PROPOSAL 7-2R (2009) & 6-2R (2009)

SCOPE: Part 3 of the *2009 Provisions*

PROPOSAL FOR CHANGE:

Add to Provisions Part 3 the attached white paper addressing wood and cold formed steel light-frame shear walls, "*Shear Wall Load-Deflection Parameters and Performance Expectations.*"

REASON FOR PROPOSAL:

The attached white paper was developed by a joint task group of TS6 and TS7 members. The task group was formed to discuss differences in seismic design and detailing requirements for wood versus cold formed steel (CFS) light-frame shear wall systems.

Because of differences in material behavior and cyclic load response between light wood frame and light cold frame systems, a simple transcription of design requirements for wood to steel and steel to wood is not practical. The approach taken by the joint task group was to identify performance expectations for light-frame seismic force resisting systems to accommodate material specific differences in design methods to achieve the desired performance. The attached white paper includes performance expectations for R=6.5 & 7 systems and R=2 & 2.5 systems.

Shear Wall Load-Deflection Parameters and Performance Expectations

1. Introduction

Light-frame shear wall buildings of wood and cold formed steel (CFS) exhibit both similarities and differences in design, construction, and anticipated seismic performance. This white paper is intended to identify anticipated load-deflection parameters and define performance expectations towards which both wood and CFS standard committees can steer future standard updates and future detailing recommendations.

2. Scope

This white paper addresses wood and CFS light-frame buildings with seismic-force resisting systems designed in accordance with the 2006 edition of the IBC (ICC, 2006) and the 2005 edition of ASCE 7 (ASCE, 2005). The main objective of the ASCE 7 design provisions is to protect the health, safety, and welfare of the general public by minimizing the earthquake-related risk to life. Structural and nonstructural damage can be expected as a result of the "design ground motions," since ASCE 7 allows inelastic response in the structural system. For ground motions in excess of the design levels, the intent of these design provisions is for the structure to have a low likelihood of collapse.

3. R = 6.5 &7.0 Systems

This section addresses wood and CFS light-frame shear wall systems with wood structural panel sheathing, which are assigned seismic response modification coefficient, R, of 6.5 for bearing wall systems, and 7.0 for building frame systems. Details of design and construction are to be in accordance with the *Special Design Provisions for Wind and Seismic* (SDPWS) (AF&PA, 2005) for wood construction, and AISI S213-07, *Standard for Cold-Formed Steel Framing – Lateral Design* (AISI, 2007), for CFS construction.

3.1 Analysis model. ASCE 7 equivalent lateral force or simplified seismic design procedures for determination of seismic demand are intended to be used with an analysis model that includes designated portions of the seismic-force resisting systems, sheathed with wood structural panels.

3.2 Vertical shear wall element parameters. Table 3.2A documents observed load deflection behavior and related parameters in site-built wood structural panel shear walls. Behavior that varies from these parameters is outside the scope of this paper.

Parameters given in Table 3.2A are derived from wood structural panel sheathed shear walls with wood framing and tested using the CUREE protocol (CUREE, 2001a). In particular, parameters one through three are taken from a database of 50 wood-frame shear walls assembled by the AC322 Seismic Equivalency Task Group (2007). Data from the following test reports are included: Martin, Skaggs & Keith (2005), Martin (2004), Martin & Skaggs (2003), Martin (2002), Rosowsky, Elkins & Carroll (2004), Pardoen et al. (2003)

Variation in the parameters is known to occur within the broad range of wood structural panel shear walls, based on differences in framing type, framing design, detailing, fasteners, and with testing protocols that vary from the CUREE protocol such as the sequential phased displacement protocol (SPD) (SEAOSC, 1997). See Commentary Section C3.2 for further discussion.

