PROPOSAL 6-114 (2009)

SCOPE: Part 2, Commentary Chapter 14

PROPOSAL FOR CHANGE:

Add Chapter 14 to Part 2, of the 2009 Commentary:

Proposed Chapter is attached. Text is not underlined to allow easier review.

REASON FOR PROPOSAL:

One of the basic tasks of the 2009 *NEHRP Recommended Provisions* update is to develop a viable commentary to Part 1. Since Part 1 adopts ASCE 7-05 and lists any exceptions to it, the Commentary is developed in accordance with the format and sections of ASCE 7-05.

This chapter was developed by one or more Technical Subcommittees. It was edited and redistributed for review and approval before submission to BSSC.

Chapter14

MATERIAL SPECIFIC SEISMIC DESIGN AND DETAILING REQUIREMENTS

Because seismic loading is expected to cause nonlinear behavior in structures, seismic design criteria require not only provisions to govern loading, but also provisions to define the required configurations, connections, and detailing to produce material and system behavior consistent with the design assumptions. Thus, while ASCE 7 is primarily a loading standard, compliance with Chapter 14, which covers material specific seismic design and detailing, is required. In general, Chapter 14 adopts material design and detailing standards developed by industry material standards organizations. These materials standards organizations maintain complete commentaries covering their standards and such material is not duplicated here.

The refinements, additions, and recommended changes to the material standards produced by the Provisions Update Committee appear in Part 1 of the Provisions, as exceptions to ASCE/SEI 7, along with associated commentary.

C14.0 SCOPE

The scoping statement in this section clarifies that foundation elements are subject to all of the structural design requirements of the standard.

C14.1 STEEL

C14.1.1 Reference Documents. This section lists a series of structural standards published by AISC, AISI, ASCE and SJI that are to be applied in the seismic design of steel members and connections in conjunction with the requirements of ASCE 7-05. The AISC references are available free of charge in electronic format at www.aisc.org.

C14.1.2 Seismic Design Categories B and C. For the lower Seismic Design Categories B and C, the engineer is allowed a choice in the design of a steel lateral force resisting system. The first option is to design the structure to meet the design and detailing requirements for structures assigned to higher Seismic Design Categories, with the corresponding seismic design parameters (R, Ω_0, C_d and). The second option is to use a lower R factor of 3 (and higher resulting base shear), an Ω_0 of 3, and a C_d value of 3 but without specific seismic design and detailing requirements. The concept of this option is that design for a higher base shear force will result in essentially elastic response that will compensate for the limited ductility of the members and connections, resulting in performance similar to that of more ductile systems.

C14.1.3 Seismic Design Categories D through F. For the higher Seismic Design Categories, the Engineer is not given a choice, but must follow the seismic design provisions of either AISC or AISI using the seismic design parameters specified for the chosen structural system. It is not considered appropriate to design structures without specific design and detailing for seismic response in these high seismic design categories.

C14.1.4 Cold-Formed Steel. This section adopts two standards by direct reference –

AISI NAS, *North American Specification for the Design of Cold-Formed Steel Structural Members,* and

ASCE 8, *Specification for the Design of Cold Formed Stainless Steel Structural Members.*

Both of the adopted reference documents have specific limits of applicability. AISI NAS applies to the design of structural members that are cold-formed to shape from carbon or low-alloy steel sheet, strip, plate, or bar not more than one-inch in thickness. **[AISI NAS:** A1.1] ASCE 8 governs the design of structural members that are cold-formed to shape from annealed and cold-rolled sheet, strip, plate, or flat bar stainless steels. [**ASCE 8:** 1.1.1] Both documents focus on load-carrying members in buildings; however, allowances are made for applications in nonbuilding structures, if dynamic effects are considered appropriately.

Within each document, there are requirements related to general provisions for the applicable types of steel; design of elements, members, structural assemblies, connections, and joints; and mandatory testing. In addition, AISI NAS contains a chapter on the design of cold-formed steel structural members and connections undergoing cyclic loading. Both standards contain extensive commentaries for the benefit of the user.

