



The 2005 Seismic Provisions for Structural Steel Buildings

Background on the New Standards

By Ronald O. Hamburger, SE

Since the publication of the first International Building Code 5 years ago, design of seismic-force-resisting systems of structural steel in Seismic Design Categories D, E and F has been required to conform to the requirements of the *Seismic Provisions for Steel Buildings*, a standard published by the American Institute of Steel Construction (AISC). The first AISC Seismic Provisions were published in the early 1990s, at a time when most steel structures were designed using allowable stress procedures and when the design of most structures for seismic resistance was performed under the Uniform Building Code.

At the time the first AISC *Seismic Provisions* were published, most engineers believed that steel structures inherently performed well in earthquakes, regardless of the precautions taken in their design and construction. Consequently, the first *Seismic Provisions* was a relatively thin document, with comparatively few detailed requirements. Then, on January 17, 1994, the Northridge Earthquake struck the San Fernando Valley north of Los Angeles, one of the most densely populated regions in the United States. The Northridge earthquake caused more than \$30 billion of damage and affected a wide range of construction types. Engineers and researchers both were shocked by the discovery of brittle fracture damage in moment-resisting steel frames, a system that was thought to be among the most resistant to earthquake-induced damage.

In response to the unanticipated damage, the Federal Emergency Management Agency (FEMA) funded a coalition known as the SAC Joint Venture, comprised of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC) and the Consortium of Universities for Research in Earthquake Engineering (CUREe), to conduct a major program of research and development. The goals of the FEMA/SAC project were to determine the cause of damage to moment-connections experienced in the Northridge earthquake, and to develop a series of recommendations that covered: inspection and repair of damaged buildings, evaluation of the risks associated with existing buildings; upgrade of existing buildings and design of new buildings. The resulting \$12 million program engaged the resources and participation of practicing engineers and researchers from around the United States, as well as fabricators, inspectors and representatives of the American Institute of Steel Construction and the American Welding Society.

In August of 2000, the FEMA/SAC program culminated with the publication of a series of recommended engineering guidelines. FEMA-350 addressed the design and construction of new steel moment-frame buildings, FEMA-351 the evaluation and upgrade of existing buildings, FEMA-352 the inspection and repair of earthquake damage in existing buildings, and FEMA-353 construction quality assurance requirements that covered both the construction of new buildings and the repair and upgrade of existing buildings.

The FEMA/SAC recommendations for the design and construction of new buildings were broad sweeping and covered material characteristics used for structural shapes and welding filler metals, the configuration and detailing of connections, design



procedures for evaluating strong-column weak-beam conditions and panel zone strength requirements, fabrication methods and tolerances and programs for construction quality assurance. Of these, perhaps the most significant recommendation was to limit connection configurations and design procedures to those that had been validated through a qualification program of analysis and testing and that had been demonstrated to be capable of providing satisfactory performance under extreme cyclic loading. FEMA-350 included a series of connection configurations, together with specific design procedures that were categorized as pre-qualified for use in seismic applications within specific limits of frame and member geometry, based on the testing and analyses conducted under the FEMA/SAC program. The FEMA/SAC recommendations represented significant improvements in practice but were not in the form of consensus standards and could not be referenced or adopted by the building codes. Thus, use of the recommendations was largely voluntary.

Even before the publication of FEMA-350 and its companion documents, the American Institute of Steel Construction and the American Welding Society began the process of developing ANSI consensus standards to incorporate the recommendations into code-adoptable format. AISC was first to succeed in this process, with its publication of the 1997 edition of the AISC *Seismic Provisions*. The 1997 edition incorporated the FEMA/SAC recommendations published in the FEMA-267 *Interim Guidelines* document and also updated requirements for concentric and eccentric braced framing systems. Several supplements to the 1997 AISC *Seismic Provisions* were published in the next several years as the FEMA/SAC program continued to develop recommendations.