Table 3.2A Vertical Shear Wall Element Parameters[1]

Vertical Shear Wall Element Parameter	Value
1. Ratio of peak capacity (V_U) to ASD design capacity (V_{ASD})	2.5 to 5.0[2]
2. Minimum ratio of drift at 0.80 V_U post peak capacity ($\Delta_{0.8VU}$) to drift at ASD design capacity (Δ_{ASD})	11
3. Minimum drift at 0.80 V_U post peak capacity ($\Delta_{0.8VU}$)	0.028h[3]
4. Drift at peak element strength (V_U)	0.01h to 0.04h
5. Minimum equivalent viscous damping in a single loading cycle reaching peak strength (V_U)	15% of critical

[1]h= story clear height.
[2]Where the ratio exceeds 5, vertical and lateral element detailing must consider the effect of additional overstrength.
[3]This value is the minimum drift at 0.80 V_U post peak capacity for elements with a drift at peak element strength less than 0.028h. This minimum drift at residual strength should always be greater than the drift at peak strength (see Figure 3.2-1).

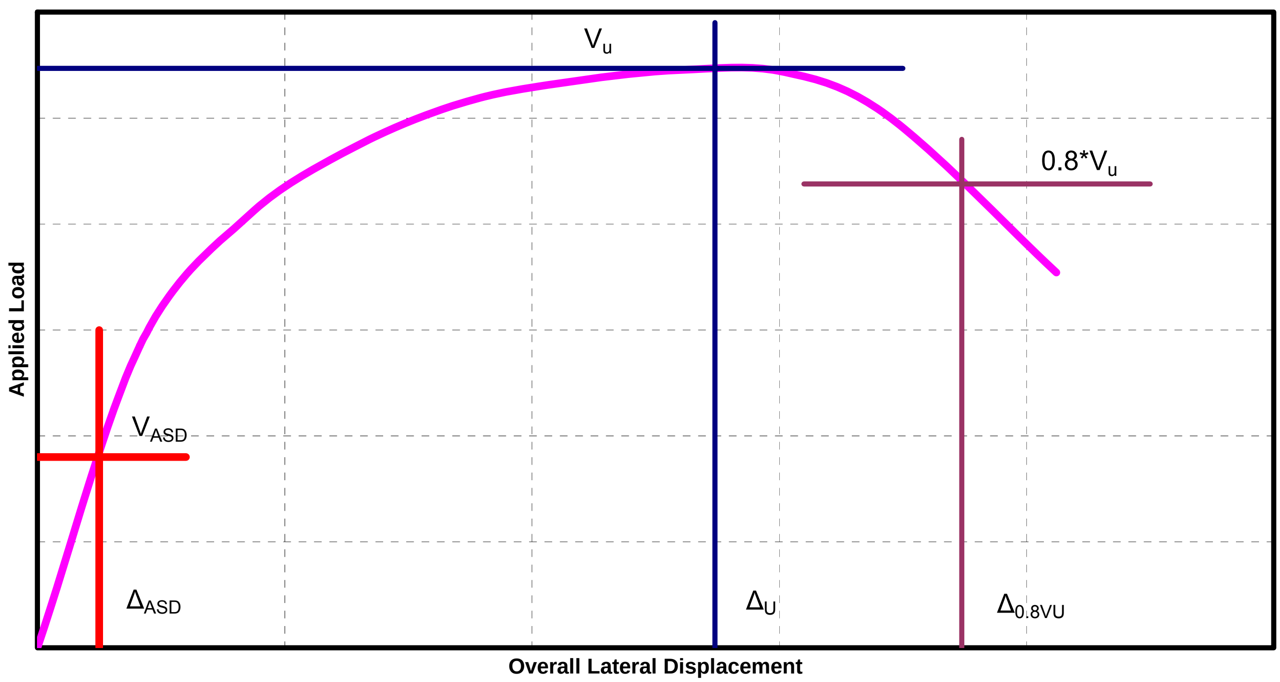

Figure 3.2-1 Vertical shear wall element parameters

3.3 Vertical shear wall element performance expectations. The following recommendations for vertical shear wall elements are intended to allow development of a yield mechanism in the sheathing to framing connection to enable performance as specified in Section 3.2:

3.3.1 Shear wall shear capacity is intended to act as the weak link in the shear wall assembly.

3.3.2 Vertical boundary member tension strength should not act as a weak link in the shear wall assembly. Boundary member design for tension should address reduced net sections and accumulated tension from multiple stories.

