The 2005 AISC Seismic Provisions for Structural Steel Buildings

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The 1994 Northridge earthquake resulted in an unprecedented level of interest in the seismic performance of steel frame structures. This interest led to major research and development programs sponsored by the Federal Emergency Management Agency (FEMA) and others, including the FEMA/SAC Steel Program. As a result of these efforts, significant modifications to the United States seismic design provisions for steel structures have taken place. The AISC *Seismic Provisions for Structural Steel Buildings*, hereafter referred to as the AISC Seismic Provisions, were almost completely rewritten in 1997, with additional major modifications in 1999 and late in 2000. The 2002 AISC Seismic Provisions are the basis for the steel seismic design provisions in the 2002 National Fire Protection Association (NFPA) 5000 and the 2003 International Building Code (IBC). The 2002 AISC Seismic Provisions incorporate information from the final FEMA/SAC recommendations presented in FEMA 350 through 355. The contents of the previous editions of the AISC Seismic Provisions will be briefly summarized in this paper. The paper will focus on the 2005 AISC Seismic Provisions that are referenced in the 2006 IBC. In addition to updating the requirements for systems used in current design practice to be consistent with recent research results, these provisions also have incorporated two new systems that can be used for various applications (steel plate shear walls and buckling restrained braced frames). The 2005 AISC Seismic Provisions (AISC, 2005b) have been developed so that the main AISC *Specification for Structural Steel Buildings* (AISC, 2005a) (also completed in 2005) can be used as a primary reference.

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HISTORY OF U.S. SEISMIC CODE DEVELOPMENT FOR STEEL BUILDINGS

Between the advent of incorporation of modern seismic design provisions into the Uniform Building Code (UBC) in the early 1950s, and the late 1980s, the Structural Engineers Association of California (SEAOC) Seismology Committee was almost completely responsible for the content of these provisions with the primary resource for the UBC being the SEAOC *Recommended Lateral Force Requirements and Commentary*, more widely known as the "SEAOC Bluebook." Spurred on by the federally funded National Earthquake Hazard Reduction Program (NEHRP), in the late 1970s and early 1980s seismic design issues began to be seen as more of a nationwide issue. The initial seminal document that resulted was developed by the Applied Technology Council (ATC), ATC 3-06 (ATC, 1978). During these years, the NEHRP program began to fund the Building Seismic Safety Council (BSSC) in developing model building code provisions for seismic design. The BSSC established a nationally represented committee structure, with technical subcommittees addressing each of the main structural materials, including structural steel. To support this effort, AISC established a parallel effort and began the development of a set of seismic design provisions for steel buildings. These provisions, first published in 1992 under the direction of Professor Egor Popov, were similar in scope and content to the SEAOC developed UBC Provisions, but developed in the strength design basis or load and resistance factor design (LRFD) format rather than allowable stress design (ASD). Since the entire 1994 NEHRP Provisions (FEMA, 1994) document was based on a strength design basis (as opposed to the ASD based provisions of the UBC provisions of the time), the 1992 AISC Provisions (AISC, 1992) were adopted by reference with minor modifications.

With the damage to steel buildings caused by the Northridge earthquake, there was a significant effort to update the seismic design provisions. A major program was undertaken, sponsored by FEMA, to develop reliable, practical, and cost effective guidelines for the design and construction of new steel moment frame structures, as well as for the inspection, evaluation and repair, or upgrading of existing ones. This program was completed for FEMA by the SAC Joint Venture

[SAC being the first initial of the three joint venture partners, SEAOC, ATC, and the California Universities for Research in Earthquake Engineering (CUREE)]. The FEMA/SAC steel project culminated late in 2001 with the publication of design guidelines applicable to moment frame buildings located throughout the U.S. The project recommendations are contained in:

- FEMA 350—*Recommended Seismic Design Criteria for New Steel Moment Frame Buildings* (FEMA, 2000a).
- FEMA 351—*Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment Frame Buildings* (FEMA, 2000b).
- FEMA 352—*Recommended Post-Earthquake Evaluation and Repair Criteria for Welded, Steel Moment Frame Buildings* (FEMA, 2000c).
- FEMA 353—*Recommended Specifications and Quality Assurance Guidelines for Steel Moment Frame Construction for Seismic Applications* (FEMA, 2000d).

Detailed derivations and explanations of the basis for these design and evaluation recommendations may be found in a series of State of the Art Reports, also published by FEMA (FEMA, 2000e). In addition, technical reports for each of the more than 60 investigations were prepared to document the analytical and experimental studies completed as part of the program.

THE 1997 AISC SEISMIC PROVISIONS

With significant input and coordination with the SEAOC Seismology Committee, the 1997 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) were completed as a joint effort of the AISC and BSSC subcommittees on seismic design. As a result of this joint effort, these provisions were adopted by reference in the 1997 NEHRP Provisions (FEMA, 1997a), without modification. The 1997 AISC Seismic Provisions incorporated many of the early findings and advances achieved as part of the FEMA/SAC program and other investigations and developments related to the seismic design of steel buildings. Part I of these provisions, in LRFD format, updated design requirements for materials, welded and bolted joints, columns, and column splices that apply to all structural systems. System specific new requirements were also provided for each different structural system. Three new systems—intermediate moment frames (IMF), special truss moment frames (STMF), and special concentrically braced frames (SCBF)—were introduced in the 1997 AISC Seismic Provisions. A major expansion of the quality assurance requirements for the seismic system was also included. Finally, an appendix for the testing of steel moment resisting connections was developed to assist engineers engaged in project-specific testing. Part II of the Provisions addressed the design and construction of composite steel and reinforced concrete. Part III of the Provisions mimics Part I but is written in ASD format. This part was included in the AISC Seismic Provisions to ease the transition from working stress design to strength-based design of steel structures. Part I of the 1997 AISC Seismic Provisions was incorporated into the 2000 IBC (ICC, 2000) by reference.

