Developments in US Building Codes for Seismic-Resistant Steel Buildings

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Abstract

Building code regulations for seismic-resistant steel structures in the US have undergone major changes over the last several years. Many of these changes have resulted from the intensive research efforts that followed the 1994 Northridge Earthquake and the subsequent incorporation of research results into building codes. Additional changes have also resulted from the introduction of new steel lateral force resisting systems in the US, most notably the buckling restrained brace frame system. This paper will provide an overview of significant changes that have occurred over the last several years in design and construction practices and in building code regulations in the US for seismic-resistant steel buildings. Emphasis will be placed on how lessons learned from Northridge have been incorporated into building codes, particularly in the areas of welding, structural steel materials, and connection design requirements.

Keywords: steel, seismic-resistant, welding, materials, building codes

1. Introduction

It has been just over ten years since the occurrence of the 1994 Northridge Earthquake in the US, and nearly ten years since the 1995 Kobe Earthquake in Japan. In both of these earthquakes, modern welded steel moment frame buildings experienced unexpected widespread damage in the form of fractures at beam-to-column connections. This damage motivated the development of intensive research efforts in the US, Japan and many other locations to understand the causes of this damage, and to develop the needed improvements in design and construction practices to prevent such damage in future earthquakes.

In the US, the majority of large research programs aimed at studying the Northridge steel damage have now been completed. A great deal has been learned from these research efforts that has resulted in some important changes to US practices for seismic-resistant steel construction. Many of the lessons learned from Northridge and Kobe have also been rapidly adopted into US building code regulations.

The purpose of this paper is to review some of the key lessons learned from the Northridge Earthquake and to examine how these lessons have been incorporated into US building codes for steel building structures. The paper will also discuss other developments in US practices for seismic-resistant steel buildings, most notably the introduction of new steel lateral force resisting systems in US practice and codes.

2. US Seismic Codes for Steel

For many years, building codes in the US have been fragmented, with different regions of the US adopting different code provisions. At present, the US is moving towards a single national building code with the development of the \textit{International Building Code} (IBC), first released in 2000 and most recently updated for 2003 (ICC 2003). For design and detailing of steel building structures for seismic resistance, the IBC requires the use the \textit{Seismic Provisions for Structural Steel Buildings} published by the American Institute of Steel Construction (AISC). The first edition of the AISC \textit{Seismic Provisions} was published in 1990, with subsequent editions released in 1992, 1997 and 2002 (AISC 2002). The 2002 AISC \textit{Seismic Provisions} is the most current edition. The next update is scheduled for release in 2005.

In the US, the AISC \textit{Seismic Provisions} represents the key document containing building code regulations for detailing of seismic-resistant steel building structures. Consequently, the lessons learned from Northridge have been incorporated into new building code regulations through changes to the AISC \textit{Seismic Provisions}.
3. Northridge Steel Moment Frame Damage

Widespread damage to steel moment frames in the Northridge Earthquake occurred at the typical welded flange-bolted web moment connection detail widely used in the US in the early 1970’s. Damage manifested itself in these connections in the form of fractures occurring at beam flange groove welds. Figure 1 shows an example of a damaged connection.

![Fig. 1. Example of Damaged Steel Connection](image)

The majority of fractures appeared to initiate at groove welds connecting the beam bottom flange to the column. Once initiated, the fractures propagated through the weld, as seen in Fig. 1, or in some cases propagated into the column (Youseff et al 1995, Engelhardt and Sabol 1995). Subsequent research suggested the majority of these fractures likely were brittle fractures, and occurred while the beams were still in the elastic range (Kaufmann and Fisher 1995).

The total number of steel moment frame buildings damaged in the Northridge Earthquake has not been documented, although the number of damaged moment connections appears to be in the thousands. The damaged steel buildings covered a wide spectrum of ages, heights, and configurations (Youssef et al 1995). Many of the damaged buildings were designed and constructed to the most recent building codes in effect at the time of the earthquake. The widespread nature of the damage suggested fundamental problems in design and construction practices for seismic-resistant steel buildings and the need for fundamental changes in building codes.

