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LATEST UPDATES TO U.S. SEISMIC DESIGN PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

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SUMMARY

The American Institute of Steel Construction (AISC) document "*Seismic Provisions for Structural Steel Buildings*", are becoming the reference document for seismic design of steel structures throughout the United States. The 2000 International Building Code incorporated these provisions by reference, and the 2002 National Fire Protection Association 5000 Building Code and the 2003 International Building Code have also followed this approach. Since their 1997 publication, the AISC Seismic Provisions have been updated on a regular basis. Two such supplements have been produced and published by AISC, the first early in 1999 and the second late in 2000. The next revision culminated with the publication of a completely new set of provisions late in 2002. This revision continued to incorporate the findings of the recently completed FEMA/SAC steel project. Coordinated efforts between AISC and the Building Seismic Safety Council (BSSC) are intended to continue the process of keeping the seismic design provisions for structural steel buildings as current as possible. This presentation will summarize the changes incorporated into the 2002 AISC Seismic Provisions. It will also speculate on future modifications and additions to the U.S. seismic design provisions for structural steel buildings.

INTRODUCTION

Between the advent of incorporation of modern seismic design provisions into the Uniform Building Code (UBC) in the early 1950's, and the late 1980's, the SEAOC Seismology Committee was almost completely responsible for the content of these provisions with the primary resource for the UBC being the SEAOC Bluebook. Spurred on by the federally funded National Earthquake Hazard Reduction Program (NEHRP), in the late 1980's and early 1990's seismic design issues began to be seen as more of a nationwide issue. 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, the American Institute of Steel Construction (AISC) established a parallel effort and began the development of a set of seismic design

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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 LRFD format rather than 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 the U.S. Federal Emergency Management Agency (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. 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 Postearthquake 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. In addition technical reports for each of the over sixty 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, Special Truss Moment Frames (STMF) and Special Concentrically Braced Frames (SCBF) were introduced in the 1997 Provisions. A major expansion to 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 provisions to ease the transition from working stress to strength seismic design of steel structures. Part I of the 1997 AISC Seismic Provisions were incorporated into the 2000 IBC (ICC, 2000) by reference.

SUPPLEMENTS NO. 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 Specifications Committee 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 was completed and published on February 15, 1999. This supplement incorporated the new ASTM A992 specification for rolled steel 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 force resisting system be made with filler metals that have a rated notch toughness, and clarified the intent of the gusset plate configuration in SCBF's. Supplement No. 1 to the 1997 AISC Seismic Provisions (AISC, 1999) was also incorporated into the 2000 IBC by reference.

Supplement Number 2 to the 1997 AISC Seismic Provisions (AISC, 2000) was published on November 11, 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's), such that excessively weak panel zones would be avoided, 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 stricter the limitations on the use of Ordinary Concentrically Braced Frames (OCBF) 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

The most recent publication of the AISC Seismic Provisions occurred in 2002. Because the scope of changes that have been made to these provisions since 1997 was been so large, the provisions were republished in their entirety. 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 ASCE 7 document, "Minimum Design Loads for Buildings and Other Structures". This allowed the document to be incorporated by reference into upcoming editions of the IBC and NFPA building codes that will use ASCE 7-02 as their basis for design loads. As a result, no matter which of the two codes are adopted by local jurisdictions in 2003 and 2004, 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. 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. The following paragraphs will summarize the important elements of the 2002 AISC Seismic Provisions.

The first five sections of Part I of the provisions integrate the technical provisions that are presented in the following sections with the AISC LRFD 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 LRFD provisions, 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 can 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. The Commentary to the provisions states that "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 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 yield stress, Fy, is limited to 50 ksi except for columns where inelastic behavior is not expected. 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 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 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 strength (as opposed to the ASTM specified minimum value) of all the members in the system. For all base materials, Table I-6-1 specifies a term Ry that when multiplied by the nominal yield strength Fy, results in the expected yield strength of the material. Other sections in the provisions define when the Ry term is 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 Design Strength are calculated on the same member or connecting element, Ry can also be applied to the Design Strength.

Section 7 of the provisions addresses the design of connections, joints and fasteners in the seismic load resisting system (SLRS). All bolts are to be pre-tensioned, 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 line of force. 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. Finally, the engineer is required to demonstrate that the capacity of all bolted connections is controlled by a ductile limit state.

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. lbf at minus 20F as demonstrated by AWS classification or manufacturer certification. To ensure proper performance at operating temperatures, additional toughness requirements are placed on critical CJP welds in SMF and IMF (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. The final paragraphs of Section 7 alert the Engineer that discontinuities in the members of the SLRS must be

avoided to limit the chance for premature, brittle fracture of the members. Special limitations of this type are presented for expected regions of plastic hinging.

