THE AISC SEISMIC PROVISIONS: PAST, PRESENT AND FUTURE

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BIOGRAPHY OF AUTHOR

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ABSTRACT

The American Institute of Steel Construction (AISC) document *Seismic Provisions for Structural Steel Buildings (AISC 341-05)*, 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 (IBC) 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 and other work in this area. The latest revision culminated in 2010 with the publication of a new set of provisions that includes a new format that is more consistent with the main AISC design specification (AISC 360-10). The 2010 edition of AISC 341 was developed in conjunction with ASCE 7-10, and will be incorporated into the 2012 IBC. This paper will summarize the proposed changes to 2010 AISC Seismic Provisions and will also provide an update to the most recent changes to AISC 358 *Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications*. It will also postulate on the future for structural seismic design of structural steel systems.

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, and are still in use throughout the U.S. 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 recently was completed and will be incorporated into the 2012 IBC. Finally this paper will present some thoughts on future directions that may occur in the seismic design of steel structures.

2005 AISC SEISMIC PROVISIONS (AISC 341-05)

A major change to the 2005 AISC Seismic Provisions was in format. Consistent with the changes to the main design specification, the 2005 Seismic Provisions combined ASD and LRFD into a single specification. As such, Part III in previous editions (which addressed ASD) of the Seismic Provisions was 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 were 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 integrated 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 were 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 were to be incorporated on <u>all</u> 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 <u>not</u> 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 were 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 defined 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 (FEMA, 2000) and was developed in conjunction with the American Welding Society (AWS) D1.8 (AWS, 2005). 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 dealt with the base materials to be used in seismic applications. This section required 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 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). It is 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 to ensure that the members subjected to significant inelastic behavior are well understood. 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 defined when the R_y and R_t terms are to be used in determining the Required Strength of the members.

Section 7 of the provisions addressed the design of connections, joints and fasteners in the SLRS. All bolts are to be pre-tensioned, high strength, with faying surfaces prepared for Class A or better Slip-Critical joints. Standard holes are to be used except short-slotted holes are allowed when placed perpendicular to the line of force 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 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 defined the term “Protected Zone” and alerted the Engineer that discontinuities in the members of the SLRS must be avoided to limit the chance for premature, brittle fracture of the members.

General member design requirements were presented in Section 8 of the provisions. The section begins with Table I-8-1 that presented 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 next three sections of the provisions addressed 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 pre-qualified for use as a SMF in accordance with ANSI/AISC 358.
2. Using a connection prequalified for use as a SMF in accordance with Appendix P of the provisions. This appendix established minimum requirements for pre-qualification of SMF, IMF and link-to-column connections in Eccentrically Braced Frames (EBF).
3. Providing qualifying tests results in accordance with Appendix S of the provisions. Appendix S addressed how such tests are to be conducted and demonstrated to be adequate for the proposed design. 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 was 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. Another important consideration for SMF design is the so-called “strong column-weak beam” provision. This provision was 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 addressed 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.

The final requirement for SMF systems was 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 (Section 11) 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 were presented in Section 12. These provisions defined 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. 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 were provided for V and inverted-V bracing to reflect the potentially undesirable behavior of these bracing configurations. K braced frame configurations were not permitted in SCBF's. Column splices in SCBF were 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.

EBF systems are addressed in Section 15. 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 were significant lateral stability bracing requirements for EBF systems. Lateral bracing was 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, were required to have a capacity that is sufficient to develop the fully strain hardened link beams. Column capacities were 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 were specified in this Section and Appendix T. As with EBF systems, the provisions intended 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 recognized 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 presented 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 was also been developed in conjunction with the AWS subcommittee on seismic design. A similar table for bolted connections was also provided.