SUPPLEMENTS NOS. 1 AND 2 TO THE 1997 AISC SEISMIC PROVISIONS

Recognizing that rapid and significant changes in the knowledge base were occurring for the seismic design of steel buildings, especially moment frames, the AISC Committee on Specifications committed to generating frequent supplements to the AISC Seismic Provisions. This commitment was intended to keep the provisions as current as possible. The first such supplement had an approval date of February 15, 1999. This supplement incorporated the new American Society of Testing and Materials (ASTM) A992 specification for W-shapes, alerted the design community of the need to properly consider the potential for low toughness material in the "k-area" of rolled shapes, added the requirement that all welds in the seismic load resisting system be made with filler metals that have a rated notch toughness, and clarified the intent of the gusset plate configuration in SCBFs. Supplement No. 1 to the 1997 AISC Seismic Provisions (AISC, 1999) was also incorporated into the 2000 IBC by reference.

Supplement No. 2 to the 1997 AISC Seismic Provisions (AISC, 2000) was dated November 10, 2000. Supplement No. 2 attempted to incorporate many of the final recommendations generated by the FEMA/SAC Project. This supplement added requirements to avoid material discontinuities in recognition that such discontinuities in these critical zones can lead to premature fracture, revised the requirements for panel zone shear strength in special moment frames (SMF) such that excessively weak panel zones would be avoided, and tightened the column width-thickness ratio and lateral bracing requirements for conditions where column inelasticity is a possibility. Also, this supplement redefined the intermediate moment frame (IMF) and ordinary moment frame (OMF) systems and made the limitations on the use of ordinary concentrically braced frames (OCBF) more stringent, reflecting the limited ductility of this system. Consistent changes were also made to Part II of the document. Commentary changes were made to be consistent with the proposed changes and to improve other sections.

THE 2002 AISC SEISMIC PROVISIONS

Because the scope of changes that were made to the AISC Seismic Provisions since 1997 was so large, the provisions were republished in their entirety in 2002 (AISC, 2002). The 2002 edition of the AISC Seismic Provisions further incorporated the results of the FEMA/SAC project that were published late in 2000. In addition, these provisions were modified as necessary to be consistent with the American Society of Civil Engineers (ASCE 7-02) standard, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002). This allowed the document to be incorporated by reference into the editions of the IBC and NFPA building codes that use ASCE 7-02 as their basis for design loads. As a result, no matter which of the two codes have been adopted by local jurisdictions since 2003, the seismic design of steel buildings in the United States will be governed by this document. This should help allow engineers to develop their designs in a more consistent fashion. Similar to the 1997 edition, the 2002 AISC Seismic Provisions are presented in three parts: Part I, the LRFD provisions for structural steel buildings; Part II, the provisions for composite structural steel and reinforced concrete buildings; and Part III, the provisions for ASD.

A number of specific changes were made to make these provisions as current as possible. Some of the updates included the following:

1. A clarification to the scope statement to verify that chord and collector/drag elements in floor diaphragms are considered to be part of the seismic load resisting system (SLRS) and therefore must meet the provisions.

2. Additional requirements for the toughness of filler metals to be used in critical welds of some seismic systems [for example, complete-joint-penetration (CJP) groove welds in moment-resisting frames]. These additional requirements include a two-level toughness as proposed in FEMA 350 and FEMA 353 (FEMA, 2000a and 2000d).

3. A revision to clarify member slenderness ratio requirements and better coordinate with the LRFD provisions.

4. Increase the moment frame column splice requirements to be consistent with the FEMA/SAC recommendations.

5. Clarify column base design demands for various systems.

6. Clarify lateral bracing requirements of moment frame beams.

7. Increase SMF web connection design requirements to be consistent with FEMA 350.

8. Incorporate FEMA 350 recommendations for weld access holes in OMF systems.

9. Provide new commentary to alert designers about potential net section weakness for hollow structural shape (HSS) braces connected through a single gusset plate in concentrically braced steel frames.

10. A new appendix that defines procedures to be used in the prequalification of moment connections. These procedures are based on recommendations from the FEMA/SAC program. AISC has established a separate committee, the Connection Prequalification Review Panel (CPRP) specifically for this purpose.

11. Provisions for the appropriate slenderness ratios to be used in steel H-piles subjected to seismic demands.

12. Update Parts II and III to be consistent with Part I.

THE 2005 AISC SEISMIC PROVISIONS

A major change in the 2005 AISC Seismic Provisions is in format (AISC, 2005b). Consistent with the changes to the main design specification, the 2005 Seismic Provisions combine ASD and LRFD into a single specification. As such, Part III in previous editions (which addressed ASD) of the Seismic Provisions has been absorbed into Part I.