3.3.3 Vertical boundary member compression strength should not act as a weak link in the shear wall assembly. Boundary member design for compression should include consideration of member buckling, and transfer of compression loads in and out of compression members.

3.3.4 Boundary member tension connections between elements and to the foundation should not act as a weak link in the shear wall assembly.

3.3.5 Shear transfer connections between elements and to the foundation should not act as a weak link in the shear wall assembly.

3.3.6 Collector members, splices in collectors, and connection of collectors to vertical shear wall elements should not act as a weak link in the shear wall assembly.

3.3.7 Boundary member and connection deformation should be accounted for in shear wall design and detailing.

3.4 Estimated peak unit shear capacity, v_U.

For CFS elements, the shear wall peak unit shear capacity, v_U, is intended to be determined from testing. If the SPD protocol is used to determine the nominal unit shear strength for wood structural panel shear walls with CFS framing, the unit shear strength should be increased by 20 % (Boudreault, 2005). In addition, if the stabilized backbone curve is used, the unit shear strength should be increased another 10%. If the CUREE protocol is used, no adjustment is required. The shear wall peak unit shear capacity, v_U, may alternately be estimated as the AISI tabulated nominal seismic unit shear strength times 1.3.

For wood elements, the shear wall peak unit shear capacity, v_U, is intended to be determined from monotonic or cyclic (CUREE, 2004) testing, but may also be taken from nominal unit shear values set forth in Table 4.3A column B of AF&PA Special Design Provisions for Wind and Seismic (SDPWS).

4. R=2.0 & 2.5 Systems

In ASCE 7, wood and CFS light-frame shear walls are assigned seismic response modification coefficients, R, equal to 2.0 or 2.5, if they are sheathed with other than wood structural panels or steel sheets. Sheathing materials may include gypsum wallboard, interior plaster, exterior three coat Portland cement plaster (stucco), fiberboard, particleboard, and diagonal lumber sheathing, when permitted by the applicable AF&PA or AISI standard. Section 4 applies where the R = 2.0 or 2.5 system is used in the building direction under consideration, in each story from the level under consideration to the roof; mixed seismic-force resisting systems are beyond the scope of this paper.

4.1 Analysis model. ASCE 7 equivalent lateral force or simplified seismic design procedures for determination of seismic demand are intended to be used with an analysis that includes designated portions of the seismic-force resisting system. All vertical elements considered in the analysis model are intended to meet the aspect ratio, design and detailing requirements of the applicable AF&PA or AISI standard.

4.2 Vertical shear wall element parameters. Table 4.2A documents observed load deflection behavior and related parameters in site-built light-frame shear walls sheathed with other than wood structural panels (and other than sheet steel). Parameters are given in Table 4.2A for both the CUREE protocol and the SPD protocol. Behavior that varies from these parameters is outside the scope of this paper.

Table 4.2A Vertical Shear Wall Element Parameters[1]

Vertical Shear Wall Element Parameter	Value
1. Minimum drift (Δ_U) at peak element strength (V_U)	0.0025h
2. Minimum ratio of peak capacity (V_U) to ASD design capacity (V_{ASD})	2.0

[1]h= story clear height.

4.3 Vertical shear wall element performance expectations. Adequate seismic performance of buildings designed using shear walls sheathed with other than wood structural panel shear walls (or steel sheets, also an R=6.5 system) is almost entirely dependent on shear wall element strength, rather than ductility. Building drift demands are anticipated to be significantly less than those for R = 6.5 or 7 buildings. To accommodate this behavior, it is recommended that the nominal strength of the following be adequate to match or exceed the peak capacity, V_U, of the shear wall sheathing:

4.3.1 Boundary member tension connections between elements and to the foundation.
4.3.2 Shear transfer between elements and to the foundation.
4.3.3 Collector member splices and connections to vertical elements.
4.3.4 Members and connections supporting discontinued shear walls or frames (revises ASCE 7 Section 12.3.3.3)

5. Details of Construction

Seismic performance of light-frame shear walls requires attention to details of construction and quality assurance. Provisions for construction and quality assurance are incorporated in the AFPA, AISI and ASCE 7 standards and the model building codes.