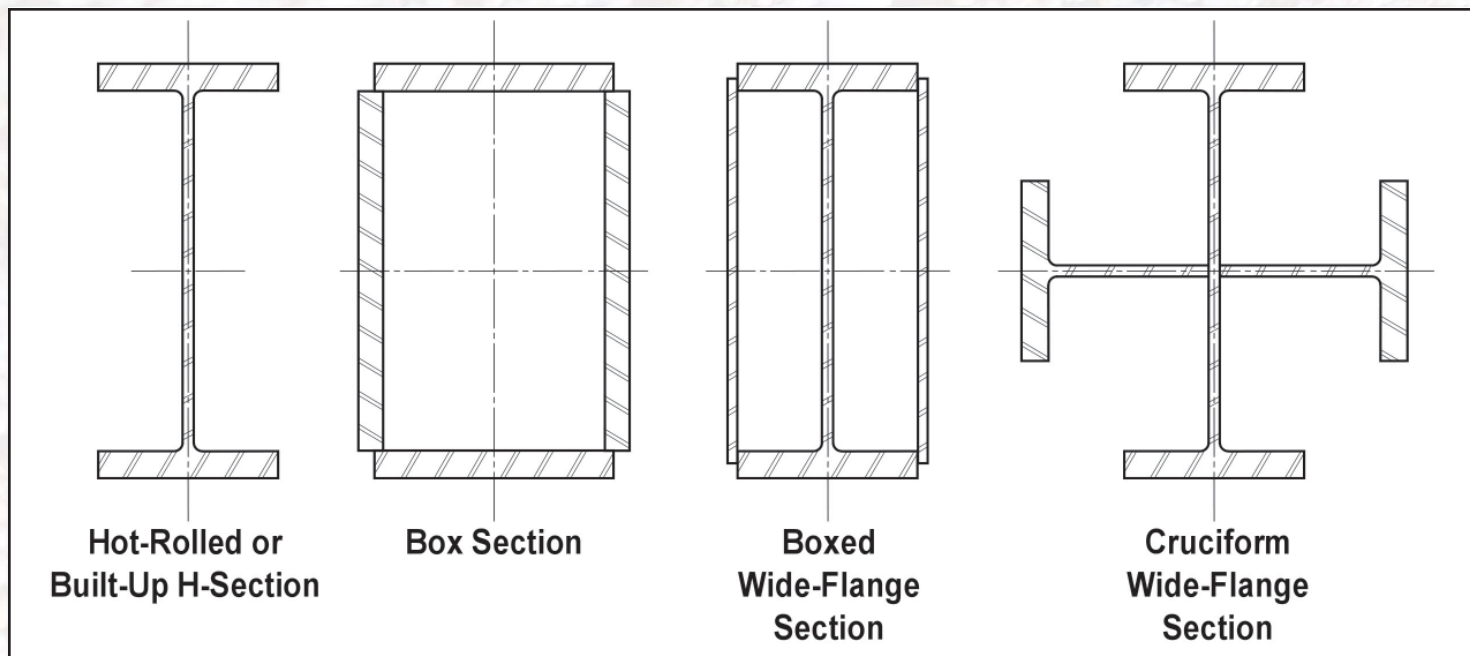


Figure 1: Column sections permitted under AISC-358

The AISC *Seismic Provisions* are primarily intended to provide criteria for the design of seismic-force-resisting systems of structural steel. Material characteristics, detailing and workmanship requirements for structural steel systems have generally been covered by the American Welding Society's D1.1 *Structural Welding Code*, the Research Council on Structural Connections *Specification for Structural Joints Using ASTM A325 and A490 Bolts* and the *Steel Construction Manual*. However, because these other publications were unable to quickly adopt the appropriate requirements, the AISC *Seismic Provisions* were revised and republished in 2002 incorporating essentially all of the recommendations contained in FEMA-350 and many of the quality assurance and material requirements in FEMA-353. The 2002 AISC *Seismic Provisions* did not, however, incorporate the pre-qualified connections, instead referring to connections approved by a connection pre-qualification review panel.