Two systems that were initially developed and incorporated into the 2003 NEHRP Provisions are the buckling-restrained braced frame (BRBF) and the special plate shear wall (SPSW). Both of these systems are included in the 2005 Seismic Provisions. A brief summary of these two new systems is included later in this paper.

A number of other significant technical modifications occurred. These include the following:

1. Clarified the scope of structures to include "building-like" non-building structures.

2. Clarified that all steel buildings designed with an R factor greater than 3 must comply with the AISC Seismic Provisions (see later discussion).

3. Added new requirements to delineate the expectations for structural design drawings and specifications, shop drawings, and erection drawings.

4. Added new ASTM material specifications that are commonly used in the metal building industry.

5. Added R_t values for all materials to be used in determining susceptibility of connections to fracture failure modes.

6. Relaxed the limitations of oversized holes in bolted joints.

7. Defined a new term, "demand critical welds," that have additional quality and toughness requirements. For each system, welds considered to be demand critical are defined.

8. Defined a new term, "protected zone," to ensure that areas expected to be subjected to significant inelastic strains

are not disturbed by other building operations. For each system, areas that are considered to be protected zones are defined.

9. Made the requirements on column splices that are not part of the SLRS apply to all systems, not just moment frames.

10. Improved the provisions related to column bases.

11. Made the stability bracing requirements more consistent throughout the document.

12. Added references to the new AISC standard on *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358) (AISC, 2005) as one means for SMF, IMF, and EBF (link-to-column) connection acceptance. ANSI/AISC 358 is also adopted into ASCE 7-05 with Supplement No. 1 (ASCE, 2005) by reference.

13. Decreased the column splice shear capacity requirements for SMF systems.

14. Increased the stability bracing requirements for IMF systems.

15. Clarified that connections meeting the requirements for SMF or IMF systems are also acceptable for OMF applications.

16. Increased the requirements on SCBF systems that employ high Kl/r ratio braces.

17. Reduced the connection force demand on OCBF bracing to allow the use of the amplified seismic load factor.

18. Eliminated the requirement to design all members in OCBF systems for the amplified seismic load. This was done in concert with a commensurate reduction in the R factor for this system in ASCE 7-05.

19. Added special bracing configuration requirement checks for V and inverted-V bracing configurations to OCBF systems.

20. For BRBF systems, increased the deformation that the system is required to demonstrate to accommodate from 1.5 to 2.0 times the design story drift.

21. Added a strong column-weak beam check for frames in SPSW systems.

22. Significantly improved the provisions related to quality assurance and quality control to address many of the issues identified in FEMA 353.

23. Added new requirements for welded joints based on FEMA 353 recommendations including the following:

    - Qualifications for inspection personnel.
    - Acceptable nondestructive testing procedures.
    - Intermixing of filler metals.
    - Acceptable limits of diffusible hydrogen in filler metal.
    - Maximum wind speeds for gas shielded welding.
    - Limitations on maximum interpass temperatures.
    - Proper application of weld tabs.
    - Additional requirements for demand critical welds related to welding processes, filler metal packaging and exposure, and the use of tack welds.

24. Made changes to Part II to be consistent with the modifications to Part I and changes to the American Concrete Institute standard ACI 318 (ACI, 2005).

The following sections summarize the important elements of the 2005 AISC Seismic Provisions.

Part I: Structural Steel Buildings—Provisions

The first four sections of Part I of the provisions integrate the technical provisions that are presented in the following sections with the AISC *Specification for Structural Steel Buildings* (AISC, 2005a), the applicable building code (ABC), and other applicable national standards (for example, ASCE and ASTM). The provisions are intended to apply to buildings that are classified in the ABC as seismic design category D (or equivalent) and higher or when required by the engineer of record. In other words, the AISC Seismic Provisions are to be incorporated on *all* buildings in the higher seismic design categories. In the lower seismic design categories (A through C, as defined in ASCE 7 or the ABC), the engineer has a choice. He/she may either design the system for an R factor of 3 and design the system solely using the 2005 AISC *Specification for Structural Steel Buildings* or design the system using the AISC Seismic Provisions using the higher R factor. It should be noted that in the lower seismic design categories, the engineer cannot use the higher R factor without also designing the system to meet the ductility and detailing requirements of the AISC Seismic Provisions. In addition, it should be noted that the provisions have been specifically developed for building design. Non-building structures with building-like characteristics are also included in the scope. The Commentary to the Provisions states the following: “The Provisions, therefore, may not be applicable, in whole or in part, to non-building structures. Extrapolation of their use to non-building structures should be done with due consideration of the inherent differences between the response characteristics of buildings and non-building structures."

Section 5 of the Provisions is a completely new section that defines the expectations of the project documents to be prepared by various project participants. Much of this section was taken from the recommendations of FEMA 353 and was developed in conjunction with the American Welding Society (AWS) D1.8 SC 12 committee. Design drawings and specifications are required to provide designation of all elements of the seismic load resisting system (SLRS), demand critical welds and protected zones, the configuration of connections, welding requirements, etc. Shop drawings are required to provide similar information to verify that the design intent is understood by the fabricator. Similar requirements are placed on the erection drawings for that phase of the work. Welding requirements are presented in Appendix W.