4. Post-Northridge Research

The discovery of widespread damage to welded steel moment connections following the Northridge Earthquake led to intensive and sustained research efforts to understand the causes of the damage, and to develop improved approaches for these connections to avoid similar damage in future earthquakes. In the US, research was sponsored by a variety of groups and agencies, including the National Science Foundation, the National Institute of Standards and Technology, the American Institute of Steel Construction, and numerous other private and public sponsors. The largest effort by far, however, was funded by the US Federal Emergency Management Agency (FEMA). The FEMA funded program was managed by an organization known as “SAC,” which was a joint venture of the Structural Engineers Association of California, the Applied Technology Council, and the California Universities for Research in Earthquake Engineering. The FEMA-SAC program on steel moment frames represented a research program of extraordinary size and scope, with funding in excess of US$10 Million, and with work extending from 1994 through the year 2000.

The FEMA-SAC research program on steel moment frames has now been completed, as have many of the other post-Northridge steel related studies. This work has led to a large increase in the knowledge base on the behavior of steel moment frames in severe earthquakes. The research has been documented in hundreds of reports, conference and journal proceedings, and other publications. The results of the FEMA-SAC research were ultimately distilled into a set of design guidelines. Key among these guidelines is the FEMA 350 report on Recommended Seismic Design Guidelines for New Steel Moment Frame Buildings (FEMA 2000a).

5. Causes of Connection Damage

While a complete and unanimous consensus among the structural engineering profession in the US regarding the causes of the Northridge moment connection damage has not been achieved, the extensive research programs described above have led to a considerable understanding of the problem. Foremost among these were problems related to welding and connection design. Issues related to properties of structural steel itself were also identified. In the following sections, some of the key factors contributing to the poor performance of moment connections in the Northridge Earthquake are briefly discussed.

**Welding Factors**

Since the majority of fractures observed after the Northridge Earthquake were at beam flange groove welds, welding related issues were at the forefront of interest in post-Northridge research. This research identified several key welding problems that likely contributed to the observed damage.

The most important welding factor contributing to the moment connection damage appears to be low weld metal toughness. Beam flange groove welds prior to the Northridge Earthquake were commonly made using the E70T-4 electrode using the self-shielded flux-cored arc welding process. Charpy V-Notch (CVN) tests on weld metal from damaged
Several investigators have noted that a backing bar usually left in place after completion of the weld and weld tabs (AISC 1994, Kaufmann and Fisher Northridge) damage is the presence of backing bars fracture. The connections exceptionally vulnerable to brittle fracture. The presence of weld defects and high stress levels, made expected in the field. In many instances, it appears that little, if any, US research on moment connections prior to the Northridge Earthquake identified the importance of weld metal toughness for seismic applications.

In addition to low weld metal toughness, inadequate welding workmanship and inspection have also been identified as potentially important contributors to the observed damage (Tide 1995, AISC 1994). Upon discovery of the moment connection damage after the Northridge Earthquake, poor quality welding combined with inadequate inspection was widely speculated to be a primary cause. While field investigations clearly revealed welding defects and poor welding practices in some instances, the precise role that these defects played in the failures is not clear. For example, large-scale connection tests conducted shortly after the Northridge Earthquake evaluated welded flange - bolted web connections that were constructed under close scrutiny with very high quality welding workmanship (Engelhardt et al 1995). In addition to the very close attention to workmanship, the backing bars and weld tabs were removed from the groove welds to eliminate any potential notch condition introduced by these items. The welds passed two independent ultrasonic tests. Despite these measures to assure the highest levels of workmanship, the connections still experienced brittle fractures at the beam flange groove welds at very low levels of ductility. These failed specimens were welded using the low toughness E70T-4 electrode, emphasizing the importance of weld metal toughness. These test results implied that even with high quality welding workmanship, failures would likely still have been expected in the field. In many instances, it appears that low weld metal toughness, combined with the presence of weld defects and high stress levels, made the connections exceptionally vulnerable to brittle fracture.