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 Table B5.1 of the LRFD Specification 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, due to the potentially catastrophic effects of such a failure. Similar limitations are also placed on column splices and column bases. In addition, in moment frames, 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 indicated that continuity of these columns significantly improved the seismic performance of these frames in severe seismic events. The final paragraphs of Section 8 address the design of steel H-piles 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 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 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 level. The CPRP will set the limits of prequalification for each connection type. AISC has established a CPRP that is presently reviewing and developing prequalification limits for a number of 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. 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. Equation 9-1 of the provisions defines the capacity of the panel zones. 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. 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.

Sections 9.7 and 9.8 of the provisions address 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 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 plastic hinges at each 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 either Appendix P or 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 severely limits the use of these systems in the higher seismic design categories. Other than the requirement for connection qualification by testing, the design of these systems is generally performed to be in accordance with the LRFD specification.

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 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 applications.

The design requirements for STMF systems are presented in Section 12. This system was developed in a series of testing and analytical research projects under the supervision of Professor Subhash Goel at the University of Michigan. 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 do not place any of the ductility provisions for SCBF's other than to require that bracing connections be designed for the tensile strength of the braces. Members and connections in OCBF's are designed including the Amplified Seismic Load, which results in an effective R factor of approximately 2.5 for these systems.

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. 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 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. An exception is provided if there is substantial reinforcement of the connection which 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.

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. All complete joint penetration and partial joint penetration welds are required to have non-destructive testing in accordance with AWS D1.1. In addition, this section highlights the need for extensive visual inspection to confirm that appropriate procedures are being followed. Finally, this section includes a brief paragraph that requires non-destructive testing when the k-area of rolled sections are subjected to welding of doubler plates or continuity plates, since these areas are potentially of low toughness due to roller straightening that occurs during production.

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. These Provisions for the seismic design of composite structural steel and reinforced concrete buildings are based upon 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, Part I of these Provisions, the LRFD Specification (AISC, 1999) and ACI 318 (ACI, 2002), form an important basis 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 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 this 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 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 the requirements of Part I of the Provisions with the LRFD provisions, ACI 318 requirements for reinforced concrete members and the results of composite construction research. The next 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. Example connections, such as that shown in Figure 1, are also included.

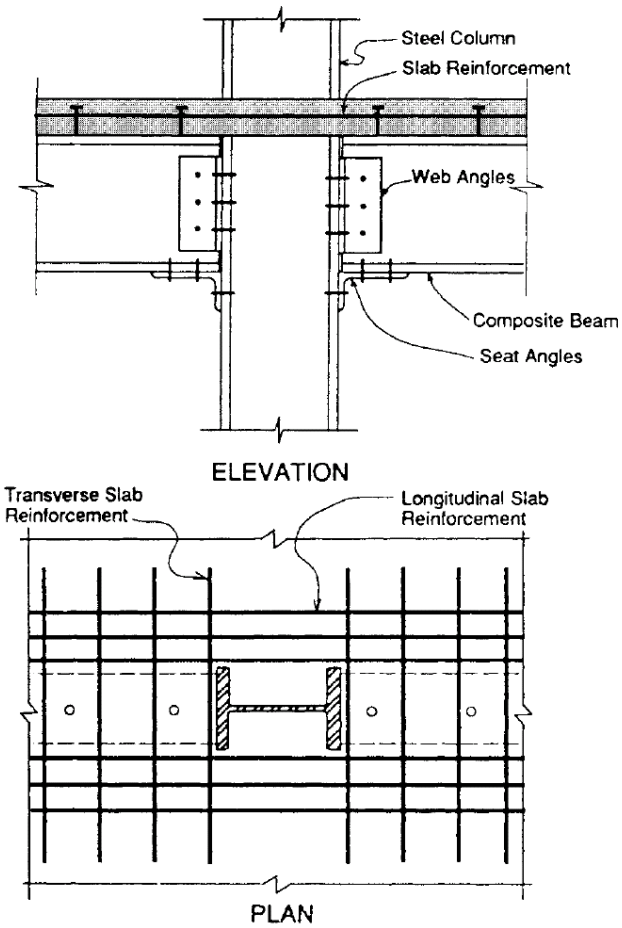

Figure 1 – Example of composite partially restrained connection

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 1. For braced frame systems, there are two concentrically braced and one eccentrically braced systems 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 (generally acting as part of the boundary elements, as shown in Figure 2) 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.

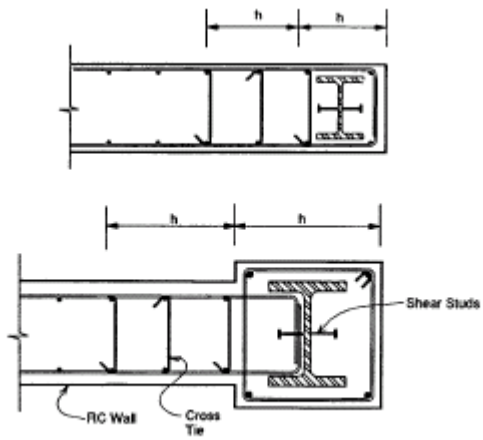

Figure 2 – Composite shear wall boundary element

Part III of the 2002 AISC Seismic Provisions has been included for designers that choose to use ASD in the seismic design of steel structures. These provisions all for the selection of members in ASD format to provide performance as intended for those designed in LRFD using Part I. Part III is intended as an overlay to Part I, so that the designer will use Part I for the seismic design unless a section is replaced by or modified by a section in Part III. Since the seismic requirements of seismic design are based on the expected nonlinear performance of a structure, the use of ASD in its traditional form is difficult since member strength, not allowable stress, is the dominant consideration to ensure that the inelastic action is in the intended members of the frame. The procedures of this section provide a means to convert allowable stresses into nominal member strength. Provisions have not been included for the use of ASD with Part II because ACI 318 is only presented in a limit-state design format.

