a partnership of SEAOC, ATC, and CUREe
SAV, sum of absolute values
SAW, submerged arc welding
SBC, *Standard Building Code*
SBCCI, Southern Building Code Congress International
SCCGHAZ, subcritically reheated CGHAZ
SCHAZ, subcritical HAZ
SCWB, strong column, weak beam
SCWI, Senior Certified Welding Inspector
SDC, Seismic Design Category
SDOF, single degree of freedom
SE, Seattle, Washington
SEAOC, Structural Engineers Association of California
SFRS, seismic-force-resisting system
Si, silicon
SMAW, shielded metal arc welding
SMF, Special Moment Frame
SMRF, special moment-resisting frame (in 1991 UBC)
SMRF, Steel Moment Frame
SMRSF, special moment-resisting space frame (in 1988 UBC)
SN, strike-normal, fault-normal
Sn, tin
SP, Side Plate (connection)
SP, strike-parallel, fault-parallel
SP, Systems Performance (team)
SPC, Seismic Performance Category
SRSS, square root of the sum of the squares
SSPC, Steel Shape Producers Council
SSRC, Structural Stability Research Council
SUG, Seismic Use Group
SW, Slotted Web (connection)
SwRI, Southwest Research Institute
T, transverse
TBF, top and bottom flange (fracture)
Ti, titanium
TIGW, tungsten inert gas welding
TMCP, Thermo-Mechanical Processing
TN, Tennessee
TT, through-thickness
TWI, The Welding Institute
UB, upper bound (building)
UBC, *Uniform Building Code*
UCLA, University of California, Los Angeles
UM, University of Michigan
URM, unreinforced masonry
US, United States of America
USC, University of Southern California
USGS, US Geological Survey
UT, ultrasonic testing
UTA, University of Texas at Austin
UTAM, Texas A & M University
V, vanadium
VI, visual inspection
w/o, without
WBH, Welded Bottom Haunch (connection)

WCPF, Welded Cover Plate Flange (connection)
WCSB, weak column, strong beam
WF, wide flange
WFP, Welded Flange Plate (connection)
WFS, wire feed speed
WPQR, Welding Performance Qualification Record
WPS, Welding Procedure Specification
WSMF, welded steel moment frame
WT, Welded Top Haunch (connection)
WTBH, Welded Top and Bottom Haunch (connection)
WUF-B, Welded Unreinforced Flanges – Bolted Web (connection)
WUF-W, Welded Unreinforced Flanges – Welded Web (connection)

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