A final welding related factor implicated in the Northridge damage is the presence of backing bars and weld tabs (AISC 1994, Kaufmann and Fisher 1995, Tide 1995). In typical practice prior to the Northridge Earthquake, the beam flange groove welds were made using backing bars and weld tabs that were usually left in place after completion of the weld. Several investigators have noted that a backing bar may act as an artificial edge crack, and can initiate a brittle fracture. Similarly, the weld runoff regions contained within the weld tabs have been speculated to be potential fracture initiation sites. In addition to introducing a notch, the left-in-place backing bar may also inhibit inspection of the weld. The backing bar may increase the difficulty in properly interpreting ultrasonic test results, and also prevent visual inspection of the weld root.

Connection Design Factors

While welding related issues were certainly a key problem in the Northridge moment connection damage, research has clearly shown that improved welding alone is not sufficient to achieve good connection performance. Tests conducted on the welded flange – bolted web connection both before and after the Northridge Earthquake indicate that even if weld fracture is prevented, connection performance is often still unsatisfactory (Engelhardt and Husain 1993, Tsai and Popov 1988, Stojadinovic et al 2000). Rather than fractures within the weld, fractures are frequently observed in the base metal region immediately adjacent to the weld. Such base metal fractures often occur after only limited ductility is developed. Such evidence suggests fundamental design deficiencies with the welded flange-bolted web detail.

Both laboratory and analytical studies indicate that very high levels of stress and strain are developed in the vicinity of the beam flange groove welds (El-Tawil et al 2000, Yang and Popov 1995, Mao et al 2001). These high demands have been attributed to a number of causes. Inadequate participation of the bolted beam web connection in transferring moment and shear has been identified as a cause of high beam flange stresses. It appears that much of the moment and shear normally carried in the beam web, as predicted by simple beam theory, is actually transferred through the flanges at the connection, which serves to significantly increase the stress at the beam flange groove welds. Very high localized stresses and strains are also possible due to local bending of the column and beam flanges near the beam flange groove welds. The weld access hole also introduces a significant stress concentration (El-Tawil et al 2000, Mao et al 2001). The severity of this stress concentration depends on the size, placement and geometry of the access hole. Deformations of the column panel zone also appear to have an important effect on the state of stress in the region of the beam flange groove welds, as does the presence and thickness of continuity plates. Finally, analyses have also suggested that a high degree of restraint can exist in the beam flange groove welds near the face of the column, resulting in the development of complex triaxial states of stress. The presence of such triaxial states of stress has also been conjectured to contribute to poor ductility in this region (Blodgett 1995, Yang and Popov 1995).

Analyses of welded flange-bolted web connections
indicate that very high localized stresses can occur at the root of the bottom flange groove weld, particularly near the center portion of the flange in the vicinity of the weld access hole. This region also frequently contains weld defects due to the difficulty of welding and inspection in this area. Consequently, it appears that very high stresses occurred in a region of the connection that had a high likelihood of defects and very low toughness material. The combined effects of high stresses, large defects and low toughness seem to have virtually assured poor performance of this connection.

Steel Material Related Issues

Post-Northridge research revealed no fundamental deficiencies in the material properties of the structural steel used for beams and columns in steel moment resisting frames. No pervasive problems were identified in regards to the ductility, toughness, through-thickness properties or weldability of the steel. Nonetheless, the post-Northridge research highlighted the importance of recognizing the difference between minimum specified yield stress and the actual yield stress of steel.

In typical US practice prior to the Northridge Earthquake, beams in steel moment frames were specified to be of ASTM A36 steel (minimum specified $F_y = 250$ MPa). Columns were normally specified to be either of ASTM A36 steel or of ASTM A572 Grade 50 steel (minimum specified $F_y = 345$ MPa). While ASTM A36 and A572 Grade 50 specify a minimum yield stress, neither standard specifies a maximum yield stress.