AISC established the Connection Prequalification Review Panel in 2002 specifically to develop a new standard containing pre-qualified connections and corresponding design procedures for critical connections in moment-resisting frames and eccentric braced frames designed under the AISC Seismic Provisions. Meanwhile, the American Welding Society commissioned Subcommittee 12-Seismic of its D1.1 Standards Committee, specifically to develop a seismic supplement to the D1.1 *Structural Welding Code*.

As a result of these efforts, two new standards are about to be published. The first is the AISC-358 *Pre-qualified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* which has been approved by AISC and will be included in the new Seismic Design Manual to be published early in 2006. This new standard contains design and detailing requirements for three types of moment-resisting connections: Reduced beam section connections, end plate connections and extended end plate connections. Although it does not include several of the other pre-qualified connections contained in FEMA-350, it does significantly expand the limits of application for the connections that are included. The Connection Prequalification Standard permits the use of these connections with hot-rolled and built-up "H" section columns to depths of 36 inches. It also permits use of these connections with box columns up to 24 inches in depth, boxed-wide flange columns and cruciform shape columns fabricated from standard wide flange shapes (Figure 1). The box, boxed-wide flange and cruciform column sections can be used

in moment-frames acting in orthogonal directions, an important addition, since there are no pre-qualified connections for joining beams to the minor-axis of wide flange section columns. Even as the standard is being published, the Committee is continuing to work to include additional pre-qualifications and to extend the standard for application to eccentric braced frames.

AWS is in the final process of publishing the second new standard: D1.8 *Seismic Supplement* to the AWS D1.1 *Structural Welding Code*. This document covers the materials, workmanship and requirements for welded construction on steel seismic force resisting systems. Topics considered by this standard include: responsibilities of the engineer, the contractor and the inspector in the design and construction process; requirements for certifying and documenting procedures and materials; detailing of welded joints including treatment of access holes, corner clips, backing and weld tabs; qualifications of welders and inspectors; limitations on welding procedures; material, handling and storage requirements for weld filler metals; fabrication tolerances and repair requirements; and special procedures for qualifying weld filler metals for seismic applications.

Finally, the AISC Seismic committee has published a 2005 edition of the *Seismic Provisions* that updates references to the other new standards and also incorporates design requirements for two new systems: special steel plate shear walls and buckling restrained braced frames. In addition, the 2005 Provisions updates design requirements for the other popular structural steel seismic-force-resisting systems. A pending supplement to the 2005 Provisions modifies the design requirements for ordinary concentric braced frames and permits their use in seismically isolated structures.

Engineers who design in structural steel should become acquainted with these new standards. AISC members can download the AISC standards without charge from the *epubs* page on AISC's website. The new D1.8 standard should be available from AWS in early 2006. ■

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The 2005 AISC Seismic Provisions for Structural Steel Buildings

An Overview of the Provisions

By James O. Malley, S.E.

An update to the American Institute of Steel Construction (AISC) *Seismic Provisions for Structural Steel Buildings* was recently completed. Using the 2002 edition as the basis, the 2005 AISC *Seismic Provisions* considers the most current research and known building performance to develop appropriate design specifications for structural steel buildings in seismic regions. This article highlights the important features of these provisions.

One major change to the *Seismic Provisions* is format: it combined Allowable Stress Design (ASD) and Load and Resistance Factored Design (LRFD) into a single, unified approach. As such, Part III in previous editions accommodating ASD was absorbed into Part I for Structural Steel Building and in Part II for Composite Steel and Concrete Systems.

Part I – Structural Steel Buildings

The first four sections of Part I integrate the seismic design provisions with the *AISC LRFD Specification*, the *Applicable Building Code* (ABC) and other applicable national standards such as ASCE7, ASTM, etc.

The *Seismic Provisions* apply to buildings classified by the ABC as Seismic Design Category D or more severe. In less severe Seismic Design Categories A, B and C, the system must satisfy one of two conditions: Use a Structural Modification Factor, R , of 3 and design elements to satisfy the main AISC *Specification* only, or use a higher R value and designed to the AISC *Seismic Provisions*. The second requirement is to prevent large R -values being used for a structure without meeting the ductile detailing requirements of the *Seismic Provisions*.