C14.1.4.1 Light-Framed Cold-Formed Construction. This subsection of cold-formed steel relates to light-framed construction, which is defined as a method of construction where the structural assemblies are formed primarily by a system of repetitive wood or cold-formed steel framing members or subassemblies of these members. [**ASCE 7:** 11.2] Not only does this subsection repeat the direct adoptions of AISI NAS and ASCE 8, but it also allows the user to choose from an additional suite of standards that address different aspects of construction, including the following:

1) AISI GP, *Standard for Cold-Formed Steel Framing— General Provisions*, applies to the design, construction, and installation of structural and non-structural cold-formed steel framing members where the specified minimum base metal thickness is between 18 mils and 118 mils. [**AISI GP:** A1]
2) AISI WSD, *Standard for Cold-Formed Steel Framing – Wall Stud Design,* applies to the design and installation of cold-formed steel studs for both structural and non-structural walls in buildings. [**AISI WSD:** A1]
3) AISI Lateral, *Standard for Cold-Formed Steel Framing – Lateral Design,* contains design requirements for shear walls, diagonal strap bracing (as part of a structural wall), and diaphragms. [**AISI Lateral:** A1]

The requirements of AISI GP apply to all light-framed cold-formed steel and, consequently, the standard is adopted by direct reference in both AISI WSD and AISI Lateral. In addition, all of these documents include commentaries to aid the user in the correct application of their requirements.

C14.1.5 Prescriptive Framing. This section adopts AISI PM, *Standard for Cold-Formed Steel Framing—Prescriptive Method for One and Two Family Dwellings*, which applies to the construction of detached one-and two-family dwellings, townhouses, and other attached single-family dwellings not more than two stories in height using repetitive in-line framing practices. [**AISI PM:** A1] This document adopts AISI GP by direct reference and includes a commentary to aid the user in the correct application of its requirements.

C14.1.6 Steel Deck Diaphragms. Design of steel deck diaphragms is to be based upon recognized national standards or a specific testing program directed by a person experienced in testing procedures and steel deck. All fastener design values (welds, screws, power actuated fasteners, button punches) for attaching steel deck sheet to steel deck sheet or for attaching the steel deck to the building framing members must be per recognized national design standards or specific steel deck testing programs. All steel deck diaphragm and fastener design properties must be approved for use by the authorities in whose jurisdiction the construction project occurs. Steel Deck diaphragm in-plane design forces (seismic, wind, or gravity) must be determined per Section 12.10.1. Steel deck manufacturer test reports prepared in accordance with this provision can be used where adopted and approved by the authority having jurisdiction for the building project. The diaphragm design manual produced by the Steel Deck Institute (SDI, 2004) is also a potential reference for design values.

Steel deck is assumed to have a corrugated profile consisting of alternating up and down flutes that are manufactured in various widths and heights. Use of flat sheet metal as the overall floor or roof diaphragm is permissible where designed by engineering principles, but is beyond the scope of this section. Flat or bent sheet metal may be used as closure pieces for small gaps or penetrations or for shear transfer over short distances in the steel deck diaphragm where diaphragm design forces are considered.

Steel deck diaphragm analysis must include design of chord members at the perimeter of the diaphragm and around interior openings in the diaphragm. Chord members may be steel beams attached to the underside of the steel deck designed for a combination of axial loads and bending moments due to acting gravity and lateral loads.

Where diaphragm design loads exceed the bare steel deck diaphragm design capacity, then either horizontal steel trusses or a structurally designed concrete topping slab placed over the steel deck must be provided to distribute lateral forces. Where horizontal steel trusses are used, the steel deck must be designed to transfer diaphragm forces to the steel trusses. Where a structural concrete topping over the steel deck is used as the diaphragm, the diaphragm chord members at the perimeter of the diaphragm and edges of interior openings must be either 1) designed flexural reinforcing steel placed in the structural concrete topping or 2) steel beams located under the steel deck with connectors (that provide a positive connection) as required to transfer design shear forces between the concrete topping and steel beams.

C14.1.7 Steel Cables. These provisions reference ASCE 19-96 “Structural Applications of Steel Cables for Buildings” for the determination of the design strength of steel cables. ASCE 19-96 uses service level load combinations with a safety factor relative to the cable design strength. The service level load combinations specified in ASCE 19-96 are adjusted in two ways. First, the prestress loading is multiplied by a factor of 1.1 to account for any over prestressing that may occur in the field. Second, the safety factor for load combinations including seismic effects is reduced from 2.0 to 1.5 to account for the dynamic nature of seismic loading and the ductility of the system. While T3 and T4 in ASCE 19-96 may be calculated using either wind or seismic loads, the modifications of this section apply only to load combinations including seismic loadings.

C14.1.8 Additional Detailing Requirements for Steel Piles in Seismic Design Categories D through F. Steel piles used in higher Seismic Design Categories are expected to yield just under the pile cap or foundation due to combined bending and axial load. Design and detailing requirements of AISC 341 for H-piles are intended to produce stable plastic hinge formation in the piles. Since piles can be subjected to tension due to overturning moment, mechanical means to transfer such tension must be designed for the required tension force, but not less than 10 percent of the pile compression capacity.