For typical non-seismic design, having steel with an actual yield stress significantly in excess of the minimum specified value usually poses no particular problem. In fact, the higher actual yield stress will normally be beneficial by providing additional reserve strength beyond that calculated using minimum material properties. However, the Northridge damage brought to light the fact that having steel members with significantly elevated yield stress values can be detrimental for seismic-resistant design.

In the case of a seismic-resistant steel moment frame, a basic cornerstone of design is that the beam-to-column connection must be stronger than the beam. That is, to achieve overall ductile frame response, plastic hinges must form at the beam ends without failure occurring at the beam-to-column connection. Thus, the connection must be designed to resist the full plastic moment of the beam. However, if the actual yield stress of the beam is significantly larger than the minimum specified value, and this fact has not been considered in the connection design, then the end result may be a connection that is, in fact, weaker than the actual as-delivered beam.

The Northridge steel damage highlighted the need to consider actual, as opposed to minimum specified steel yield stress values in certain aspects of seismic design. While the need to consider actual yield stress values is rather obvious in hindsight, this issue was not well appreciated in typical seismic design practice or in seismic building code provisions prior to the Northridge Earthquake.


As described earlier, the primary building code regulations for seismic resistant steel building structures in the US are contained in the AISC Seismic Provisions. Since the Northridge Earthquake, these provisions have been updated on a frequent basis to rapidly incorporate lessons learned from post-Northridge research. The 2002 AISC Seismic Provisions, which is the most current version of the Provisions, reflects the majority of key research findings since Northridge. As such, the 2002 AISC Seismic Provisions incorporates some major changes compared to pre-Northridge building code provisions for seismic-resistant steel construction.

Many of the post-Northridge studies focused on moment frames. Nonetheless, it is clear that many of the problems exposed in this earthquake, such as welding related problems, are issues not only for moment frames, but for all seismic-resistant framing systems. Consequently, many of the changes to the AISC Seismic Provisions resulting from the Northridge Earthquake have impacted all areas of seismic-resistant steel construction. The following sections briefly highlight what the writers consider to be some of the most important changes to the AISC Seismic Provisions that have resulted form lessons learned from the 1994 Northridge Earthquake.

Weld Metal Toughness

One of the most important outcomes of post-Northridge research has been recognition of the importance of weld metal toughness. As described earlier, poor fracture toughness of weld metal has been identified as a leading contributor to the observed damage. Prior to the Northridge Earthquake, the AISC Seismic Provisions had no requirements for weld metal toughness. Since the Northridge Earthquake, requirements for weld metal toughness have become common practice and are now mandated by code. While there has been some debate on what level of weld metal toughness should be required, there is broad consensus that weld metal toughness is crucial to the satisfactory performance of welded joints in seismic-resistant framing.

The 2002 AISC Seismic Provisions mandates minimum levels of weld metal toughness, referenced to the Charpy V-Notch (CVN) test. First, all welds in any steel seismic force resisting system (moment frames, braced frames, etc.) must have a minimum specified CVN value of 27 J at -29°C (20 ft-lbs at -20°F). Further, certain critical welds must have a minimum specified CVN value of 27 J at -29°C (20 ft-lbs at -20°F) and 54 J at 21°C (40 ft-lbs at 70°F). This requirement applies to groove welds at beam-to-column moment connections, welds at column splices, and welds at other critical
connections.

**Backing Bars and Weld Tabs**

Practices with respect to groove weld backing bars and weld tabs at moment connections have also changed. Prior to the earthquake, it was common practice to leave backing bars and weld tabs in place. Since the Northridge Earthquake, it has become common practice to remove the bottom flange backing bar in an attempt to eliminate the notch effect of the bar and to permit better inspection of the weld root. At the top flange groove weld, the backing bar is permitted to be left in place, but is welded to the column face in an attempt to mitigate the bar’s notch effect. Weld runoff tabs are removed at both the top and bottom flange groove welds. These improved practices for backing bars and weld tabs are required by the 2002 AISC *Seismic Provisions*, either through directly specified details, or indirectly by reference to other documents.