The newly developed Section 5 outlines information needed for construction documents as prepared by the project team consultants. Design drawings and specifications are required to identify all elements of the Seismic Load Resisting System (SLRS), demand critical welds, protected zones, connection configuration, welding requirements, etc. Similar information is required by shop drawings and erection drawings to verify that the fabricator and erector understand the design intent.

Section 6 considers acceptable material properties and characteristics for structural steel systems in seismic regions. One of the more important aspects of this section is the need to consider the expected yield strength and the expected tensile strength in determining the Required Strength. For each structural material type, R_y is specified that when multiplied by the nominal yield strength F_y , results in the expected yield strength of the material. A second term R_t has been introduced that when multiplied by the nominal tensile strength, F_u , results in the expected tensile strength of the material. The remaining seismic design provisions identify when the R_y and R_t terms are to be used in determining the required strength of the members.

The design of connections, joints and fasteners in the Seismic Load Resisting System (SLRS) is addressed in Section 7. All connections should be detailed so that a ductile limit state controls the strength of the components. It also defines the "Protected Zone," or the critical regions of elements in the SLRS where discontinuities must be avoided to minimize the chance of premature, brittle fracture of the members.

All bolted connections are to use pre-tensioned, high strength bolts, with the faying surface prepared for Class A or better Slip-Critical joints. But, bolted connections may be designed for the strength in bearing. This requirement is to avoid joint slip during small earthquakes, while recognizing that bolts will eventually develop bearing during a design-level seismic event. Standard holes are required at bolted joints, except short-slotted holes are acceptable when the axis is perpendicular to the direction of load. Oversized holes may also be used, if they are in only one ply of the slip-critical joint. Bolts and welds are not allowed to share load at the same joint.

Welded connections must be made with filler metals having a minimum CVN toughness of 20 ft-lbs at 0°F. This is a relaxation from the previously adopted temperature of -20°F. An additional requirement of 40 ft-lbs at 70°F CVN toughness is placed on Demand Critical complete joint penetration groove welds (e.g. welds of beam flanges to columns, column splice joint, and welds of beam webs to column flanges) in various systems. Specific detailing requirements for continuity plates are also provided in this section.

Requirements for local and global instabilities, as well as other general member requirements, are considered by Section 8. The maximum limit of width-to-thickness ratios of flanges and webs for members in the SLRS is provided. These ratios are more restrictive than the compact section ratios given in the main AISC *Specification* because of the expected inelastic demand during seismic behavior. The remaining portion of this section emphasizes column design. Splices for columns that are not part of the SLRS now have special design requirements since research indicates these columns may have significant flexure and shear demand during a severe seismic event.

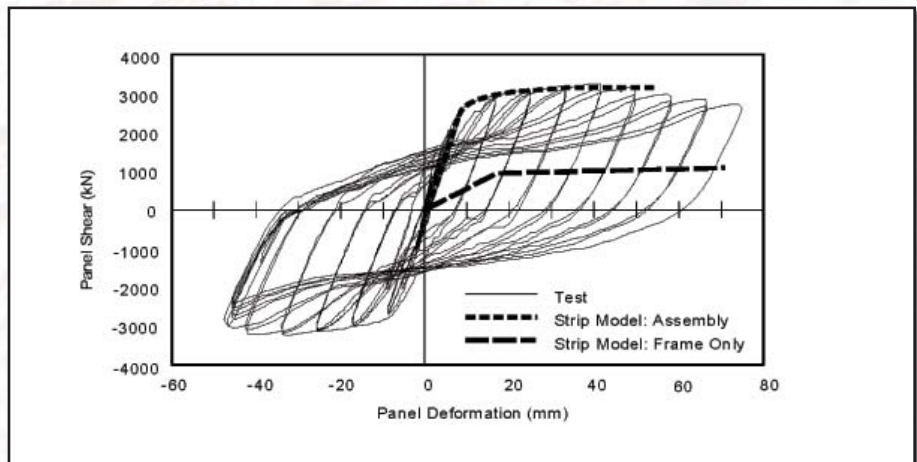


Figure 1: Steel Plate Shear Wall test results (Driver, et. al.)