Section 6 of the Provisions deals with the base materials to be used in seismic applications. In general, no special limitations are placed on which base materials are deemed to be acceptable for seismic applications. One limitation that is made is that the specified minimum yield stress, F_y, is limited to 50 ksi except for columns where inelastic behavior is not expected and for lower ductility systems (OMF and OCBF) that allow the use of 55-ksi material. In addition, this section requires that any member of the seismic system that has thick elements (2 in. or thicker for plate materials and 1½ in. or thicker for rolled shapes), have a minimum level of Charpy V-Notch (CVN) toughness to help ensure ductile behavior of these members. Perhaps the most important part of this section is the requirement to consider the expected yield strength and expected tensile strength in the determination of the required strength (Section 6.2). This is necessary because in seismic design one of the key goals is to focus the primary inelastic action in the frame to certain key elements that are specifically designed and detailed for this purpose. It is therefore important to have the best estimate possible of the actual yield and tensile strengths (as opposed to the ASTM specified minimum values) of all the members in the system. For all base materials, Table I-6-1 specifies a term, R_y, that when multiplied by the nominal yield strength, F_y, results in the expected yield strength of the material. A second term, R_t, multiplied by the minimum nominal tensile strength, F_u, results in the expected tensile strength of the material. Other sections in the provisions define when the R_y and R_t terms are to be used in determining the required strength of the members. This section also has a statement to clarify that when both the required strength and the available strength are calculated on the same member or connecting element, R_y and R_t can also be applied to the available strength.

Section 7 of the provisions addresses the design of connections, joints, and fasteners in the SLRS. The section begins with a general statement that all connections should be configured so that a ductile limit state controls the strength of the connection. All bolts are to be pretensioned, high strength, with faying surfaces prepared for Class A or better slip-critical joints. But, it is permitted to design the bolts in bearing. This apparent contradiction between design approach and surface preparation is presented because it is felt that avoidance of bolt slip in smaller earthquakes would be desirable, but that in the design event, the bolts would go into bearing. Another important requirement is that bolts and welds are not allowed to share the same force in a joint. This is more stringent than for typical steel design because of the expectation that the joints will be subjected to yield level forces and that the difference in stiffness between welds and bolts may not allow them to completely share the load. Standard holes are to be used, except short-slotted holes are allowed when placed perpendicular to the line of force, again to limit the chance for excessive deformation due to bolt slip. For brace diagonals, oversized holes may also be used, if they are in one ply of slip-critical joints.

Section 7 also addresses the requirements for welds in the seismic load resisting system. All such welds must be made with filler metals that have a minimum CVN toughness of 20 ft-lb at minus 0 °F as demonstrated by AWS classification or manufacturer certification. This change from the previous temperature of –20 °F was made to be more consistent with the FEMA 353 recommendations. To ensure proper performance at operating temperatures, additional toughness requirements are placed on the demand critical CJP groove welds in various systems (for example, welds of beam flanges to columns, column splices, and welds of beam webs to column flanges). The additional requirement is that a CVN toughness of 40 ft-lb at 70 °F be provided for a wide range of test conditions. The range of test conditions is presented in Appendix X of the Provisions. Section 7 also defines the term "protected zone" and alerts the engineer that discontinuities in the members of the SLRS must be avoided to limit the chance for premature, brittle fracture of the members. Specific detailing requirements for continuity plates are also provided in this section.

General member design requirements are presented in Section 8 of the provisions. The section begins with Table I-8-1 providing the limiting width-thickness ratios for compression elements of members in the SLRS. It should be noted that these ratios are somewhat more restrictive than those presented in Table B4.1 of the 2005 AISC *Specification for Structural Steel Buildings* to reflect the expected inelastic demand on these members. The majority of the remainder of this section focuses on column design. Column demands are limited to help ensure that the potential for column failure is minimized, due to the potentially catastrophic effects of such a failure. Similar limitations are also placed on column splices and column bases. In addition, the splices in columns that are *not* part of the SLRS also have special requirements. This is the only reference to members that are not part of the SLRS in the document and is provided because studies conducted as part of the FEMA/SAC project and other research indicated that continuity of these columns significantly improved the seismic performance of steel frames in severe seismic events. The final paragraphs of Section 8 address the design of steel H-piles used as part of building foundations.

The next three sections of the provisions address the requirements for the design of moment resisting frame buildings. The damage to this class of buildings caused by the 1994 Northridge earthquake caused extensive research on this system that resulted in a number of significant changes to these sections. SMF, addressed in Section 9, are intended to have the most ductile response and have been assigned the highest *R* factor. Because of the damage caused in the Northridge earthquake, SMF connections must be demonstrated to be capable of performing through a tested interstory drift of 0.04 radian based on a standard cyclic testing protocol. Demonstration of this capacity can be accomplished by one of the following means:

1. Using a connection prequalified for use in an SMF in accordance with ANSI/AISC 358. This document was developed by the AISC Connection Prequalification Review Panel (CPRP). In their first edition of this document, the AISC CPRP included prequalification requirements for the reduced beam section connection and two types of end plate connections. It is hoped that eventually all widely used connections will be prequalified by the AISC CPRP to simplify the project approval process for this class of buildings.