C14.2 CONCRETE

The section adopts ACI 318-05 by reference for structural concrete design and construction. In addition, modifications to ACI 318-05 are made to coordinate the provisions of that material design standard with the provisions of ASCE/SEI 7-05.

C14.2.2.1 ACI 318, Section 7.10. The reinforcement details for ties in compression members prescribed in Section 7.10.5 of ACI 318 are appropriate for SDC A and B structures. This modification prescribes additional details for ties around anchor bolts of structures assigned to SDC C, D, E, or F.

C14.2.2.2 ACI 318, Section 10.5. This provision affects ordinary moment frames. It is intended to improve continuity, and thereby lateral force resistance and structural integrity, compared to that of frames designed to the provisions of Chapters1 through 18 of ACI 318 only. The provision does not apply to slab-column moment frames.

C14.2.2.3 ACI 318, Section 11.11. This requirement is intended to provide additional toughness to resist shear for columns of frames in SDC B. Otherwise the proportions of those columns make them more susceptible to shear failure under earthquake loading.

C14.2.2.4 Definitions. The first four definitions relate the wall types of ASCE 7 with detailing requirements of ACI 318 and distinguish between ordinary reinforced concrete structural walls and ordinary precast structural walls. These definitions are essential to the proper interpretation of the R and C_d factors for each wall type specified in Table 12.2-1.

A wall pier is recognized as a separate category of structural element in this document but not in ACI 318.

C14.2.2.5 Scope. ACI 318 uses the terminology of low, moderate, and high seismic risk for structures assigned to SDC A and B, SDC C, and SDC D through F, respectively. The modifications of this provision show how the ACI 318 provisions should be interpreted for consistency with the ASCE 7 provisions.

C14.2.2.6 Reinforcement in Members Resisting Earthquake-Induced Forces. ACI 318 does not allow the use of prestressing tendons in special and intermediate moment frames. This provision and Sections 14.2.2.7 and 14.2.2.8 impose conditions that have been demonstrated to permit the safe use of such tendons.

These provisions are intended to apply to frames containing unbonded tendons only. The average prestress in plastic hinge regions is restricted to limit the strain in the prestressing steel under the design displacement to not greater than 1 percent. The strain in the prestressing steel at the design displacement should be calculated considering the anticipated inelastic mechanism of the structure.

C14.2.2.7 Anchorages for Unbonded Posttensioning Tendons. Fatigue testing for 50 cycles of loading between 40 and 80 percent of the specified tensile strength of the prestressing strand has been an industry practice of long standing (ACI 423.6). The 80 percent limit is increased to 85 percent for seismic applications in order to correspond to a 1% limit, and therefore the effective start of yielding, in the prestressing steel. Testing over this range of stress conservatively simulates the effect of a severe earthquake on structures prestressed in accordance with the requirements of Sections 14.2.2.6 and 14.2.2.8.

C14.2.2.8 Flexural Members of Special Moment Frames. The restrictions on the flexural strength provided by the tendons are based on the results of analytical and experimental studies (Ishizuka and Hawkins, 1987; Park and Thompson, 1977; Thompson and Park, 1980). Although satisfactory seismic performance can be obtained with greater amounts of prestressing steel, this restriction is needed to allow the use of the same response modification and deflection amplification factors as those specified for special moment frames without prestressing steel.

C14.2.2.9 Wall Piers and Wall Segments. Wall piers are typically segments between openings in walls that are thin in the direction normal to the face of the wall. In current practice these elements are often not regarded as columns or as part of the special structural walls. If not properly reinforced these elements are vulnerable to shear failure, and that failure prevents the wall from developing the assumed flexural hinging. Section 21.7.10 is written specifically to preclude such pre-emptive shear failure. Wall segments with a horizontal length-to-thickness ratio less than 2.5 and a clear height-to-length ratio of at least 2 are required to be designed as columns in compliance with Section 21.4 if they are used as part of the lateral-force-resisting system, even though the shortest cross-sectional dimension may be less than 12 inches in violation of Section 21.4.1.1. Such wall segments may be designed to comply with Section 21.11 if they are not used as part of the lateral-force-resisting system. Wall segments with a horizontal length-to-thickness ratio larger than or equal to 2.5, which do not meet the definition of wall piers (Section 14.2.2.4), must be designed as special structural walls or as portions of special structural walls in full compliance with Section 21.7.

C14.2.2.12 Members Not Designated as Part of the Lateral-Force-Resisting System. Section 21.4.3.2 of ACI 318 permits lap splices only within the center half of the column. Section 21.11.2. of ACI 318 applies where the magnitude of the moments induced in the column by the design displacement are explicitly checked. Section 21.11.3 applies where the effects of the design displacement are not explicitly checked. Section 21.11.2.2 of ACI 318, if not modified, would permit lap splices to be placed at any location over the height of the column if the column is expected to yield. If, however, the column is not expected to yield the wording effectively requires the splice to be located near mid-height. This is not rational and the modification results in a more rational provision.