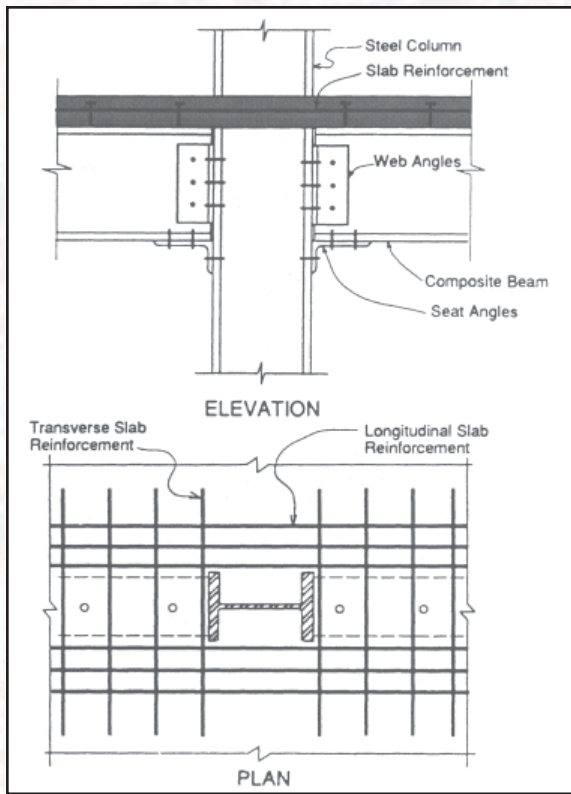


Figure 2: Example of a partially restrained composite connection

The next nine sections (Sections 9 through 17) provide design requirements for each of the codified structural steel building systems:

Special Moment Frames (Section 9)

SMFs are considered highly ductile, and, therefore, have been assigned the highest R factor. The proposed use of a particular moment-resisting joint must have a demonstrated capability of accommodating an interstory drift of 0.04 radians. This is accomplished by one of the following:

- 1) Using a connection pre-qualified for use as a SMF in accordance with ANSI/AISC 358, a document developed by the AISC Connection Prequalification Review Panel (CPRP). In its first edition published in 2005, the AISC CPRP included prequalification of the Reduced Beam Section and End Plate connections, but efforts continue to eventually pre-qualify all widely used connections.
- 2) Using a connection pre-qualified for use as a SMF in accordance with Appendix P, criteria that establishes minimum requirements for any moment-resisting joint. A Connection Prequalification Review Panel (CPRP) is to be established that will review all test results and other data to ensure the connection satisfies all minimum requirements.
- 3) Providing qualifying test results in accordance with Appendix S. This appendix requires the test assembly to be consistent with joints proposed in the prototype building, defines essential test parameters, and identifies the test program implementation and the adequacy of the joint to sustain the required seismic demand. Test results can be taken from tests reported in the literature, or from tests performed specifically for the project under consideration.

The panel zone must be consistent with the pre-qualified test configuration and the expected strength must be approximately “balanced” with the yield strength of the beams. In addition, all column splice strengths in bending and shear must be designed to develop the full flexural capacity of the smaller column spliced.

Intermediate Moment Frames (Section 10)

Like SMFs, IMFs must have moment-resisting connections qualified in accordance with ANSI/AISC 358, Appendix P or Appendix S. The qualifying interstory drift limit is reduced to 0.02 radians for these connections to reflect the more limited ductility demand expected from these systems. Current building codes limit the use of IMFs in high Seismic Design Categories. Other than the pre-qualified connection and the more restrictive lateral bracing requirements, the main AISC *Specification* governs the design requirements of these frames.