2. Using a connection prequalified for use as an SMF in accordance with Appendix P of the Provisions. This appendix establishes minimum requirements for prequalification of SMF, IMF, and link-to-column connections in eccentrically braced frames (EBF). A connection prequalification review panel (CPRP) is to be established that will review all test results and other data to assure that the connection can meet the required interstory drift level. The CPRP will set the limits of prequalification for each connection type. As noted earlier, AISC has established a CPRP that is presently reviewing and developing prequalification limits for a number of connections. Other CPRPs may be established, but would need to be approved by the authority having jurisdiction.

3. Providing qualifying test results in accordance with Appendix S of the Provisions. Appendix S addresses how such tests are to be conducted and demonstrated to be adequate for the proposed design. This appendix specifies that the test subassemblage match the prototype as closely as practical and defines the essential test variables that must be considered such that the connection design, detailing, construction features, and material properties are consistent with that proposed for the building. The test results can be taken from tests reported in the literature, or from tests performed specifically for the project under consideration.

In addition to having deformation capacity demonstrated by testing, the shear connection of SMFs must be designed for the gravity shear force plus the shear generated by the formation of plastic hinges at each end of the beam.

The design of the panel zone capacity is intended to be consistent with that provided in the qualifying connection tests. In addition, the panel zone must have an expected strength that is adequate to provide an approximately “balanced” yielding condition between the beams and the panel zone, since it is believed that this will result in good performance. Another important consideration for SMF design is the so-called strong column–weak beam provision. This provision is provided to prevent weak story conditions from occurring in this system, by requiring that the design confirm that the moment capacity of the columns exceed that of the beams framing into the SMF connections. Exceptions are provided for lightly loaded columns and conditions where individual connections that do not meet the strong column–weak beam concept contribute a relatively minor amount to the capacity of the moment frame at the level under consideration.

Section 9.8 of the Provisions addresses the out-of-plane stability of the beams, columns, and connections in SMF systems. Provision of this stability is obviously critical to such systems that are expected to undergo significant inelastic response in the design earthquake. Both strength and stiffness requirements are provided. For beams, the provisions required that bracing be provided near all expected plastic hinge locations. The design force to be considered is 6% of the expected beam flange capacity.

The final requirement for SMF systems is that the column splices be designed to develop the full flexural capacity of the smaller column, and that the shear connection be strong enough to develop a plastic hinge at one end of the column. This stringent requirement on column splices resulted from extensive analytical studies demonstrating that large moments, on the order of the yield capacity of the columns, can be developed over the height of the columns in severe earthquakes.

The requirements for IMF systems are presented in Section 10. Like SMF, these systems must have their moment connections qualified by connection testing in accordance with ANSI/AISC 358, Appendix P or Appendix S. The qualifying interstory drift limit for these connections is reduced to 0.02 radian to reflect the more limited ductility demands expected to be placed on these systems. It should be noted that ASCE 7-05 (ASCE, 2005) severely limits the use of these systems in the higher seismic design categories. Other than the requirement for connection qualification by testing and more restrictive lateral bracing requirements, the design of these systems is generally performed to be in accordance with the 2005 AISC *Specification for Structural Steel Buildings.*

OMF systems are permitted to be designed without being based on connection testing. The connection must have a strength of 1.1 times the expected strength of the connected members, in an effort to force the inelastic action into the members and away from the connections. This section provides a number of connection detailing requirements to promote ductile performance of the connections. Specific requirements are provided for continuity plates, weld backing and run-off tabs, weld access holes, etc. OMFs are typically used in light metal building and small building applications in the higher seismic design applications.

The design requirements for STMF systems are presented in Section 12. This system was developed from the results of a series of testing and analytical research projects at the University of Michigan (Goel, 1992; Hassan and Goel, 1991). These provisions define a special segment of the truss that is intended to be the location of the inelastic behavior in the system. All other members in the frame are designed to be able to develop the capacity of the special segment. This design concept is parallel to EBF design, where all other members are designed to be strong enough to force yielding into the link beams. Both vierendeel and X-braced special segment panels are allowed. The requirements also provide lateral bracing requirements similar to those required for SMF systems to ensure out-of-plane stability.

SCBF design requirements are presented in Section 13. The design concept for SCBF systems is that the diagonal braces should buckle and dissipate energy in the design earthquake. Special provisions are included to improve the ductility of the system. For example, the orientation of bracing in all frame lines must be such that there is approximately the same number of braces in compression and tension. In addition, there are strict limits on the width-thickness ratios and stitching requirements for built-up brace members. Bracing connections in SCBF must be designed to develop the full tensile capacity of the members or the maximum force that can be delivered to the brace by the rest of the system. Full flexural strength must also be provided in the bracing connections, unless the connection includes a gusset plate that will yield in such a manner to allow the ductile post-buckling behavior of the braces. Special limitations are provided for V-type and inverted V-type bracing to reflect the potentially undesirable behavior of these bracing configurations. K-braced frame configurations are not permitted in SCBFs. Column splices in SCBF are required to develop a shear capacity of approximately 50% of the member capacity to reflect the substantial demands on these elements when subjected to severe earthquake ground motions.

