AISC SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

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ABSTRACT :

The American Institute of Steel Construction (AISC) document "*Seismic Provisions for Structural Steel Buildings*", has become the reference document for seismic design of steel structures throughout the United States. Since its initial publication in 2000, the International Building Code has incorporated these provisions by reference. Since their 1997 publication, the AISC Seismic Provisions have been updated on a regular basis in order to incorporate new developments from the FEMA/SAC project on moment frames and other work in this area. The latest revision culminated in 2005 with the publication of a completely new set of provisions that is in the same unified format as the main AISC design specification. In addition, this edition adds two new structural systems (Buckling Restrained Braced Frames and Steel Plate Shear Walls) and makes reference to a new AISC standard for the pre-qualification of seismic moment connections that was also completed in 2005. Major improvements to the quality control sections, based on FEMA 353, are also included for the first time. Coordinated efforts between AISC, ASCE-7 and the Building Seismic Safety Council (BSSC) will continue the process of keeping the seismic design provisions for structural steel buildings as current as possible. This paper will summarize the 2005 AISC Seismic Provisions and the use of the new moment connection pre-qualification standard. It will also address work that is underway to update the standard for the 2010 edition of the AISC Seismic Provisions.

KEYWORDS: Structural Steel, Composite Construction, Seismic Design Provisions, AISC

1. INTRODUCTION

The 1994 Northridge earthquake resulted in an unprecedented level of interest in the seismic performance of steel frame structures. As a result of these efforts, significant modifications to the U.S. seismic design provisions for steel structures have taken place. The AISC Seismic Provisions were almost completely re-written 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 NFPA 5000 and the 2003 IBC, incorporating information from the final FEMA/SAC recommendations presented in FEMA 350 through 355. The 2005 Seismic Provisions (ANSI/AISC 341-05) were developed so that the new main AISC Specification (ANSI/AISC 360-05, also completed in 2005) could be used us a primary reference and were referenced in the 2006 IBC. The contents of the previous editions of the AISC Seismic Provisions will be briefly summarized in this paper. The paper will also focus on the 2010 Edition of the AISC Seismic Provisions that is currently being developed for incorporation into the 2012 IBC.

2. THE 2005 AISC SEISMIC PROVISIONS

A major change to the 2005 AISC Seismic Provisions is in format. 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 Brace Frame (BRBF) and the Special Plate Shear Wall (SPSW). Both of these systems are included in the 2005 Seismic Provisions. The following paragraphs summarize the important elements of the 2005 AISC Seismic 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 Unified Specification, the Applicable Building Code (ABC) and other applicable national standards (ASCE, ASTM, e.g.). 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 Unified Specification, 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 may not 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 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 was properly interpreted 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. In addition, this section requires that any member of the seismic system that has thick elements (2 inches or thicker for plate materials and 1 ½ inches or thicker for rolled shapes), have a minimum level of Charpy Vnotch (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 that when 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 pre-tensioned, high strength, with faying surfaces prepared for Class A or better Slip-Critical joints. 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 0F as demonstrated by AWS classification or manufacturer certification. To ensure proper performance at operating temperatures, additional toughness requirements are placed on the Demand Critical CJP welds in various systems (welds of beam flanges to columns, column splices, and welds of beam webs to column flanges, e.g.). The additional requirement is that a CVN toughness of 40 ft. lbf. at 70F 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 that presents 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 ANSI/AISC 360-05 to reflect the expected inelastic demand on these members. The majority of the rest of this section focuses on column design. Column demands are limited to help ensure that the potential for column failure is minimized. Similar limitations are also placed on column splices and column bases. In addition, the splices in columns that are <u>not</u> 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. 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 radians, 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 as a SMF in accordance with ANSI/AISC 358. This document was developed by the AISC Connection Prequalification Review Panel (CPRP).

2) Using a connection prequalified for use as a SMF in accordance with Appendix P of the provisions. This appendix establishes minimum requirements for pre-qualification 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 levels.

3) Providing qualifying tests 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 in order to assure that the connection design, detailing, construction features and material properties are consistent with that proposed for the building. Such 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 SMF's 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 help assure that weak story conditions will not occur 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.

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 expected to undergo significant inelastic response in the design earthquake. Both strength and stiffness requirements are provided. For beams, the provisions require that bracing be provided near all expected plastic hinge locations. The design force to be considered is six per cent 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 that demonstrated 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 radians to reflect the more limited ductility demands expected to be placed on these systems. It should be noted that ASCE 7-05 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 in accordance with the Unified Specification.

OMF systems may be designed without being based on connection testing. The connection strength must be 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 help ensure ductile performance of the connections. Specific requirements are provided for continuity plates, weld backing and run-off tabs, weld access holes, etc. OMF's are typically used in light metal building and small building applications in the higher seismic design categories.