Commentary

C1 Introduction

This white paper has been developed by a joint task group of TS6 and TS7 members, in recognition of both similarities and differences in wood and cold formed steel (CFS) light-framed shear wall systems with wood structural panel sheathing. Because of differences in material behavior and cyclic load response, a simple transcription of design requirements for wood to steel and steel to wood is not practical. The approach taken is to identify performance expectations for light-frame shear wall seismic force resisting systems; material specific differences in design method can then be accommodated in future development of approaches to achieving the desired performance. The white paper includes performance expectations for R=6.5 & 7 systems and R=2 & 2.5 systems.

C2 Scope

This white paper focuses on recommending performance expectations for wood and CFS light-frame shear walls sheathed with wood structural panels, as currently included in ASCE 7. Questions regarding appropriateness of the assigned R-factors are beyond the scope of this paper. Steel sheet shear walls are beyond the scope of this paper.

C3 R = 6.5 & 7.0 Systems

Section 3 addresses wood structural panel sheathed shear wall systems. It is intended that, except as specifically addressed in this paper, these systems be designed in accordance with ASCE 7-05, AF&PA *Special Design Provisions for Wind and Seismic* for wood construction, and AISI S213-07, *North American Standard for Cold-Formed Steel Framing – Lateral Design,* for CFS construction. Because the R factors assigned to these systems are relatively high, significant inelastic behavior is anticipated in design level seismic events, including potential for loading in the range of post-peak-capacity deflections.

The typical wood light-frame building responds to a seismic event by racking the wall elements while the floor and roof diaphragms remain close to elastic. Consequently, the walls largely determine the seismic response characteristics of light-frame construction. In R = 6.5 & 7.0 wood structural panel shear wall systems, sheathing is most commonly installed in 4 foot by 8 to 10 foot sheets and fastened to wall framing with nails (for wood frame) and screws (for CFS). The primary source of seismic drift and energy dissipation of wood shear walls is the bending and yielding of the shear wall sheathing to framing fasteners around the perimeter of each sheathing panel, accompanied by slip between the sheathing and framing. In CFS frame shear walls, drift and energy dissipation are generally related to the tilting (rotation) of the sheathing fasteners as well as bearing deformations in the wood structural panel or steel adjacent to the connections; again the deformations at the sheathing fasteners are accompanied by slip between the sheathing and framing.

This white paper is intended to only address the combinations of sheathing and fastening currently included in the AF&PA and AISI standards. This is because the cycled load behaviors of these combinations are known to provide for required inelastic behavior. Other combinations of sheathing and fastening and other methods of attachment should be tested in reverse-cyclic loading.

C3.1 Analysis model. Virtually all wood or CFS shear wall buildings are designed for seismic loads using the ASCE 7 equivalent lateral force method or simplified method. Analysis for these systems includes vertical wall elements that are designated to be part of the seismic-force resisting system. For R= 6.5 & 7 systems, all designated shear walls will be sheathed with wood structural panel sheathing. Recent seismic studies have affirmed that, for these buildings, the strength and stiffness contribution of finish materials and partition walls plays a significant role in the seismic performance of these buildings. Despite this understanding, analysis models used to evaluate and distribute seismic demand are intended to only include designated wood structural panel shear walls. This does not preclude consideration of the effect of walls sheathed with other than wood structural panels when evaluating a building for presence of structural irregularities.

C3.2 Vertical shear wall element parameters. The parameters addressed in this section are intended to allow discussion of hysteretic behavior for site-built wood structural panel shear walls, from which performance expectations and detailing recommendations can follow. The parameters in Table 3.2A were accepted by the authors of this white paper for wood structural panel shear walls with wood framing; they were accepted as interim values for wood structural panel shear walls with CFS framing until a data base of testing with CFS members is developed.

The parameters are not intended to be used to assign R-factors to vertical shear wall systems, nor are they intended to address prefabricated shear wall elements. Vertical elements whose parameters do not conform to those described in Table 3.2A are outside the scope of this paper.