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, Cd, and Ω. 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.
Two such systems that are presently being developed and incorporated into the 2003 NEHRP Provisions are the Buckling Restrained Brace Frame (BRBF) and the Steel Plate Shear Wall (SPSW). Both of these systems will be considered for inclusion in the 2005 version of the AISC Seismic Provisions.

The BRBF system, was originally developed in Japan, and has recently been used on a number of projects on the West Coast. This system 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 have been developed by a task force comprised of members from the SEAOC Seismology Committee and AISC TC9.

The SPSW system has been used on a number of buildings in high seismic regions dating back to the early 1970's. Renewed interest in this system developed in the 1990's in Canada as the result of a series of research projects at the University of British Columbia and the University of Alberta. Figure 3 shows the ductile response from a wall tested as part of this research. 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 to being developed by a BSSC TS6 task committee will be based on both of these research efforts and will be consistent with other system definitions in the AISC Seismic Provisions.

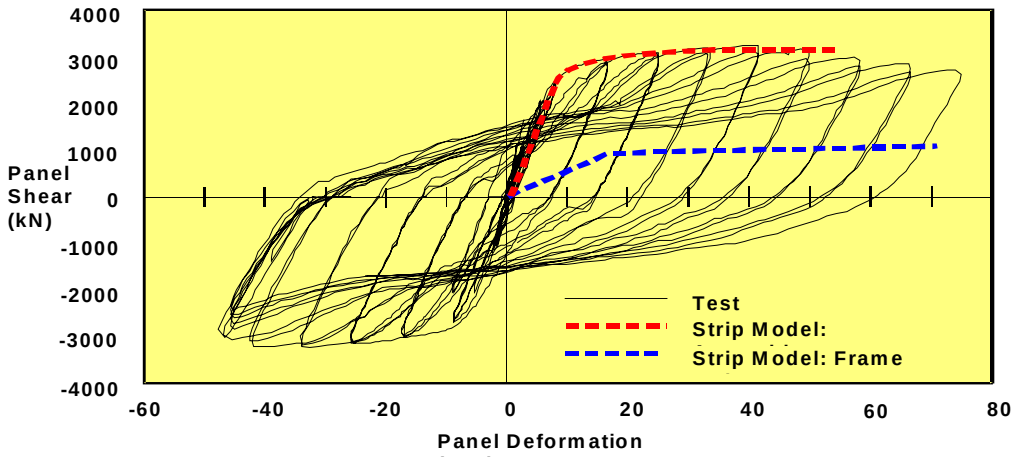

Figure 3 – Steel Plate Shear Wall test results (Driver, et. al.)

Another major change to the 2005 AISC Seismic Provisions will be in format. Consistent with the changes to the main design specification, the 2005 Seismic Provisions will combine ASD and LRFD into a single specification. As such, Part III will be absorbed into Part I. A number of other significant technical modifications are anticipated. These include the following:

1) Relaxing the limitations of oversize holes in bolted joints.
2) Making the requirements on splices in columns that are not part of the SLRS apply to all systems, not just moment frames.
3) Improving the provisions related to the design of column bases.
4) Making the stability bracing requirements more consistent throughout the document.
5) Increasing the stability bracing requirements for IMF systems.
6) Significantly improving the provisions related to quality assurance and quality control.

As noted above, the primary responsibility for the development of the specific seismic design provisions for steel buildings will rest with AISC, and the prescription of design coefficients and development of new systems will be led by BSSC/NEHRP. In addition, the SEAOC Seismology Committee will undoubtedly continue to provide significant input and recommendations to the process. The American Welding Society (AWS) also has a seismic subcommittee that plans to publish a document to specifically address welding related issues that relate specifically to seismic applications. This document will be an important link to the AISC Seismic Provisions, helping to ensure that the design intent is accomplished on the constructed projects. Since it is expected that the AWS document will not yet be approved, it is planned to incorporate a number of the topics related to welding quality in an Appendix in the 2005 provisions. Once the AWS standard is published, this appendix will be removed.

CONCLUSIONS

Over the last ten 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, SEAOC 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 1997 AISC Seismic Provisions were supplemented twice to keep them as up to date as possible. The most recent publication of the AISC Seismic Provisions in 2002, allowed for this edition to be incorporated into both the 2002 NFPA 5000 document and the 2003 IBC. As a result, 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.

A brief discussion of the major changes planned for the 2005 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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