C14.2.2.13 Columns Supporting Reactions from Discontinuous Stiff Members. Discontinuous shear walls and other stiff members can impose large axial forces on supporting columns. The specified transverse reinforcement is to improve column toughness under anticipated seismic demands.

C14.2.2.14 Intermediate Precast Structural Walls. Section 21.13 of ACI 318 imposes requirements on precast walls for moderate seismic risk applications. The intent is to produce ductile behavior by yielding of the steel elements or reinforcement between panels or between panels and foundations. The 2003 edition of the IBC restricted yielding to steel reinforcement because of concern that steel elements in the body of a connection could fracture due to strain demands.

Several steel element connections have been tested under simulated seismic loading and the adequacy of their load-deformation characteristics and strain capacity of yield has been demonstrated (Schultz and Magana). One such connection was used in the five-story building test that was part of the PRESSS Phase 3 research. The connection was used to provide damping and energy dissipation, and demonstrated a very large strain capacity (Nakaki, Stanton, and Sritharan). Since then several other steel element connections have been developed that can achieve similar results (Banks and Stanton; Nakaki et al.). In view of these results it is appropriate to allow yielding in steel elements that have been shown experimentally to have adequate strain capacity to maintain at least 80% of their yield force of through the full design displacement of the structure. This provision requires the designer to determine the deformation in the connection corresponding to the earthquake design displacement, and then to check for experimental data that the connection type used can accommodate that deformation without significant strength degradation.

The wall pier requirements of Section 21.13.5 duplicate the same requirements of Section 14.2.2.9 for wall piers in special structural walls.

C14.2.2.15 Detailed Plain Concrete Shear Walls. Design requirements for plain masonry walls have existed for many years, and the competing type of concrete construction is the plain concrete wall. To allow the use of such walls as the lateral-force-resisting system in SDC A and B, this provision requires such walls to contain at least the minimal reinforcement specified in Section 22.6.7.2.

C14.2.2.16 Plain Concrete in Structures Assigned to Seismic Design Category C, D, E, or F. Modifications are made to Section 22.10 of ACI 318 that restrict markedly the use of ordinary and detailed structural plain concrete walls in SDC C, D, E, and F.

C14.2.2.17 General Requirements for Anchoring to Concrete. ACI 318 uses the terminology of regions of moderate or high seismic risk, and structures assigned to intermediate or high seismic performance or design categories. In this modification the only changes to ACI 318 in provisions D3.3.3 through D3.3.4 are the replacement of that terminology with the SDC terminology.

In D3.3.5, there are two changes to the ACI 318 provision. The first is the use of the SDC terminology, and the second is the addition of the last phrase of the provision referring to the minimum design strength of the anchors. The last phrase requires an anchor strength that is at least the maximum likely Ω_0 value (2.5) times the design force calculated as being transmitted to the attachment by the lateral-force-resisting system.

C14.2.2.18 Strength Requirements for Anchors. ACI 318 requires laboratory testing to establish the strength of anchor bolts greater than 2 inches in diameter or exceeding 25 inches in tensile embedment depth. This modification makes the ACI 318 equation giving the basic concrete breakout strength of a single anchor in tension in cracked concrete applicable irrespective of the anchor bolt diameter and tensile embedment depth.

Korean Power Engineering (KPE) has made tension tests on anchors with diameters up to 4.25 inches and embedment depths up to 45 inches and found that the diameter and embedment depth limits of Section D4.2.2 of ACI 318 for the design procedure for anchors in tension (Section D5.2) can be eliminated. KPE has also made shear tests on anchors with diameters up to 3.0 inches and embedment depths as large as 30 inches and found no effect of the embedment depth on shear strength. However, the diameter tests showed that the basic shear breakout strength equation (D-24) needed some modification for the complete elimination of the 2 inch limit to be fully appropriate. Analytical work performed at the University of Stuttgart supports the need for some modification to the equation D-24. Changes consistent with the Korean and Stuttgart findings have already been made to the FIB Design Guide for anchors and a change proposal consistent with those changes has been submitted to ACI 318 for consideration.

C14.2.3.1.2 Reinforcement for Uncased Concrete Piles (SDC C): The transverse reinforcing requirements in the potential plastic hinge zone of uncased concrete piles in Seismic Design Category C is a selective composite of two ACI 318 requirements. In the potential plastic hinge region of an intermediate moment-resisting concrete frame column, the transverse reinforcement spacing is restricted to the least of: (1) 8 times the diameter of the smallest longitudinal bar, (2) 24 times the diameter of the tie bar, (3) one-half the smallest cross-sectional dimension of the column, and (4) 12 inches. Outside of the potential plastic hinge region of a special moment-resisting frame column, the transverse reinforcement spacing is restricted to the smaller of: 6 times the diameter of the longitudinal column bars and 6 inches.