Part II of the AISC Seismic Provisions addressed the design of composite systems of structural steel and reinforced concrete. These provisions were 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 were 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 began with a treatment of composite elements. The requirements for design of composite slabs and beams were followed by an extensive treatment of composite column elements. The requirements combined Part I of the Provisions with AISC 360, ACI 318, and the results of composite construction research. The next section addressed the design of connections between composite elements. The provisions in this Section were 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 addressed the design of various composite systems. These sections were 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 were Composite SMF, IMF and OMF systems requirements. In addition, there was a Composite Partially Restrained Moment Frame (C-PRMF) system. For braced frame systems, there were two concentrically braced and one eccentrically braced system addressed, similar to Part I of the provisions. In addition to the frame systems, Part II identified a number of composite systems that have wall elements as the primary vertical elements in the SLRS. For each system, the provisions presented specific requirements for the design of the various members and connections.

AISC 341-10

The next edition of the AISC Seismic Provisions, was recently completed early in 2010. 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 was in the 2005 AISC Seismic Provisions (Appendix X and W, e.g.) has been removed and referenced to AWS D1.8. Some of the most significant modifications to this edition of these *Provisions* are related to format. The organization of the chapters has been changed to be more consistent with that of AISC 360. In the 2005 edition, AISC 341 separated the requirements for structural steel buildings from that of composite structural steel/reinforced concrete construction into two Parts. The 2010 edition of these *Provisions* eliminates Part II, combining all systems together. In addition, each structural system is presented in a unified manner with parallel headings that will ease comparison of requirements between systems and application of the document.

The basic organization of AISC 341-10 is listed below. For comparison purposes, the parallel section heading in the 341-05 version of is listed in parentheses, with the former Part II section listed with “II”:

A. GENERAL REQUIREMENTS
   - A1. Scope (1, II-1)
   - A2. Referenced Specifications, Codes and Standards (2, II-2)
   - A3. Materials (6.1 through 6.3, 7.3b, II-5.1 through 5.2)
   - A4. Structural Design Drawings and Specifications (5.1 through 5.2, II-5.1 through 5.2 and 18)

B. GENERAL DESIGN REQUIREMENTS
   - B1. General Seismic Design Requirements (New)
   - B2. Loads and Load Combinations (4.1)
   - B3. Design Basis (3)
   - B4. System Type (New)

C. ANALYSIS
   - C1. General Requirements (New)
   - C2. Additional Requirements (New)
   - C3. Nonlinear Analysis (New)

D. GENERAL MEMBER AND CONNECTION DESIGN REQUIREMENTS
   - D1. Member Requirements (8.2 through 8.3, II-6.2 through 6.5)
   - D2. Connections (7, 8.4a, 8.5, and II-7)
   - D3. Deformation Compatibility of Non-SFRS Members and Connections (New)
   - D4. H-Piles (8.6)

E. MOMENT FRAMES
   - E1. Ordinary Moment Frames (11)
   - E2. Intermediate Moment Frames (10)
   - E3. Special Moment Frames (9)
   - E4. Special Truss Moment Frames (12)

E5. Ordinary Cantilever Column Systems (New)
E6. Special Cantilever Column Systems (New)

F. BRACED-FRAME AND SHEAR-WALL SYSTEMS
F1. Ordinary Concentrically Braced Frames (14)
F2. Special Concentrically Braced Frames (13)
F3. Eccentrically Braced Frames (15)
F4. Buckling-Restrained Braced Frames (16)
F5. Special Plate Shear Walls (17)

G. COMPOSITE MOMENT FRAME SYSTEMS
G1. Composite Ordinary Moment Frames (II-11)
G2. Composite Intermediate Moment Frames (II-10)
G3. Composite Special Moment Frames (II-9)
G4. Composite Partially Restrained Moment Frames (II-8)

H. BRACED-FRAME AND SHEAR-WALL SYSTEMS
H1. Composite Ordinary Concentrically Braced Frames (II-13)
H2. Composite Special Concentrically Braced Frames (II-12)
H3. Composite Eccentrically Braced Frames (II-14)
H4. Composite Ordinary Shear Walls (II-15)
H5. Composite Special Shear Walls (II-16)
H6. Composite Special Plate Shear Walls (II-17)