The changes to welding practices are illustrated in Figs. 2 and 3. Figure 2 shows a beam bottom flange groove weld as typically done prior to the Northridge Earthquake. Note that the backing bar and weld tabs have been left in-place. Figure 3 shows a beam bottom flange groove weld typical of post-Northridge practice. The weld tabs and backing bar have been removed and the root of the weld has been cleaned and finished with a small reinforcing fillet weld.

![Fig. 2. Typical Pre-Northridge Bottom Flange Weld](image1)

The elimination of stress concentrations introduced by the backing bar and weld tabs, the improved inspection permitted by removal of the backing, and the use of higher toughness weld metal make post-Northridge welds significantly less susceptible to brittle fracture.

**Welding Quality**

Poor workmanship and lack of adherence to proper welding procedures was clearly a factor in at least some of the Northridge moment connections failures. Welding quality and quality control, however, is a difficult issue to address. Nonetheless, some important strides have been made in this regard.

**Post-Northridge steel practice and the 2002 AISC *Seismic Provisions* address the problem of welding quality using two different approaches. The first approach is to significantly lessen the sensitivity of welded joint performance to the need for high quality welding. The pre-Northridge welded flange – bolted web moment connection constructed with low toughness weld metal was highly sensitive to weld quality. In post-Northridge practice, achieving good connection performance does not demand extraordinarily high levels of welding quality. This has been achieved by the use of higher toughness weld metal (which is more tolerant of defects), by reducing defects and other stress concentrations at welds (through improved practices for backing bars and weld tabs), and by reducing stress levels at welds though improved connection designs (discussed below). The goal has been to develop welded joint designs that can provide satisfactory performance with realistic quality welding.

![Fig. 3. Typical Post-Northridge Bottom Flange Weld](image2)

The second approach to addressing weld quality has been to promote better quality control measures. As part of the FEMA-SAC research programs, recommendations were developed on methods to achieve better weld quality control. These recommendations are contained in the *FEMA 353* report on *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications* (FEMA 2000b).

A simplified set of quality control measures will also be included in the next edition of the AISC *Seismic Provisions*, due out in 2005.

**Structural Steel Material Properties**

As described earlier, post-Northridge research highlighted the need to consider actual, as opposed to minimum specified steel yield stress values in certain aspects of seismic design, and to provide an upper limit on yield stress. These issues have been addressed by the introduction of a new grade of structural steel in the US, and by explicit recognition of actual yield stress in the design process.

A new grade of structural steel has been introduced in the US as a result of the Northridge Earthquake.
This new grade, designated ASTM A992 provides better control of yield stress and yield ratio. ASTM A992 has a minimum specified $F_y$ of 345 MPa and a minimum specified $F_u$ of 450 MPa, the same as A572 Grade 50. However, unlike A572 Grade 50, the new A992 steel places an upper limit on $F_y$ of 450 MPa, and an upper limit on yield ratio ($F_y / F_u$) of 0.85. This closer control on yield and yield ratio should permit more reliable seismic steel design.

The AISC Seismic Provisions have also been updated to reflect the fact that steel often has a yield stress higher than the minimum specified value. This has been accomplished by introducing into the provisions the expected yield stress of steel, designated as $R_y F_y$. The value of $R_y F_y$ is intended to provide an estimate of the actual expected yield stress of a particular grade of steel. For ASTM A36, A572 Grade 50, and A992, the value of $F_{ye}$ is specified to be 380 MPa. This indicates that the actual yield stress of each of these three grades of steel is expected, on average, to be 380 MPa. The expected yield stress is used for all capacity design calculations. For example, the beam-to-column connection must be designed to develop the capacity of the beam. When computing the maximum moment expected at the end of a yielded beam, the designer assumes the yield stress of the beam is equal to the expected yield stress. For brace connections in concentrically braced frames, brace connections must develop the full axial expected yield strength of the brace, taken as $R_y F_y A_y$, where $A_y$ is the gross cross-sectional area of the brace. There are many similar applications of expected yield stress throughout the 2002 AISC Seismic Provisions. Explicit recognition of the role that variations in yield stress play in ductile design of steel frames is one of the important outcomes of post-Northridge research.