C14.2.3.1.5 Reinforcement for Precast Nonprestressed Concrete Piles (SDC C): Transverse reinforcement requirements inside and outside of the plastic hinge zone of precast nonprestressed piles are clarified. The transverse reinforcement requirement in the potential plastic hinge zone is a composite of two ACI 318 requirements (see Section C14.2.3.1.2). Outside of the potential plastic hinge region the eight longitudinal-bar-diameter spacing is doubled. The maximum 8-in. tie spacing comes from current building code provisions for precast concrete piles.

C14.2.3.1.6 Reinforcement for Precast Prestressed Piles (SDC C): The transverse and longitudinal reinforcing requirements given in Chapter 21 of ACI 318 were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. The requirements are based on the Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling, PCI Committee on Prestressed Concrete Piling, 1993.

Equation 14.2-1, originally from ACI 318, has always been intended to be a lower-bound spiral reinforcement ratio for larger diameter columns. It is independent of the member section properties and therefore can be applied to large or small diameter piles. For cast-in-place concrete piles and precast prestressed concrete piles, the resulting spiral reinforcing ratios from this formula are considered to be sufficient to provide moderate ductility capacities.

Full confinement per Equation 14.2-1 is required for the upper 20 feet of the pile length where curvatures are large. The amount is relaxed by 50 percent outside of that length in view of lower curvatures and in consideration of confinement provided by the soil.

C14.2.3.2.5 Reinforcement for Precast Concrete Piles (SDC D through F): The transverse reinforcement requirements for precast nonprestressed concrete piles are taken from current building code requirements and are intended to provide ductility in the potential plastic hinge zones.

C14.2.3.2.6 Reinforcement for Precast-Prestressed Piles (SDC D through F): The last paragraph provides minimum transverse reinforcement outside of the zone of prescribed ductile detailing.

C14.3 COMPOSITE STEEL AND CONCRETE STRUCTURES

This section provides guidance on the design of composite and hybrid steel-concrete structures. Composite structures are defined as those incorporating structural elements made of steel and concrete portions connected integrally throughout the structural element by mechanical connectors, bond, or both. Hybrid structures are defined as consisting of steel and concrete structural elements connected together at discrete points. Composite and hybrid structural systems mimic many of the existing steel (moment and braced frame) and concrete (moment frame and wall) configurations, but are given their own design coefficients and factors in Table 12.2-1. Their design is based on the same ductility and energy dissipation concepts used in conventional steel and reinforced concrete structures, but requires special attention to the interaction of the two materials as it affects the stiffness, strength, and inelastic behavior of the members, connections, and systems.

C14.3.1 Reference Documents. Seismic design for composite structures assigned to Seismic Design Category D, E, or F is governed primarily by *Part II: Composite Structural Steel and Reinforced Concrete Buildings* of ANSI/AISC 341. Part II of ANSI/AISC 341 is less prescriptive than Part I, and provides flexibility for designers to utilize analytical tools and results of research in their practice. Composite structures assigned to Seismic Design Category A, B, or C may be designed according to principles outlined in ANSI/AISC 360 and ACI 318; as ACI 318 and ANSI/AISC 360 provide little guidance on connection design, designers are encouraged to review ANSI/AISC 341 Part II for guidance on the design of joint areas. Differences between older AISC and ACI provisions for cross-sectional strength for composite columns have been minimized by changes in the latest ANSI/AISC 360. However, there is not uniform agreement between the provisions in ACI 318 and ANSI/AISC 360 regarding detailing, limits on material strengths, stability, and shear design for composite columns. The composite design provisions in ANSI/AISC 360 are considered to be current.

C14.3.2 Metal-cased Concrete Piles. Design of metal-cased concrete piles, which are analogous to circular concrete filled tubes, is governed by Sections 14.2.3.1.3 and 14.2.3.2.4. The intent of these provisions is to require metal-cased concrete piles to have confinement and protection against long-term deterioration comparable to that for uncased concrete piles.

C14.4 MASONRY

C14.4.2 *R* Factors. Where intermediate and special reinforced masonry shear walls are designed using the allowable-stress provisions of the MSJC Code, these additional requirements are intended to produce a level of inelastic flexural deformation capacity consistent with that of intermediate and special reinforced masonry shear walls designed using the strength-design provisions of the MSJC Code. The additional requirements are discussed in Section C14.4.6.