I. FABRICATION AND ERECTION
I1. Shop and Erection Drawings (5.2 through 5.3, Appendix W2.2 through W2.3)
I2. Fabrication and Erection (7.2 through 7.5, Appendix W5.5)

J. QUALITY CONTROL AND QUALITY ASSURANCE
J1. Scope (Appendix Q1)
J2. Fabricator and Erector Documents (Appendix Q3)
J3. Quality Assurance Agency Documents (Appendix Q4)
J4. Inspection and Nondestructive Testing Personnel (Appendix Q2)
J5. Inspection Tasks (Appendix Q5)
J6. Welding Inspection and Nondestructive Testing (Appendix Q5.1 through Q5.2)
J7. Inspection of High-Strength Bolting (Appendix Q5.3)
J8. Other Steel Structure Inspections (Appendix Q5.4)
J9. Inspection of Composite Structures (New)
J10. Inspection of Piling (New)

K. PREQUALIFICATION AND CYCLIC QUALIFICATION TESTING PROVISIONS
K1. Prequalification of Beam-to-Column and Link-to-Column Connections (Appendix P)
K2. Cyclic Tests for Qualification of Beam-to-Column and Link-to-Column Connections (Appendix S)
K3. Cyclic Tests for Qualification of Buckling Restrained Braces (Appendix T)

For each structural system listed in Chapters E through H, a consistent format has been established to provide parallel sections that will facilitate the comparison of requirements between structural systems and to improve the consistency, clarity and completeness of how each structural system treats all aspects of the seismic design and detailing. For each system, the following subsections are included:

1. Scope - Primarily charging language for the remainder of the section.
2. Basis of Design – Describes the basic intended response characteristics and performance of the SFRS, including the expected amount of inelastic deformation, which elements are expected to undergo inelastic demands, and which elements are intended to remain essentially in the elastic range during seismic response.

3. Analysis – This is a new section that describes any special analyses that are required beyond that traditionally mandated by ASCE 7. The section is often used to define how to develop the distribution of forces in order to accomplish the capacity design of the systems elements to meet the performance expectations described in the Basis of Design section.
4. System Requirements – This section addresses design and/or detailing of elements of the system that impact the design of other members and the overall system.
5. Member Requirements – Describes element specific design requirements, and specifies location of Protected Zones.
6. Connections – Specifies all Demand Critical welds, column splice requirements and other critical connection requirements between elements of the SFRS.

A number of significant technical modifications have also been made in the 2010 edition of these *Provisions*, including the following:

- <u>Clarifying the intended combination of this document with the provisions of ACI 318 for composite construction systems.</u> Section A1 of the Provisions states that ACI 318 is to be used for the design of reinforced concrete components of composite systems, and that for steel and composite elements in these systems, LRFD design shall be used for steel and composite elements. It is not appropriate to mix ASD design of steel or composite elements with the design of concrete elements that must be based on LRFD per ACI 318.
- <u>Establishing a new chapter on analysis requirements that applies to all systems.</u> New Chapter C notes that for elastic analysis, cracked section properties should be used for concrete elements in composite systems. It also provides a pointer to system specific analysis requirements in Chapters E through H, and invokes the provisions of ASCE 7 for systems designed using nonlinear analysis procedures.
- <u>Adding terms to clearly identify the level of ductile response capable of various members in the seismic force resisting system (SFRS).</u> The new terms are "highly ductile" and "moderately ductile", provided as a means of providing more transparency to the design engineer. These terms are used to specify the limiting width-thickness ratios and stability bracing requirements for various elements of the SLRS of each system in Chapters E through H. In addition, there are specific design requirements for composite columns depending on whether or not they are designated as "highly" or "moderately" ductile. Generally, the "highly ductile" members are those designed to be the primary yielding elements, and therefore will have large inelastic demands, and commensurately stricter width-thickness, bracing, and/or detailing requirements. See Chapter D for the detailed requirements related to these terms.
- <u>Removing the requirement for demonstrating a ductile failure mode is controlling limit state for SFRS connections.</u> This was deemed to be onerous for connections, which are capacity controlled, especially for chords, collectors and splices.
- <u>Revisions to column splice requirements.</u> Section D2.5 clarifies splice locations, weld requirements and determination of required strength. Changes for splices are also located in the various system requirements in Chapters E through H.
- <u>Adding language to clarify the design of members and connections that are not part of the SFRS for deformation compatibility.</u> Section D3 is a new section that to assist in requirements from ASCE 7 related to the capacity of non lateral force resisting elements to undergo the deformations resulting from the required design story drift. It is generally felt that the flexible shear connections typical of steel framed construction are adequate to meet this requirement without detailed calculations. This is stated in a User Note, with more detailed discussion in the Commentary.
- <u>Revised OMF provisions to allow use of members other than wide-flange sections.</u> See Section E1.
- <u>Adding requirements for two cantilever column systems to be consistent with other systems in these *Provisions* and the definitions in SEI/ASCE 7-10.</u> Previous editions of ASCE 7 included a number of steel cantilever column system designations, which referenced AISC 341 for detailing provisions. This reference was relatively oblique, since no such systems were designed in AISC 341 prior to the 2010 edition. Chapter E of the 2010 edition now includes both Ordinary and Special Cantilever systems, with complete design requirements that parallel all the other systems. The requirements limit axial force demand on the cantilever columns for both systems, in order to ensure that there will be adequate displacement ductility. The special system has additional requirements for stability bracing at the top of the column, width-thickness ratios, demand critical welds, etc. ASCE 7-10 will have parallel system designations.

- Adding analysis requirements to address the inelastic response of Special Concentrically Braced Frames. Concentrically braced frames act primarily as a vertical truss to resist lateral forces through axial force in the members. This system dates back to the earliest seismic building codes in the 1950's, when elastic analysis and response was the primary design approach. Over time, the design emphasis has changed to capacity based design and inelastic performance. The design provisions for all systems developed since the 1980's (EBF, BRBF, SPSW, e.g.) have attempted to invoke this philosophy in a consistent and complete manner. Prior to the 2010 edition of AISC 341, only portions of the requirements for concentrically braced frames (OCBF and SCBF) incorporated a capacity based design approach. For SCBF systems, which are intended to have a relatively high level of ductility, the 2010 provisions now have addressed this issue with additional analysis requirements. The engineer is required to perform two separate analyses of the SFRS. The first assumes that capacity of the compression brace has not been reduced by inelastic cyclic demand. Note that this force distribution is consistent with elastic analysis, similar to that which has been done on these systems since their inception. The second analysis reduces the capacity of the compression braces to 30 percent of the original capacity, reflecting a distribution of forces that is consistent with that expected after significant buckling of the compression braces has occurred. This force distribution could be significantly different from the elastic one typically considered, and will likely result in re-sizing beams and/or columns in SCBF frames. See Section F2.3. Similar language is provided for EBF and BRBF systems, but this will not materially alter the design when compared to previous versions.
- Modifying the connection requirements for braced frame systems to verify the expected deformation demands can be accommodated. This new requirement is based on the fact that inelastic deformations in braced frames may result in drifts upwards of 2 to 3 percent. This can be accommodated by connections that can force inelastic rotation into the beam beyond the extent of the beam-column-brace connection, or via a connection that can specifically accommodate a rotation of 0.025 radians. A connection of this type has been studied by Fahnestock (Fahnestock, et. al., 2003). Similar requirements are included for SCBF, EBF and BRBF systems and composite systems with braced frames.
- Adding requirements for the use of box-shaped link beams in Eccentrically Braced Frames. Research at the University at Buffalo has resulted in new provisions that allow the use of built-up box shaped link members in EBF's (Berman and Bruneau, 2008a, 2008b). Because of their box shape, these link members would not be subject to lateral torsional buckling and therefore could find applications when link beams are located where lateral bracing cannot be placed, such as in elevator shafts. See Section F3.5.
- Adding requirements for the use of perforated plates in Special Plate Shear Walls. Research at the University at Buffalo has studied plates in SPSW with regular patterns of openings (Vian and Bruneau, 2009). These openings may be used for passing nonstructural elements such as pipes through the walls, and may be helpful in limiting the strength and stiffness of walls where only very thin plates are needed to meet the code strength requirements. Other provisions address the cutting out the corners of the plates which also provides a location for passing elements through the wall, and helps to reduce local demands at the beam-column connections. See Section F5.6.
- Added a stiffness requirement for beams in SPSW. This is similar to the requirement for column stiffness in AISC 341-05. See Section F4.4.
- Significantly increasing the detail for the design requirements of composite systems, such that they are consistent with structural steel systems. In previous editions of AISC 341, the system descriptions and level of detail devoted to the composite systems in Part II were somewhat lower than those provided for the structural steel systems. Since Part II has been eliminated and the composite systems have been incorporated into a single set of provisions, a major effort has been made to bring the level of detail up to a level consistent with the structural steel systems. To the greatest extent possible, the treatment of elements in composite systems that are similar to those in structural systems, have requirements that are as identical as possible. The reader is referred to Chapters G and H to review these new provisions.
- Incorporating AWS D1.8 by reference for welding related issues. The AISC 341-05 provisions were published prior to the initial publication of AWS D1.8. As a result, many items that were incorporated into AWS D1.8 have been removed from AISC 341-10 where overlap would have occurred. The primary elements where this occurred were in the requirements for weld filler metal requirements, welder qualifications, inspector qualifications, welder operations, etc. The delineation of responsibilities between AWS D1.8 and AISC 341-10 can be summarized by saying that AISC is responsible to identify all elements that the engineers need to specify what needs to be used, done and inspected, and AWS D1.8 specifies how the welding is to be done and inspected and who is to do the work. The following is taken