Like OMFs, OCBF systems (Section 14) have severely limited applications in high seismic design categories due to their limited ductility. The provisions also place limitations on the use of V-type and inverted V-type bracing. Connections in OCBFs are designed including the amplified seismic load. The previous requirement to design the members in OCBFs for the amplified seismic load was removed when the *R* factor was reduced in ASCE 7-05.

EBF systems are addressed in Section 15. As noted earlier, the basic intent of EBF design is to result in a system where the diagonal braces, columns, and beams outside the link beams remain essentially elastic under the forces that can be generated by the fully yielded and strain-hardened link beams. Because of their importance to the performance of the system, a good deal of this section is devoted to the design of the link beam elements. There are strict limits placed on width-thickness ratios for these elements to ensure proper inelastic performance. The link beams can be designed to yield in shear or flexure, or a combination of both. Laboratory testing has demonstrated that properly designed shear yielding links can undergo a link rotation angle of 0.08 radian. Such links are provided with closely spaced web stiffeners to delay web buckling. Significant strain hardening (on the order of 50% of the nominal shear yielding capacity of the link section) develops in such properly braced links. This strain hardening must be considered in the design of the rest of the frame members. Moment yielding links are designed to undergo a link rotation angle of 0.02 radian, which is consistent with SMF systems. Interpolation is allowed for links with a length that results in a combination of shear and flexural yielding. Web stiffening requirements are also modified for flexural yielding links. Because of the high local deformation demands, link-to-column connections must be demonstrated by testing similar to SMFs, 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. As with SMF and STMF systems, there are significant lateral stability bracing requirements for EBF systems. Lateral bracing is required at both ends of all link members and along the remainder of the beam to ensure that stability is provided. As noted earlier, the design of other members in the system and all the connections between the members, are required to have a capacity that is sufficient to develop the fully strain hardened link beams. Column capacities are not required to develop the simultaneous yielding and strain hardening of all links in the system.

Section 16 addresses the new system, buckling-restrained braced frames (BRBF). The BRBF system was originally developed in Japan and has recently been used on a number of projects on the West Coast. The key new feature of this system is that it relies on a brace element that is restrained from overall member buckling, thereby significantly increasing the energy dissipation of the system over that of a traditional CBF system. The seismic design provisions for the BRBF system were developed by a task force comprised of members from the SEAOC Seismology Committee and AISC Task Committee 9 on Seismic Design. The provisions were originally presented in the 2003 NEHRP Provisions (FEMA, 2003). The requirements for testing of the brace elements are specified in this Section and Appendix T. As with EBF systems, the provisions intend that the connections and other members in the BRBF system remain essentially elastic at the full capacity of the bracing elements. Connection design requirements recognize the fact that the braces are likely to be stronger in compression than tension. It should also be noted that because of the better energy dissipation characteristics of the bracing elements in a BRBF, the bracing configuration limitations are not as strict as those imposed on SCBF frames.

The SPSW system has been used on a number of buildings in high seismic regions dating back to the early 1970s. Renewed interest in this system developed in the 1990s in Canada as the result of a series of research projects at the University of British Columbia and the University of Alberta. Figure 1 shows the ductile response from a wall tested as part of this research. The key feature of this system is the ability of the thin web shear panels to develop tension field action that can yield in a ductile manner and dissipate large amounts of energy. The National Building Code of Canada has design provisions for this system based on the results of this research. Additional research on this system is presently being conducted at the University of California at Berkeley. The design provisions were initially developed by a BSSC TS6 task committee based on both Canadian and U.S. research efforts. They were first published in the 2003 NEHRP Provisions (FEMA, 2003).

Section 17 presents the SPSW design requirements. The anticipated performance is controlled by the web members. Since the design of SPSW systems is based on the use of relatively thin plates, tension field action (similar to a plate girder) develops in the web members under lateral loading. Like other systems, the other elements in the frame are designed to remain essentially elastic for the capacity of the webs. Limitations on configuration, width-thickness ratios, etc., are provided to be consistent with the successful test results.

The final section of Part I addresses quality assurance provisions. A comprehensive quality assurance plan is required to demonstrate that the intent of the structural design is met in the construction. A new Appendix Q has been provided to delineate all of the requirements related to quality. Requirements for both quality control to be provided by the contractor and quality assurance are presented. Inspection requirements for both visual and nondestructive evaluation (NDE) inspections of welds are presented in tabular form, based on the recommendations presented in FEMA 353. This section has also been developed in conjunction with the AWS subcommittee on seismic design. A similar table for bolted connections is also provided.

Part II: Composite Structural Steel and Reinforced Concrete Buildings

Part II of the AISC Seismic Provisions addresses the design of composite systems of structural steel and reinforced