C14.4.3 Classification of Shear Walls. Section 1.14 of the 2005 MSJC Code can be interpreted as permitting, in SDC A and B, masonry walls that need not be considered part of the lateral force-resisting system, and that do not need to be isolated. Section 14.4.3 is intended to preclude that interpretation.

C14.4.5.1 Separation Joints. This section is intended to address force transfer across interfaces between masonry and other materials, but it is redundant. The MSJC Specification (Article 3.2 B) requires that the interface between concrete and masonry be cleaned and acceptable for laying of units. Further, the MSJC Code (Section 1.9.4.2.4) addresses the design and transfer of shear at interfaces. MSJC Code Section 1.7.5.2 requires that a load path and force transfer between a foundation and the masonry above be maintained.

C14.4.5.2 Flanged Shear Walls. Section 1.9.4.2.3 of the 2005 MSJC Code contains the compression requirement (lesser of 6 times the flange thickness or the actual flange). The principal effect of the tension provision in Section 14.4.5.2 of the standard is to establish the amount of tensile reinforcement used in calculating flexural capacity and maximum permitted reinforcement, but this provision is not well established technically. Research in masonry, and analogous design provisions for concrete (Section 21.7.5.2 of ACI 318), suggest that effective flange widths in tension are more logically related to the total wall height rather than the floor-to-floor height. The MSJC and ASCE 7 are working together to resolve this issue, and add appropriate requirements to the MSJC Code.

C14.4.6 Modifications to Chapter 2 of ACI 530/ASCE 5/TMS 402. Chapter 2 of the 2005 MSJC Code deals with allowable-stress design.

C14.4.6.1 Stress Increase. The 2005 MSJC Code permits allowable stresses to be increased by one-third for allowable-stress loading conditions that include wind or earthquake, provided that the legally adopted building code so permits. While the alternate allowable-stress loading combinations of the 2006 International Building Code do so permit, the allowable-stress loading combinations of ASCE 7 do not.

C14.4.6.2 Reinforcement Requirements and Details.

C14.4.6.2.1 Reinforcing Bar Size Limitations. The intent of this requirement is to prevent splitting of masonry due to the presence of reinforcement. A similar requirement is appears in Chapter 3 (Strength Design) of the 2005 MSJC Code. The MSJC is working to move that requirement to Chapter 1 (General Requirements), so that it would apply to all masonry construction.

C14.4.6.2.2 Splices. In general, the first portion of Section 14.4.6.2.2 (prohibition of splices in plastic hinge zones) is intended to produce adequate inelastic deformation capacity in those regions. In general, the presence of splices in plastic hinge zones reduces inelastic deformation capacity because the area of steel is doubled at the splice, reducing the extent of yielding. However, there is some controversy concerning the technical validity and necessity for masonry walls. Similar requirements apply to plastic hinge zones of reinforced concrete frames, they do not apply to plastic hinge zones of reinforced concrete walls. Also, this requirement does not distinguish between shear-critical and flexurally dominated shear walls. The MSJC is continuing to discuss related requirements for flexurally dominated, highly ductile shear walls.

The remaining portions of Section 14.4.6.2.2 (requirements for splices) are intended to provide adequate capacity of welded splices and mechanical connections. The MSJC is developing similar provisions.

C14.4.6.2.3 Maximum Area of Flexural Tensile Reinforcement. The intent of Section 14.4.6.2.3 is to produce adequate inelastic flexural deformation capacity in flexurally dominated masonry shear walls by placing an upper limit on flexural reinforcement, so that behavior is dominated by yielding of reinforcement rather than by crushing of the compression toe. Similar provisions appear in Chapter 3 (Strength Design) of the 2005 MSJC Code, and are being developed for Chapter 2 (Allowable-Stress Design).

C14.4.7 Modifications to Chapter 3 of ACI 530/ASCE 5/TMS 402.

C14.4.7.2 Splices in Reinforcement. See Section C14.4.6.2.2.

C14.4.7.3 Coupling Beams. The intent of this requirement is to produce adequate inelastic flexural deformation capacity in coupling beams. The section is somewhat redundant with Section 3.1.3 of the 2005 MSJC Code, which requires capacity design of masonry elements for shear.

C14.4.7.4 Deep Flexural Members. The intent of this requirement is to require that the design of deep flexural members correctly addresses the presence of distributed flexural reinforcement in capacity design for shear, and that crack widths are adequately controlled.

C14.4.7.5 Shear Keys. The intent of this requirement is to increase resistance to sliding shear at the foundation level of flexurally dominated masonry shear walls. The original proposal was based on laboratory research (Leiva et al., 1990) involving isolated shear walls. In subsequent research (Seible et al., 1993), flanged walls without shear keys did not show sliding.