Connection Design

Prior to the Northridge Earthquake, the AISC Seismic Provisions and other US seismic codes had a prescriptive requirement for beam-to-column connections in moment resisting frames. This prescriptive requirement called for the use of the welded flange-bolted web connection. The Northridge Earthquake clearly demonstrated the inadequacy of this prescriptive detail, and provided an interesting lesson in just how wrong a building code can be.

Since the Northridge Earthquake, the AISC Seismic Provisions, rather than using prescriptive details, has adopted a performance-based approach for beam-to-column connections in moment resisting frames. For Special Moment Frames (the highest ductility category for moment frames), the 2002 AISC Seismic Provisions require, in simplified terms, that the beam-to-column connection can sustain, without failure, an interstory drift angle of $\pm 0.04$ radian. Any connection design meeting this performance requirement can be used. Further, the Seismic Provisions require a designer to show conformance with this performance requirement through testing of full-scale or nearly full-scale beam-column assemblages employing the proposed connection detail. The requirement for testing reflects the view that the performance of connections under severe inelastic cyclic loading, and in particular the prediction of fracture, cannot be verified with confidence by analytical means alone.

The requirement for laboratory testing of moment connection details represents one of the most dramatic changes in US building code provisions for seismic-resistant steel structures since the Northridge Earthquake. To accompany this requirement, it was also necessary to establish detailed rules for testing, including scale requirements, required loading history, etc. These rules have been developed and are contained in Appendix S of the 2002 AISC Seismic Provisions. No such testing requirements and rules appeared in any US seismic code prior to Northridge.

To reduce the burden on designers of supplying test data to building officials, and to reduce the burden on building officials to evaluate test data, the 2002 AISC Seismic Provisions introduced the concept of “Prequalified Connections,” a concept first used in FEMA 350 (FEMA 2000a). A Prequalified connection is one that has undergone exhaustive testing, analysis and review so that a high degree of confidence exists that the performance requirements in the Seismic Provisions can be achieved on a consistent and reliable basis. Requirements for a connection to become Prequalified are specified in Appendix P of the 2002 AISC Seismic Provisions.

One of the key requirements for a connection to become Prequalified is that all available data and information on the connection must be reviewed by a panel of experts in the field of seismic connection behavior, design and construction. As such, the 2002 AISC Seismic Provisions called for the creation of a Connection Prequalification Review Panel (CPRP). Only the CPRP can confer the status of Prequalification on a connection, and set the limits of prequalification (for example, the maximum beam and column sizes with which the Prequalified connection may be used, etc.). The intent of the Seismic Provisions is that if an engineer uses a Prequalified connection, then there is no need for testing and no need to present test data to a building official. In effect, Prequalified connections can be viewed as a return to the use of prescriptive details, but ones that have undergone far more exhaustive testing and scrutiny than was done prior to Northridge.

In response to the introduction of Prequalified connections in the 2002 AISC Seismic Provisions, A Connection Prequalification Review Panel was created in the US in 2002. The CPRP was created as a committee fully authorized to develop a national building standard on Prequalified connections. The CPRP is currently developing the first edition of a new standard that will be called Prequalified Connections for Special and Intermediate Steel
Moment Frames for Seismic Applications, which is scheduled for release in 2005. This new standard, modeled somewhat after FEMA 350, will serve as a primary resource for engineers for design of moment connections. For each Prequalified connection, this new standard will specify limits of use, design and detailing rules, and a detailed step-by-step design procedure. Until this new standard is released, the commentary of the 2002 AISC Seismic Provisions indicates that connections included in the FEMA 350 report are deemed to satisfy the performance requirements for moment frame connections. Consequently, FEMA 350 is currently serving the role of a de facto prequalification standard. The FEMA 350 report, however, is not an official building standard. Once the CPRP Prequalified connection standard is released in 2005, this new standard will replace FEMA 350 as the primary resource for moment frame connections.