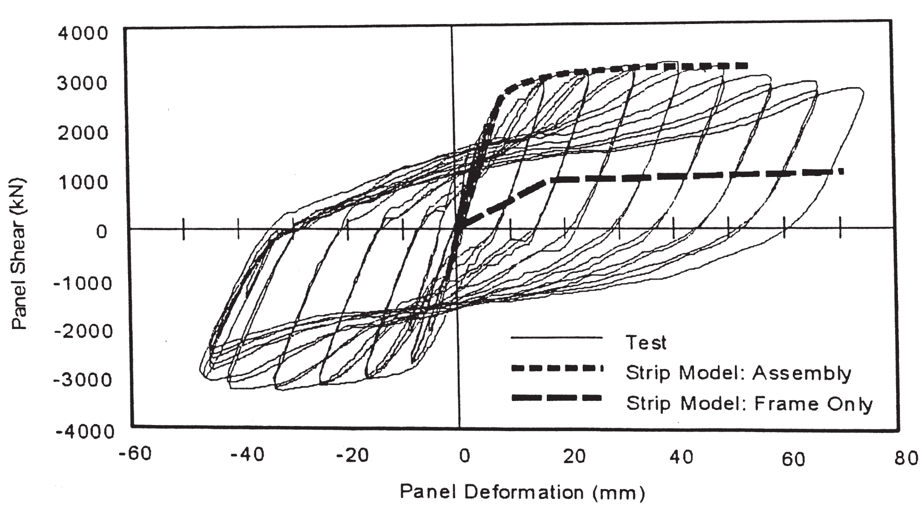

Fig. 1. Steel plate shear wall test results (Driver, Kulak, Kennedy, and Elwi, 1997).

concrete. The Part II Provisions are based on the 1994 NEHRP Provisions (FEMA, 1994) and subsequent modifications made in the 1997 NEHRP Provisions (FEMA, 1997a). Since composite systems are assemblies of steel and concrete components, ACI 318 (ACI, 2005) forms an important reference document for Part II. Consistent with Part I, Part II now addresses both LRFD and ASD approaches in a unified manner.

The available research demonstrates that properly detailed composite members and connections can perform reliably when subjected to seismic ground motions. However, there is at present limited experience with composite building systems subjected to extreme seismic loads, and many of the recommendations are necessarily of a conservative and/or qualitative nature. Composite connection details are illustrated throughout the Part II Commentary to convey the basic character of the composite systems. The design and construction of composite elements and systems continues to evolve in practice. With further experience and research, it is expected that these Provisions will be better quantified, refined, and expanded.

It is generally anticipated that the overall behavior of composite systems will be similar to that for the counterpart structural steel systems or reinforced concrete systems and that inelastic deformations will occur in conventional ways, such as flexural yielding of beams in fully restrained (FR) moment frames or axial yielding and/or buckling of braces in braced frames. However, differential stiffness between steel and concrete elements is more significant in the calculation of internal forces and deformations of composite systems than for structural steel or reinforced concrete only systems. When systems have both ductile and nonductile elements, the relative stiffness of each should be properly modeled; the ductile elements can deform inelastically while the nonductile elements remain nominally elastic.

The Part II provisions begin with a treatment of composite elements. The requirements for design of composite slabs and beams are followed by an extensive treatment of composite column elements. The requirements combine the requirements of Part I of the Provisions with the main specification, ACI 318 requirements for reinforced concrete members, and the results of composite construction research. The next section addresses the design of connections between composite elements. The use of composite connections often simplifies some of the special challenges associated with traditional steel and concrete construction. For example, compared to structural steel, composite connections often avoid or minimize the use of field welding, and compared to reinforced concrete, there are fewer instances where anchorage and development of primary beam reinforcement is a problem. In most composite structures built to date, engineers have designed connections using basic mechanics, equilibrium, existing standards for steel and concrete construction, test data, and good judgment. The provisions in this Section are intended to help standardize and improve design practice by establishing basic behavioral assumptions for developing design models that satisfy equilibrium of internal forces in the connection for seismic design. Connection examples, such as that shown in Figure 2, are also included.

The remaining sections of Part II address the design of various composite systems. These sections are presented in parallel with those in Part I and generally have R factors and system application limitations similar to the comparable structural steel systems. There are composite SMF, IMF, and OMF system requirements. In addition, there is a composite partially restrained moment frame (C-PRMF) system identified that has connection details similar to those shown in Figure 2. For braced frame systems, there are two concentrically braced and one eccentrically braced system addressed, similar to Part I of the provisions. In addition to the frame systems, Part II identifies a number of composite systems that have wall elements as the primary vertical elements in the SLRS. Two types of composite reinforced concrete wall

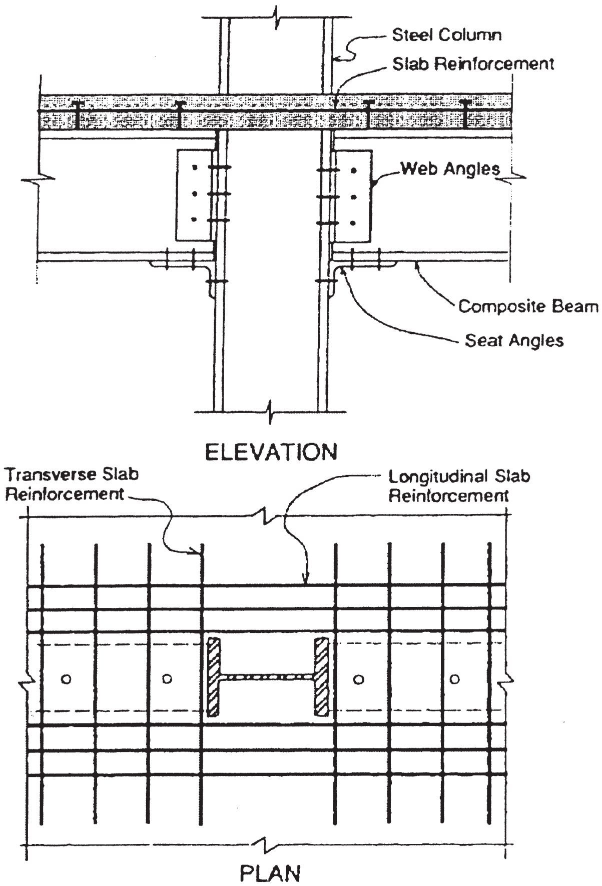

Fig. 2. Example of composite partially restrained connection.