C14.4.7.6 Anchoring to Masonry. The intent of this requirement is to guard against brittle failure of masonry anchorages that are part of the seismic force-resisting system.

C14.4.7.7 Anchor Bolts. Sections 14.4.7.7 and 14.4.7.8 augment the current anchor bolt provisions of Chapter 3 (Strength Design) of the 2005 MSJC Code to address pryout, and to include an appropriate ϕ factor.

C14.4.8 Modifications to Chapter 6 of ACI 530/ASCE 5/TMS 402. This requirement addresses an apparent inconsistency in the 2005 MSJC Code. Chapter 6 of that document, dealing with masonry veneer, permits corrugated sheet-metal anchors. Chapters 2 and 3 of that document do not permit multi-wythe, noncomposite masonry (functionally identical to veneer) to be bonded by corrugated sheet-metal anchors.

C14.4.9 Modifications to ACI 530.1/ASCE 6/TMS 602.

C14.4.9.1 Construction Procedures. This requirement was introduced originally as a result of the TCCMaR program as a way to address volume loss as a result of plastic shrinkage of grout. The original provision required the use of a particular admixture (Sika's Grout Aid®) in the grout. The MSJC *Specification* requires both consolidation and reconsolidation of masonry grout, which in combination with today's masonry construction materials can minimize grout shrinkage without the requirement of a proprietary grout admixture available from a single source.

C14.5 WOOD

C14.5.1 Reference Documents. Two national consensus standards are adopted for seismic design of engineered wood structures: the *National Design Specification* (NDS) (AF&PA, 2005a), and the *Special Design Provisions for Wind and Seismic* (SDPWS) Supplement to the NDS (AF&PA, 2005c). Both of these standards are presented in dual Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD) formats. Both standards reference a number of secondary standards for related items, such as wood materials and fasteners. SDPWS addresses general principles and specific detailing requirements for shear wall and diaphragm design, and provides tabulated nominal unit shear capacities for shear wall and diaphragm sheathing and fastening. The balance of member and connection design is to be in accordance with the NDS. A commentary to the NDS is published by AF&PA (2005b); commentary to the SDPWS is included in the SDPWS publication (AF&PA, 2005c).

C14.5.2 Framing. Section 14.5.2 provides specific guidance on two general topics related to detailing. First, vertical loads on columns and posts must be transferred in and out by end bearing only or by connectors only; mixing the capacity of end bearing and connectors is prohibited due to a potential lack of deformation compatibility. Second, load path continuity for top plates, which often function as collectors, is addressed.

REFERENCES

AF&PA, 2005a. *National Design Specification (NDS) for Wood Construction, ANSI/AF&PA NDS-2005.*. Washington, D.C.: AF&PA.

AF&PA, 2005b. *National Design Specification Commentary, 2005 ed.*, Washington D.C.: AF&PA

AF&PA, 2005c. *Special Design Provisions for Wind and Seismic (Wind & Seismic), ANSI/AF&PA SDPWS-2005*, Washington, D.C.: AF&PA.

American Institute of Timber Construction. 2005. *Timber Construction Manual, 5th ed.*. New York, New York: John Wiley and Sons, Inc.

APA The Engineered Wood Association. 2004. *Diaphragms and Shear Walls Design/Construction Guide, L350*. Tacoma, Washington: APA.

APA The Engineered Wood Association. 1994. *Northridge California Earthquake, T94-5.* Tacoma, Washington.: APA.

Applied Technology Council. 1981. *Guidelines for the Design of Horizontal Wood Diaphragms*, ATC-7. Redwood City, California: ATC.

Banks, G., and Stanton, J., (2005) “Panel-to-Panel Connections for Hollow-Core Shear Walls Subjected to Seismic Loading”, 2005 PCI Convention, Palm Springs, CA.

Breyer et. Al, 2006. *Design of Wood Structures ASD/LRFD*, Sixth Edition. New York, New York: McGraw-Hill Book Company.

Canadian Wood Council. 2005. *Wood Design Manual 2005*. Ottawa: Canadian Wood Council.

Canadian Wood Council. 1995. *Wood Reference Handbook*. Ottawa: Canadian Wood Council.

Cobeen, K., “Recent developments in the Seismic Design and Construction of Woodframe Buildings,” Chapter 18 of *Earthquake Engineering From Engineering Seismology to Performance-Based Engineering*, edited by Yousef Bozorgia and Vitelmo Bertero. Boca Raton, Florida: CRC Press.

CUREE, 2004. *Recommendations for Earthquake Resistance in the Design and Construction of Woodframe Buildings*, CUREE W-30. Consortium of Universities for Research in Earthquake Engineering, Richmond, CA: CUREE.