from User Notes included in Chapters I and J of the Provisions:

AWS D1.8/D1.8M was specifically written to provide additional requirements for the welding of seismic force resisting systems, and has been coordinated wherever possible with these Provisions. AWS D1.8/D1.8M requirements related to fabrication and erection are organized as follows, including normative (mandatory) annexes:

1. General Requirements
2. Reference Documents
3. Definitions
4. Welded Connection Details
5. Welder Qualification
6. Fabrication
7. Inspection

Annex A. WPS Heat Input Envelope Testing of Filler Metals for Demand Critical Welds
Annex B. Intermix CVN Testing of Filler Metal Combinations (where one of the filler metals is FCAW-S)
Annex C. Supplemental Welder Qualification for Restricted Access Welding
Annex D. Supplemental Testing for Extended Exposure Limits for FCAW Filler Metals
Annex F. Supplemental Ultrasonic Technician Testing
Annex G. Supplemental Magnetic Particle Testing Procedures
Annex H. Flaw Sizing by Ultrasonic Testing

AWS D1.8/D1.8M requires the complete removal of all weld tab material, leaving only base metal and weld metal at the edge of the joint. This is to remove any weld discontinuities at the weld ends, as well as facilitate magnetic particle testing (MT) of this area. At continuity plates, these Provisions permit a limited amount of weld tab material to remain because of the reduced strains at continuity plates, and any remaining weld discontinuities in this weld end region would likely be of little significance. Also, weld tab removal sites at continuity plates are not subjected to MT.

AWS D1.8/D1.8M Clause 6 is entitled "Fabrication", but the intent of AWS is that all provisions of AWS D1.8/D1.8M apply equally to fabrication and erection activities as described in the *Specification* and in these Provisions.