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 and inverted-V bracing to reflect the potentially undesirable behavior of these bracing configurations. K braced frame configurations are not permitted in SCBF's. Column splices in SCBF are required to develop a shear capacity of approximately 50 per cent of the member capacity to reflect the substantial demands on these elements when subjected to severe earthquake ground motions.

Like OMF's, 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 and inverted-V bracing. Connections in OCBF's are designed including the Amplified Seismic Load. Some specific requirements are also provided for OCBF systems over isolated bases. The previous requirement to design the members in OCBF's 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 above, 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. There are strict limits placed on width-thickness ratios for the link beams to ensure proper inelastic performance. The link can be designed to yield in shear or flexure. Laboratory testing has demonstrated that properly designed shear yielding links can undergo a link rotation angle of 0.08 radians. Such links are provided with closely spaced web stiffeners to delay web buckling. Significant strain hardening (on the order of 50 per cent 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 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. 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 SMF's, 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 above, 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 Buckling Restrained Braced Frames (BRBF) system. The key 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 requirements define the requirements for testing of the brace elements are specified in this Section and Appendix T. As with EBF systems, the provisions intend to ensure 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 BRBF's, the bracing configuration limitations are not as strict as those imposed on SCBF frames.

Section 17 presents the SPSW design requirements. The key feature of this system is the ability of the thin web shear panels forming tension field action that can yield in a ductile manner and dissipate large amounts of energy. The anticipated performance is controlled by the web members. Since the design of the 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 non-destructive 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 of the AISC Seismic Provisions addresses the design of composite systems of structural steel and reinforced concrete. These provisions have been taken from work first presented in the NEHRP Provisions for the Seismic Design of Buildings, developed by the Building Seismic Safety Council. Since composite systems are assemblies of steel and concrete components, ACI 318 (ACI, 2005) forms an important reference document for Part II.

The available research demonstrates that properly detailed composite members and connections can perform reliably when subjected to seismic ground motions. However, there is 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. It is generally anticipated that the overall behavior of the composite systems herein will be similar to that for counterpart structural steel systems or reinforced concrete systems and that inelastic deformations will occur in conventional ways, such as flexural yielding of beams in 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 only or reinforced concrete only systems. When systems have both ductile and non-ductile elements, the relative stiffness of each should be properly modeled; the ductile elements can deform inelastically while the non-ductile 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 Part I of the Provisions with AISC 360, ACI 318, 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.

The remaining sections of Part II address the design of various composite systems. These sections are presented in parallel to 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 systems 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 R/C wall systems with structural steel elements are addressed. One is denoted as an Ordinary and the other Special, parallel to the R/C 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.

3. 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 TC 9 subcommittee to develop specific code provisions for the various structural steel systems that will then be balloted through the main AISC Specifications committee. As an American National Standards Institute (ANSI) accredited consensus activity, this balloting and the subsequent document that will result will be a standards document 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, scheduled for completion in 2010, is in the development process. AWS has now completed and published D1.8 that addresses welding related issues that relate specifically to seismic applications. This document is an important link to the AISC Seismic Provisions, helping to ensure that the design intent is accomplished on the constructed projects. Since a number of the topics related to welding now in the AWS D1.8 standard, some of the information that is in the 2005 AISC Seismic Provisions (Appendix X and W, e.g.) will be removed and referenced to AWS D1.8. Additional major modifications to the 2010 AISC Seismic Provisions will likely include the following:

1. Re-formatting to integrate the composite provisions with the structural steel provisions.
2. Implementing a standard format for the design requirements of all systems.
3. Changing the lower temperature on CVN toughness to 0F to be consistent with AWS D1.8.
4. More consistent treatment of stability bracing requirements for all systems.
5. Significant increase in the completeness of the treatment of composite systems, elements and connections.
6. Improved treatment of the design parameters related to concentrically braced frames.
7. Provisions for alternate configuration of plates in special plate shear walls.
8. Consolidation of quality assurance and quality control provisions.

4. CONCLUSIONS

Over the last fifteen 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 duplicative effort, and more importantly, the development of competing documents that would result in minor differences that would undoubtedly result in major confusion in application by practicing engineers. The most recent publication of the AISC Seismic Provisions in 2005, allowed for this edition to be incorporated into both the 2006 IBC. As a result, the seismic design of all steel buildings in the United States are 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 proposed for the 2010 AISC Seismic Provisions were summarized. These anticipated changes should continue the on-going process of improving structural steel seismic design standards that should result in improved steel construction throughout the United States and other countries throughout the world that adopt this standard.

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