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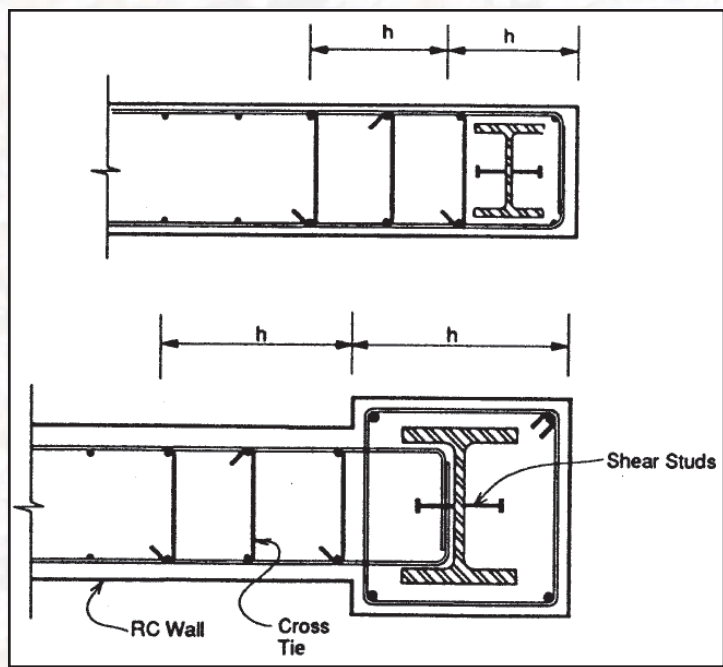


Figure 3: Composite shear wall boundary element

Ordinary Moment Frames (Section 11)

OMFs are accepted in light metal buildings and small building applications in the more severe seismic design categories. OMFs may be designed without the pre-qualified performance-testing requirement. In an effort to induce inelastic behavior into the adjoining elements, the connection strength must exceed 1.1 times the expected strength of the connected members. Specific requirements such as continuity plates, removing weld backing and run-off tabs and weld access holes help ensure minimum ductile performance of OMF connections.

Special Truss Moment Frames (Section 12)

STMF provisions define a special segment of the truss that is intended to be the primary location of inelastic behavior in the system. All other frame elements are designed with sufficient over-strength to develop yielding in the special segment. Both vierendeel and cross-braced special segment panels are allowed. The requirements also provide lateral bracing requirements similar to those required for SMF systems to prohibit out-of-plane instability.

Special Concentrically Braced Frames (Section 13)

The concept for SCBF systems is that diagonal braces buckle and dissipate energy resulting from the design earthquake. Provisions have been modified to improve the ductility of the system. For example, brace orientation in each line of framing must have approximately the same number of braces in compression and tension. Connections in SCBF must develop the full tensile capacity of the brace or the maximum force that can be delivered to the brace by the rest of the system. Full flexural strength must also be considered unless the connection includes a yield-line gusset plate that allows ductile, post-buckled behavior of the brace. Special limitations are provided for V and inverted-V bracing to reflect the potentially undesirable characteristics of these brace configurations. Column splices in SCBF are required to develop a shear capacity of approximately 50 percent of the member capacity to reflect the substantial demands on these elements during the earthquake.

Ordinary Concentrically Braced Frames (Section 14)

Like OMFs, OCBFs have highly restricted applications in high Seismic Design Categories due to their limited expected ductility. Connections in OCBF's are designed to consider the Amplified Seismic Load. The previous requirement of member design in OCBF's for the Amplified Seismic Load was removed to address the reduced R factor given in ASCE 7-05 (the building code that references the 2005 AISC *Seismic Provisions*).

Eccentric Braced Frames (Section 15)

Design provisions for the EBF design are to induce full yield and strain hardened within the eccentric link while the diagonal braces, columns and beams outside the link beams remain essentially elastic. Because of their importance to system performance, proper design of the link beam is the primary focus of this section. Link beams may be designed to yield in either shear or flexure, or in combination of both. Laboratory testing has demonstrated that properly designed shear yielding links can undergo a link rotation angle of 0.08 radians. Moment yielding links are designed to undergo a link rotation angle of 0.02 radians, which is consistent with SMF systems. Interpolation is allowed for links with a length that results in a combination of shear and flexural yielding. Because of the high local deformation demands, link-to-column connections must be demonstrated by testing similar to SMF connections, in accordance with Appendices S and P or ANSI/AISC 358. An exception is provided if there is substantial reinforcement of the connection that would preclude inelastic behavior in the connection welds.