Parameters one through three in Table 3.2A mirror parameters recommended by the AC322 Seismic Equivalency Task Group (AC322, 2007). The values are derived from a group of 50 wood light-frame shear walls sheathed with wood structural panel sheathing nailed to wood framing and tested using the CUREE protocol. The tabulated numbers for parameters two and three are the average values minus one standard deviation.

> Table 3.2A, Parameter 1. Provision of overstrength beyond ASD design capacity is understood to be fundamental to earthquake performance of buildings braced with wood structural panel shear walls.
>
> Table 3.2A, Parameter 2. Deformation capacity at peak strength well beyond deformation capacity at design level is understood to be fundamental to earthquake performance of buildings braced with wood structural panel shear walls.
>
> Table 3.2A, Parameter 3. Testing and analysis suggests that wood structural panel sheathed shear walls are capable of supporting post-peak loading. At a post peak capacity of 80% of peak, the shear wall element drift of not less than 0.028h is expected.

Table 3.2A, Parameter 4. The range of vertical element drift recognizes both the allowable story drift permitted by ASCE 7, and some variation of vertical shear wall elements above and below this drift.

Table 3.2A, Parameter 5. The criterion looks at one cycle in which the shear wall element reaches peak strength. If the CUREE protocol is used, this would be anticipated to be in the range of the 6^{th} to 8^{th} loading cycle. The areas within the curve for both positive and negative excursions are intended to be summed and an equivalent viscous damping ratio calculated. See Filiatrault et al. (2003) for details of equivalent viscous damping calculation.

C3.3 Vertical Shear Wall Element Performance Expectations. Performance expectations in this section are intended to support the sheathing to framing fastening as the primary sources of inelastic behavior and source of energy dissipation in the vertical shear wall elements. Failure of the boundary members or connections addressed in items 3.3.2 to 3.3.6 could cause a more critical and possibly sudden and brittle failure of the vertical shear wall element. In general, further study is needed to determine whether or not the desired behavior requires detailing provisions beyond those currently required by ASCE 7, SDPWS, and AISI S213.

C3.3.2 Vertical boundary member tension design. Tension boundary members should be sized such that they are not the weak link in the shear wall assembly. For wood boundary members, it is recognized that most members will be stronger than the calculated nominal capacity due to the 5% basis of reference wood member strength properties in underlying design and product standards. It is also recognized that tension post capacity in use will be sensitive to placement of knots and other characteristics because the strength controlling characteristic of the wood member is not always located in the area of maximum tension force.

Section 3.3.2 includes a reminder that net tension at reduced net sections and accumulated tension from multiple stories need to be considered. In addition, tension member design should also include consideration of flexure due to the eccentricity of the tie-down load.

C3.3.3 Vertical boundary member compression design. Compression boundary members should be sized such that they are not the weak link in the shear wall assembly. See also wood comments in C3.3.2.

C3.3.4 Boundary member tension connections between elements and to the foundation. Tension connections for boundary members are required as part of a complete load path through the building. This includes both tension connections from boundary members above to boundary members below and anchorage to the foundation. Tension connections use tie-down brackets, steel straps, or continuous rod or cable systems. Again, the tension connection should not be the weak link in the shear wall element.

C3.3.5. The lateral forces at the foundation are resisted by a distributed connection along the shear walls oriented parallel to and in the plane of the load. These connections are most commonly anchor bolts at foundation sill plates and nailing or sheet metal angles at framed floors. This connection is typically designed to be independent of the connections used to resist the overturning forces (i.e. resists only horizontal shear and not tension due to overturning). This connection should be designed such that it is not the weak link in the shear wall system.

C3.3.6 Collectors. There has been a lack of observed failures in light-frame collector elements. To ensure that the failure is not in the collector, however, the connection of the collector to the shear wall, or the splice of the collector should be designed such that they are not the weak link in the shear wall system.

C3.3.7 Boundary member and connection deformation. Excessive deformation of boundary members and their connections can lead to the premature failure of sheathing to framing fastening due to large imposed deformations. See discussion in commentary sections 12.2.3.11 & 12.2.3.12 of the 2003 NEHRP Provisions (FEMA, 2003).