There are a large number of other technical modifications that have been made to AISC 341-10 that are too numerous to address in this paper. Many of these changes have been made to clarify existing provisions that have raised questions by practicing engineers and to increase the consistency and transparency of the document. The reader is referred to the provisions and a fully updated Commentary for more information.

THOUGHTS ON THE FUTURE DIRECTIONS FOR AISC 341

With the publication of the 2010 edition of AISC 341, the provisions have a format that is much more consistent in a number of respects. The format is now more consistent with that of AISC 360. The composite systems have been brought into the main body of the document with the deletion of Part II. Each system now has a consistent series of section headings and subheadings that eases comparison across systems and the identification of differences between the requirements of various systems. And finally, with the updating of the requirements for braced frame systems, all moderate and high ductility systems are firmly rooted in a capacity based design approach, where elements of the SFRS are specifically identified and detailed to withstand significant inelastic response and the other elements are designed to ensure that they have strength sufficient to ensure the intended inelastic performance of the overall system.

Engineers being engineers, it is clear that future editions of AISC 341 will attempt to continue to improve the usability of the provisions, and to incorporate new developments that are the result of research and possibly, actual performance of steel structures in large earthquakes. Some areas that could be updated include the following:

- The increased use of applications with high strength materials. Structural steels up to 65 ksi are incorporated into the provisions, and have seen limited applications. Higher strength materials may be useful in some future applications if they can meet the other demands for seismic performance. High strength concrete materials in composite construction could also be incorporated.
- New structural systems. Staggered truss systems have been used in non-seismic applications for mid-to-high rise applications. There has been interest expressed in extending these systems to seismic design.
- Improved element and connection design and detailing. Methods of reducing the cost of steel frame construction could be achieved through new research results. Some potential examples would be relaxing requirements for protected zones, demand critical welds, column splices, stability bracing width-thickness, etc.
- Better definition of demands on capacity controlled elements. Presently, many elements of the SFRS have their forces controlled by the capacity of ductile elements. It is not clear that a consistent margin is provided for the design of these elements in all systems. AISC has a project underway to provide this increase consistency across systems.
- More system specific detailing provisions for composite systems. As noted above, the "default" for specifying requirements in composite systems is either that required for reinforced concrete elements in ACI 318 or structural steel in AISC 341. More system specific studies could result in relaxed requirements that would encourage the use of composite systems.

Today's seismic building codes are rooted in the concept of requiring a minimum level of performance that ensures a limited risk of collapse for structures when subjected to large earthquake demands. The continued advance of computing power allows engineers to more accurately model and analyzed structural systems. It is now within reach of engineers to perform inelastic response history analyses for postulated earthquake ground motions in an attempt to simulate actual structural response. This advanced computing power allows engineers to predict structural performance in ways not possible until very recently. A framework for Performance Based Earthquake Engineering (PBEE) has developed over the last decade that provides engineers with a tool to take their seismic designs well beyond the minimum requirements of the building codes. To date, engineers have implemented PBEE in two primary ways:

1) To demonstrate equivalent seismic performance. In this approach, the engineer takes a building system that is either new, or has limitations on application for height limits or other parameters, and performs a series of detailed seismic analyses, which are likely to include a suite of nonlinear response history analyses, to demonstrate structural performance that is consistent with intent of the building code. This approach is performed under the "alternate means of compliance" language that is included in ASCE 7.
2) To demonstrate superior seismic performance that will allow immediate occupancy and/or continued functionality after a major earthquake. Some building owners require post-earthquake functionality for emergency response (hospitals, police and fire stations, e.g.) or the need for continuous operations (critical manufacturing processes, e.g.). Presently, the building codes primarily address higher performance in a relatively crude fashion, by increasing the design base shear through the use of the Importance Factor. This factor was incorporated into codes many years ago, long before the ability to performance complex nonlinear response history analyses was even contemplated. Realistically, in order to validate that a structural system will be able to provide superior seismic performance requires much more than simply increasing the design lateral forces, so engineers have used PBEE techniques to validate their design approaches.