Department of the Army, Navy and Air Force. 1992. *Seismic Design for Buildings*, TM5-809-10 (Tri-Services Manual). Washington, D.C.: U.S. Government Printing Office.

Dolan, J.D., 2003. “Wood Structures,” Chapter 15 of *Earthquake Engineering Handbook*, edited by Wai-Fah Chen and Charles Scawthorn, Boca Raton, Florida: CRC Press.

Earthquake Engineering Research Institute. 1996. Northridge earthquake reconnaissance report, Chapter 6, Supplement C to Volume 11, pp. 125 et seq., *Earthquake Spectra*.

Faherty, Keith F., and T. G. Williamson. 1989. *Wood Engineering and Construction Handbook*. New York, New York: McGraw-Hill.

FEMA, 2003. NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures FEMA 450-1 and 450-2, Federal Emergency Management Agency, Washington, D.C.

FEMA, 2005. Coastal Construction Manual, Third Edition, FEMA 55. Washington D.C. FEMA.

Forest Products Laboratory. 1986. *Wood: Engineering Design Concepts*. University Park: Materials Education Council, The Pennsylvania State University.

Goetz, Karl-Heinz, Dieter Hoor, Karl Moehler, and Julius Natterer. 1989. *Timber Design and Construction Source Book: A Comprehensive Guide to Methods and Practice*. New York, New York: McGraw-Hill.

Hoyle and Woeste. 1989. *Wood Technology and Design of Structures*. Iowa State University Press.

ICC, 2006. ICC Standard on the Design and Construction of Log Structures, Third Draft, Country Club Hills, Illinois: ICC.

Ishizuka, T and Hawkins, N.M.,(1987) “Effect of Bond Deterioration on the Seismic Response of Reinforced and Partially Prestressed Concrte Ductile Moment Resistant Frames,” Report SM 87-2, Department of Civil Engineering, University of Washington, Seattle, WA.

Karacabeyli, E. and M. Popovsky “Design for Earthquake Resistance,” Chapter 15 of *Timber Engineering*, edited by H. Larsen & S. Thelandersson, Chichester. West Sussex: John Wiley & Sons.

Keenan, F. J. 1986. *Limit States Design of Wood Structures*. Morrison Hershfield Limited.

Nakaki, S., Becker, R., Oliva, M.G., and Paxson, D., (2005) “New Connections for Precast Wall Systems in High Seismic Regions,” 2005 PCI Convention, Palm Springs, CA.

Nakaki, S., Stanton, J.F., and Sritharan, S., (2001) “The PRESSS Five-Story Precast Concrete Test Building, University of California, San Diego, La Jolla, California,” PCI Journal, V.46, No.5, Sept.-Oct., pp.20-26.

Park, R., and Thompson, K.J.,(1977) “Cyclic Load Tests on Pretsressed and Partially Prestressed Beam-Column Joints,” PCI Journal, V.22, No.3, pp.84-110.

Schultz, A.E., and Magana, R.A., (1996) “Seismic Behavior of Connections in Precast Concrete Walls,” Proceedings, Mete A. Sozen Symposium, SP-162, American Concrete Institute, Farmington Hills, MI, pp. 273-311.

SDI (2004), *Diaphragm Design Manual,* 3rd Edition, No. DDMO3, Steel Deck Institute, Fox River Grove, IL.

SEAOC, 1999. Recommended Lateral Force Requirements and Commentary. Structural Engineeers Association of California, Sacramento, California.

SEAONC, 2005. *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up Buildings and Other Rigid Wall/Flexible Diaphragm Structures*, Sacramento, California: SEAOC.

Sherwood and Stroh. 1989. “Wood-Frame House Construction” in *Agricultural Handbook 73*. Washington, D.C.: U.S. Government Printing Office.

Somayaji, Shan. 1992. *Structural Wood Design*. St. Paul, Minnesota: West Publishing Co.

Specification for Unbonded Single Strand Tendons (ACI 423.6-01) and *Commentary* (423.6R-01), American Concrete Institute, Farmington Hills, MI, 2001.

Stalnaker, Judith J., and E. C. Harris. 1996. *Structural Design in Wood,* Second Edition. New York, New York: McGraw-Hill.

Thompson, K.J., and Park, R.,(1980) “Seismic Response of Partially Prestressed Concrete,” Structural Division Journal, ASCE, V.106, No. ST8, pp.1755-1775.

U.S. Department of Agriculture, National Oceanic and Atmospheric Administration. 1971. *San Fernando, California, Earthquake of February 9, 1971*. Washington, D.C.: NOAA.