C3.4 Estimated peak unit shear capacity, v_U. It is intended that estimated values of the peak unit shear capacity, v_U, be used in evaluating the performance expectations of Section 3.3. Because referenced testing of wood-frame shear walls uses the CUREE protocol, it is not anticipated that conversion of peak unit shear capacity from other protocols will be needed. Under no circumstances is it intended that tests conducted using the SPD protocol be converted to compare to the five parameters of Table 3.2A, since accurate adjustments of all five parameters are not available.

C4 R=2.0 & 2.5 Systems

In ASCE 7, wood and CFS light-frame walls are assigned seismic response modification coefficients, R, equal to 2.0 or 2.5, if they are sheathed with other than wood structural panels or steel sheets. Specifically, R, C_{d}, and Ω_0, are 2.0, 2.5, and 2.0 for bearing wall systems, and 2.5, 2.5 and 2.5 for building frame systems. This commonly includes sheathing with gypsum wallboard, interior plaster, exterior three coat Portland cement plaster (stucco), particle board, fiberboard and diagonal lumber sheathing. This may also include wood structural panel sheathing used alone or in combination with other sheathing materials.

Adequate seismic performance of buildings designed using shear walls sheathed with other than wood structural panels (or sheet steel) is almost entirely dependent on shear wall element strength rather than ductility. As ductility is replaced with strength, reliability of the bracing system becomes highly dependant on adequate capacity, adequate detailing, and adequate understanding of seismic demand. For the materials currently included in the AISI and AF&PA standards, there is a level of comfort with design for R=2 or 2.5 systems. This comes both from a history of design of these systems using R=4.5 under the Uniform Building Code (ICBO, various) and recent analytical studies suggesting generally adequate performance with R=2 design (ATC, 2007).

C4.1 Analysis model. Since a low R-value is being utilized, the level of inelastic response is assumed to be lower than if an $R \geq 6.0$ were to be used. Therefore, the combination of the relatively brittle finish materials with the more ductile wood structural panel walls is allowed, provided the R of 2 or 2.5 is used for all vertical elements. This level of design is often used when the building design has a relatively large number of interior walls that will be used as resistance, since interior walls are usually sheathed with brittle materials such as gypsum wallboard.

C4.2 Vertical shear wall element parameters. The vertical shear wall elements included in this group have widely varying load-deformation characteristics. In general, however, it is anticipated that they have much less ductility and deformation capacity, and more rapid post-peak drop in capacity than wood structural panel sheathing.

Table 4.2A Parameter 1. The minimum drift criterion of 0.0025h recognizes that a building braced with these shear wall types will have measurable drift during design level earthquake loading.

Table 4.2A Parameter 2. It is anticipated that shear walls using sheathing materials currently assigned R = 2 or 2.5 have ratios of ASD to peak capacity of 2 or higher.

CUREE (CUREE, 2001a) and SPD (SEAOSC, 1997) protocols are combined in Table 4.2A because there is not sufficient test data to identify separate parameters for each.

C4.3 Performance expectations. For this group of shear wall sheathing materials, the failure would ideally be the sheathing fastening to framing (or the sheathing material) rather than the boundary members or their connections. This preferred failure would allow development of the sheathing fastening capacity, and avoid what might be a more critical failure mode such as sliding or overturning of the wall framing. As a step towards achieving this, the intent of Section 4.3 is to have the sheathing peak shear capacity and the capacity of the boundary members and their connections balanced or close to balance.

For sheathing materials having an ASD to peak strength ratio of approximately 2 from ASD capacity to peak capacity, standard ASD or LRFD detailing practice is thought to provide boundary member connection factors of safety adequate to support failure in the sheathing. Factors of safety of 2.1 to 5 might be anticipated for individual fasteners. Factors of safety of 2.5 to 3 are commonly anticipated for prefabricated connectors. It is anticipated, however, that some connections such as steel straps that are controlled by steel net section might have lower factors of safety.