As we look to the future of seismic design codes, and most specifically those related to steel and composite construction, the trend will undoubtedly focus on the use of more realistic analysis to expand the use of existing structural systems and the incorporation of new structural systems. It will also focus on the desire of building owners to minimize damage, repair costs and downtime after major seismic events in ever increasing numbers of structures. This philosophy is consistent with the Sustainable Design movement, since the ability of structures to minimize the need to repair and reconstruction after an earthquake makes them more sustainable. Another term that is becoming a guidepost for the future of seismic design is "Resilient Communities". Resilient Communities will have the ability to respond to major seismic events and avoid calamitous results such as that suffered by New Orleans after Hurricane Katrina or Haiti after the earthquake in January, 2010. Having important structures remain in operation is a key element of providing resiliency. It is clear, that the seismic performance expectations of building owners and society-at-large are increasing, and engineers will be called upon to use all of the tools at their disposal to achieve these demands.

With the previous discussion as context, the following are some areas where advances in steel seismic design and construction techniques will likely occur:

- Definition and demonstration of new (and existing?) systems. FEMA P695 (FEMA, 2009), also known as ATC 63, has established guidelines for the demonstration of system performance of new systems. The process includes an extensive number of nonlinear response history analyses. It is expected that all newly developed systems will go through this type of analyses to validate their design parameters and limitations. In addition, existing systems are being studied using the same approach. Someday, the "level playing field" will truly be level, and demonstrated by quantitative rather than qualitative measures.
- Increased application of protective elements to increase structural damping. The incorporation of deliberately high levels of structural damping through viscous, visco-elastic, or friction elements is gaining increased application. These elements absorb large amounts of earthquake input energy, thereby protecting other structural and nonstructural elements from damaging forces and displacements. New damping elements and applications will continue to be developed.
- Self-centering systems. Although not specifically addressed in present codes, there is a desire for higher performing systems to have the ability to "re-center" after a major earthquake, with the resulting building lateral displacements below a threshold that would limit their post-earthquake occupancy and/or damage. Typically, a secondary lateral system with limited stiffness, but large deformation capacity is combined with a primary structural system. The secondary system has the capacity to re-center the structure in the event that the primary system undergoes significant inelastic response. A number of systems are under development for these systems, including moment frames and braced frames. In some instances, the self-centering elements take advantage of gravity through controlled rocking of the frames.
- Replaceable fuse systems. Engineers may be able to design systems such that there are defined "fuses" that may need to be replaced after a major event. These elements would go beyond the present approach for ductile elements in the present provisions, designing them specifically for replacement. Additional system requirements would likely be required (self-centering, e.g.), but other elements of the system could have reduced requirements.
- R=3. The "R=3" steel system allows the design of structural steel buildings in areas of lower seismic demand using only AISC 360 and a low R factor. Recent studies into whether or not this approach will result in equivalent performance are underway. It may be that this approach will need to be modified as a result of these studies.
- "Mix and Match" Systems. Presently, all system definitions assume that the same SLRS extends from the base of the structure to the roof, or if a change is made, that the entire structure be designed with the higher design base shear dictated by the lower ductility system. Nonlinear analyses of structures allows better identification of locations where ductility demands are high and low. It could be possible to have a moment frame building where SMF joints are at certain locations, IMF at others, and OMF at still others. Similar concepts could be applied to braced frame structures.

In order to make many of these advances in our design provisions, significant research and investigation will be required. Engineers will continue to seek new and better ways to design their structures and serve their clients, but without continued significant investment in seismic research, many of the items listed above will not be able to be realized. Coordinated effort of the practicing engineering community and international research efforts to utilize the wide array of structural testing facilities and analytical talent will be needed if this is to come to fruition.

CONCLUSION

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 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, was incorporated into 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. Many of the anticipated changes will undoubtedly improve the seismic performance of our structures, in accordance with the increase expectations of building owners and society at large.

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