Since the Northridge Earthquake, a variety of new moment connection details have been developed by researchers and engineers, and many have been implemented in building construction projects in the US. However, the connection which has emerged as the most popular for steel moment frame construction is the circular cut Reduced Beam Section (RBS). Over the last ten years, the RBS connection has seen an enormous amount of testing and research, and has demonstrated the capability of achieving the moment frame performance requirements of the AISC Seismic Provisions on a very reliable basis. Further, the large research base on RBS connections has led to the development of comprehensive and relatively simple design procedures. The RBS is included in the list of connections recommended for use moment frame construction in FEMA 350, and will be included as a Prequalified connection in the soon to be released CPRP Prequalified connection standard. Figure 4 shows a photo of one of the first circular cut RBS connections used in a building construction project in the US (Engelhardt, Winneberger, Zekany and Potyraj 1996).

7. Future Developments

The next edition of the AISC Seismic Provisions is due to be released in 2005. In regard to incorporating lessons learned from Northridge, there are not likely to be further major changes. That is, most of the key lessons learned from Northridge are already reflected in the 2002 AISC Seismic Provisions, although the 2005 edition will provide for further clarification and refinement of some of the major issues discussed above. The most significant change to the 2005 edition reflecting post-Northridge research will be an extensively expanded treatment of welding quality control provisions.

Looking beyond moment resisting frames, the 2005 AISC Seismic Provisions will also be expanded to provide design and detailing requirements for two new lateral force resisting systems: steel plate shear walls and buckling restrained braced frames (BRBFs). Building code provisions for these two systems have not previously been available in US seismic codes.

Of particular interest are the new provisions for BRBFs. This new system has been rapidly gaining in popularity in the US, and has been used on a number of building construction projects. BRBFs are braced frame systems that are constructed with buckling restrained braces, also known as unbonded braces. A buckling restrained brace is made with a steel core, surrounded by a jacket that prevents the steel core from buckling. The jacket is commonly a concrete filled tube, but can also be constructed by other means. The steel core is “unbonded” from the jacket, so that longitudinal deformation of the steel core is decoupled from the jacket. The key characteristic of a buckling restrained brace is that it can yield in a highly ductile manner, both in tension and in compression (Sabelli et al 2003). In effect, it is steel member that does not buckle in compression. BRBFs avoid many of the disadvantages of traditional concentrically braced frames where brace strength and ductility are drastically different in tension and in compression. BRBFs are a system that can combine high stiffness with high ductility, similar to eccentrically braced frames.

Figure 5 shows an example of a BRBF recently constructed in the US. The introduction of building code provisions for BRBFs in the 2005 AISC Seismic Provisions will likely encourage even more use of this system.

8. Conclusions

Steel structures have traditionally enjoyed a reputation of providing excellent safety and performance in severe earthquakes. That reputation was somewhat marred as a result of the 1994 Northridge and 1995 Kobe Earthquakes. However, the unexpected problems with steel structures observed in
these earthquakes spurred on an enormous worldwide research effort of unprecedented scope that has lasted for the last ten years. These intensive research efforts have identified many of the key causes for the damage observed in these earthquakes, and have resulted in effective new design, detailing and construction strategies for seismic-resistant steel structures. US building code regulations for seismic-resistant steel structures contained within the AISC Seismic Provisions have also changed dramatically since these earthquakes, rapidly adopting key research findings. Some of the most notable changes are in the areas of welding and in connection design, with a shift towards performance-based specifications for connections and an emphasis on testing based conformance demonstration. There have also been other notable advancements in seismic-resistant steel building technology in the US, with the introduction and growing popularity of buckling restrained braced frames. Overall, the knowledge base on the performance of seismic-resistant steel structures has grown enormously over the last ten years and should contribute substantially to mitigating losses in future earthquakes.

References