For sheathing materials having an ASD to peak strength ratio greater than 2, it is recognized that failure may potentially occur in connections of boundary members. At this time recommendations to design connections to support the higher ratios is viewed as too stringent.

Boundary members (shear wall chords and collector members) are specifically excluded from the detailing list because these members are not viewed as possible weak links given the current range of tabulated unit shears for the sheathing materials. If sheathing members with higher units shears are to be used, boundary member design should be reconsidered.

References:

AC322 Seismic Equivalency Task Group, 2007. *Methodology for Seismic Coefficient Equivalency, Presented to ICC ES July 31, 2007*, Ronald Nelson, Chair, Hermosa Beach, CA.

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Martin, Z. 2004. Wood Structural Panel Lateral and Shear Wall Connections with Common, Galvanized Box and Box Nails (APA Technical Report T2004-14), APA The Engineered Wood Association, Tacoma, WA.

Martin, Z. T. Skaggs, and E. Keith, 2005. Using Narrow Pieces of Wood Structural Panel Sheathing in Wood Shear Walls (APA Technical Report T2005-08), APA The Engineered Wood Association, Tacoma, WA.

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Rosowsky, D. L. Elkins, and C. Carroll, 2004. Cyclic Tests of Engineered Shear Walls Considering Different Plate Washers, Oregon State University Report for the American Forest and Paper Association, Corvallis, OR.

SEAOSC, 1997. *Standard Method of Cyclic (Reversed) Load Test for Shear resistance of Framed Walls for Buildings*, Structural Engineers Association of Southern California Ad Hoc committee on Testing Standards for Structural Systems and components in conjunction with the City of Los Angeles.

TS 6 VOTE:

Y=5 YR=1 N=1 NV=1

TS 6 COMMENTS:

Chia Ming Uang (N)
According to Section 2, the scope of this white paper addresses wood and CFS light-frame buildings with seismic-force resisting systems designed in accordance with the 2006 IBC and the 2005 ASCE 7. Limiting values or ranges of several key performance parameters of an expected response curve are then tabulated. The potential value of this white paper is to provide a guideline for classifying the light-frame seismic systems, especially those recently developed (proprietary) systems. But the systems presented in this paper appear rather narrow in scope. For example, Section 3 for the high ductility system addresses wood and CFS light-frame shear wall systems with wood structural panel sheathing only. Does it mean these systems, as contained in the current IBC/ASCE 7, are questionable in performance? If not, why is it necessary to single these systems out and, more significantly, to "envelope" the expected performance? It appears reasonable to believe that future development or innovation will come up some systems that may be demonstrated, for example through the ATC-63 process, to be acceptable for seismic applications, yet the response curve may not fall in the (narrow) range specified in this paper.
RESPONSE: Non-persuasive. This white paper is specifically addressing site-built shear wall systems, and not including proprietary shear wall systems. Proprietary systems are being considered through the ICC evaluation services in AC 322; a separate technical subcommittee is advising the ICC effort. All sheathing materials permitted by IBC are addressed: wood structural panel systems in Section 3, and other sheathing materials permitted by the IBC in Section 4. It is understood that the parameters included in this white paper may be changed in the future due to additional research.

Bonnie Manley (YR)

- Section 3.2, Modify as follows:
  3.2 Vertical shear wall element parameters. Table 3.2A documents ~~envisioned~~idealized load deflection behavior and related parameters in site-built wood structural panel shear walls. If behavior varies from these parameters, the applicability of the Section 3.3 performance expectations should be further evaluated. What are we saying here – could this be misinterpreted to mean that anything falling below idealized does not qualify for R=6.5. When we present just 50 tests meeting this criteria, are we misleading people into thinking that most of the existing tests are not ideal and should not qualify for R=6.5.

  RESPONSE: Persuasive, however the word "observed" was chosen by TS7.