Buckling Restrained Braced Frames (Section 16)

Originally developed in Japan, the BRBF system has been used on a number of recent projects on the West Coast. This system relies on sustained compression due to local buckling of the brace while overall member buckling is restrained. This significantly increases the energy dissipation characteristics compared to the braces in a traditional SCBF system; therefore, BRBFs do not have the in-line brace configuration or other restrictions that are imposed on the SCBF. Similar to other structural system types, braces in a BRBF require pre-qualification testing as specified by Appendix T. The remaining design provisions are intended to ensure that the connections and other members in the BRBF system remain essentially elastic at full capacity of the brace.

Special Steel Plate Shear Wall (Section 17)

Although used on a number of buildings in high seismic regions as early as the 1970's, renewed interest in SPSW systems was generated in the early 1990's resulting from a series of research projects at two Canadian universities. *Figure 1* shows typical inelastic behavior that might be expected from a SPSW. From this Canadian research, as well as on-going research in the U.S., design requirements for the system have been codified. Favorable seismic performance is achieved by controlling stable post-buckled strength in the web of the steel plate shear wall. Similar to plate girder behavior, tension field action develops as the relatively thin web buckles during lateral loading. Limitations on configuration, width-thickness ratios and other design parameters are provided to be consistent with the successful test results.

The final section of Part I (Section 18) addresses a comprehensive quality assurance plan that is required to demonstrate that structural design intent is accomplished during construction. Newly developed Appendix Q discusses requirements related to quality control to be provided by the contractor, and quality assurances. Inspection requirements, both visual and non-destructive evaluation (NDE) inspections, for welds are presented in tabular form. A similar table for bolted connections is also provided.

Part II – Composite Structural Steel and Reinforced Concrete Buildings

Part II of the Seismic Provisions considers the design of composite systems of structural steel and reinforced concrete. Since composite systems are assemblies of structural steel elements with concrete components, cross-references with ACI 318 is an important feature in this *Seismic Provision*.

Part II contains individual sections governing design requirements for beams composite with concrete slabs, composite columns and the design of connections between concrete and steel elements. To-date, engineers have designed composite connections using the basic principles of mechanics, existing standards for steel and concrete construction, test data, and engineering judgment. The connection section is intended to standardize and improve design practice by establishing basic behavioral assumptions for developing design models that satisfying equilibrium of internal forces in the connection for seismic design.

The remaining sections of Part II address the design of various composite structural system types. These sections parallel those found in Part I, and generally have R factors and system application limitations similar to the comparable structural steel systems. In addition to the Composite SMF, IMF and OMF systems requirements, there is a Composite Partially Restrained Moment Frame (C-PRMF) system having connection details similar to that shown in *Figure 2*. Similar to Part I, there are two concentrically braced and one eccentrically braced composite system addressed. Part II also identifies three composite systems utilizing wall elements as the primary component in the SLRS. Two types of composite walls, one Special and one Ordinary, parallel

the reinforced concrete wall specifications of ACI 318, except structural steel elements are used in the boundary elements (as shown in *Figure 3*). Finally, a Composite Steel Plate Shear Wall system is also codified.

Conclusion

Over the last ten years, a rational approach to the seismic design of structural steel buildings has evolved into the latest developments in 2005 AISC *Seismic Provisions*, a document that has been adopted by reference in the 2006 *International Building Code*. As a result, the seismic design of all steel buildings in the United States are now governed by this document, creating a unified design approach independent of the local building jurisdiction. This will lead to better designs and improved performance of steel buildings in future earthquakes. ■

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More information may be found in The 2005 AISC Seismic Provisions for Structural Steel Buildings by James O. Malley published in the proceedings of 2005 North American Steel Construction Conference, Montreal, Canada, April, 2005.

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