systems with structural steel elements (generally acting as part of the boundary elements, as shown in Figure 3) are addressed. One is denoted as ordinary and the other special, parallel to the reinforced concrete wall systems in ACI 318 and ASCE 7. The final wall system is a composite steel plate shear wall system. For each system, the Provisions present specific requirements for the design of the various members and connections.

ANTICIPATED FUTURE DESIGN PROVISIONS FOR STEEL STRUCTURES

The experiences of the past decade have demonstrated that continuous attention should be paid to ensure the seismic design provisions for steel building structures remain as current as possible. A systematic process has been established to efficiently accomplish this goal. This process relies on the AISC Task Committee 9 on Seismic Design to develop specific code provisions for the various structural steel systems that will then be balloted through the AISC Committee on Specifications. As an American National Standards Institute (ANSI) accredited consensus activity, this balloting and the subsequent document will be a standard that can be adopted by national building codes by reference. The BSSC TS6 subcommittee is focusing primarily on the introduction of new systems and the proper and consistent application of the design coefficients such as R, C_d, and Ω_o. This will allow such new systems to be used on a provisional basis so that actual building applications can be used to test the efficacy of the provisions. As experience is gained with these new systems, it is expected that they would then be able to be incorporated into the AISC Seismic Provisions and, therefore, future editions of the building code. In addition, future improvements will be made to the existing provisions as experience with their use increases and as new information is developed from ongoing research programs sponsored by AISC, the National Science Foundation, FEMA, and others. At the present time, the next edition of the AISC Seismic Provisions is scheduled for completion in 2010.

The American Welding Society (AWS) also has a seismic subcommittee that has completed a document that will specifically address welding issues that relate specifically to seismic applications. This document, AWS D1.8 (AWS, 2005), is an important link to the AISC Seismic Provisions, helping to ensure that the design intent is accomplished on constructed projects. In the future, the AISC Seismic Provisions will defer to AWS D1.8 on many welding-related issues, allowing some of the information that is presently included in the AISC Seismic Provisions (for example, Appendix X and W) to be eliminated.

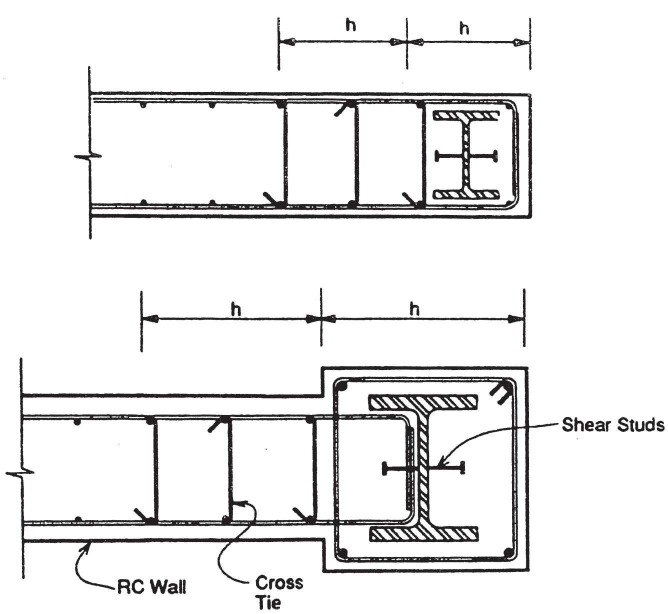

Fig. 3. Composite shear wall boundary element.

CONCLUSIONS

Over the last 10 years, a rational and efficient process and system has been instituted to incorporate the latest developments in seismic design of steel structures into building code provisions. This system relies on the coordinated efforts of AISC, BSSC, and AWS committees. The process provides a single point of responsibility for the development of these provisions, thus eliminating duplicate efforts and, more importantly, the development of competing documents that would result in minor differences that would undoubtedly result in major confusion in their application by practicing engineers. The 1997 AISC Seismic Provisions were supplemented twice to keep them as up to date as possible. The publication of the AISC Seismic Provisions in 2002 allowed for this edition to be incorporated into both the 2002 NFPA 5000 model building code and the 2003 IBC. The more recent publication of the 2005 AISC Seismic Provisions has allowed for that document to be referenced in the 2007 IBC. Ultimately, the seismic design of all steel buildings in the United States will be governed by this document, allowing engineers to develop their designs in a consistent fashion, no matter what the jurisdiction. This will lead to better designs and better performance by steel buildings in future earthquakes. The major changes, including the unified format approach, to the 2005 AISC Seismic Provisions were summarized. These changes will continue the ongoing process of improving structural steel seismic design standards that should result in improved steel construction throughout the United States and other countries around the world that adopt this standard.

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