  Parameters given in Table 3.2A are derived from wood structural panel sheathed shear walls with wood framing and tested using the CUREE protocol (CUREE, 2001a). In particular, parameters one through three are taken from a database of 50 wood-frame shear walls assembled by the AC322 Seismic Equivalency Task Group

(2007). Variation in the parameters might occur with wood structural panel shear walls and cold-formed steel framing, and with testing using the sequential phased displacement (SPD) (SEAOSC, 1997) or other protocols. See Commentary Section C3.2 for further discussion.
RESPONSE: Persuasive, editorial

- Section 3.4, Available Information on Meeting Performance Expectations: This seems to be a new section and I have the following questions:
    - This is not what we agreed to at our last meeting, unless my memory has failed me. Why the change in direction? I was asked to develop a comprehensive list on wood shear panel testing on cfs framing, which was provided but not used.
    - I'm not convinced that this language is located in the correct place – it references information in both Sections 3 and 4, yet it comes at the end of Section 3. Shouldn't it come after Section 4? It is also incorrectly numbered – as it stands now, there are two sections labeled as 3.4.
    - I'd like to see a stronger introduction to this section. Both of the sentences use the word 'illustration' – they seem to be a bit redundant and don't really clarify our intent.
    - Three years from now, will the average reader be able to track down the "AC322 data base"? Has it been published or is there a plan to publish this information? If not, are we comfortable referencing an unpublished report?
    - I'm not too familiar with how the tests were chosen for inclusion in the AC322 data base and report. I imagine that there have been thousands of tests with wood structural panels on wood studs. Are these the only ones that passed the criteria? Or where these the only ones that the group had time to analyze? I thought that the AC322 TG had a very narrow focus – they were asked to develop an acceptance criteria for proprietary products being used in CFS and wood framed construction. Are we completely comfortable extrapolating their recommendations to all light framed construction?
    - There doesn't seem to be any commentary on this new language.

  RESPONSE: Persuasive. The information in section 3.4 has been moved to section 3.2, and introductory language revised.

TS 7 VOTE:

Y=10 YR=1 N=1(withdrawn 1/14/08) NV=2

TS 7 COMMENTS:

Andre Filiatrault (Y)

Technical Comments

Section 3.4, Lines 17 to 19, Page 4: From this paragraph, it appears that the increase in strength of 20% if the SPD protocol is used instead of the CUREE protocol applies only

to CFS elements as described in the previous paragraph (Lines 10 to 15). Comparative testing by Gatto and Uang[1] during the CUREE-Caltech Woodframe Project has shown the same effect of the SPD protocol for wood light-frame shear walls. This paragraph should be modified to include the correction for wood elements as well.
RESPONSE: Non-persuasive. The wood testing being referenced is already uses the CUREE protocol, so no conversion is needed. TS7 felt that allowing conversion of SPD tests could lead to miss-use, so it is preferable not to discuss.

Editorial Comments
Page 4, Line 12: “(Boudreault, 2005)” instead of “(Boudreault, 2005))”
RESPONSE: Persuasive, editorial.
Page 6, Line 17: The word “with” seems to be missing between “sheathed wood”
RESPONSE: Persuasive, editorial.
Pages 6 to 12: The fonts should be harmonized throughout the Commentary. Some sections uses 12 pts fonts while others use 10 pts fonts.
RESPONSE: Persuasive, editorial.
1 Gatto, K, and Uang, C (2002). “Cyclic Response of Woodframe Shearwalls: Loading Protocol and Rate of Loading Effects,” CUREE-Caltech Woodframe Project Report No. W-13, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.
RESPONSE: Non-persuasive, reference not needed.

Marjan Popovski (Y)
At the end of C4.1 a version of the following statement can be included:
"The Canadian Standard for Engineering Design in Wood (CSA O86, 2001) contains design provisions for shearwalls with wood-based panels on one side and gypsum wallboard on the other. Research results on this topic are available in Ceccotti and Karacabeyli 2002."
This paper is also attached to this E-mail. Both documents should also be included in the reference section.
RESPONSE: Non-persuasive, similar references are not included and are not needed.

Vladimir Kochkin (NV)
Comments on White Paper
The paper improved significantly and many of concerns raised with my previous negatives as well as concerns raised by others were addressed. However, I cannot fully support the paper due to the remaining issues summarized below. I am not submitting an YR vote because I do not expect some of the issues to be resolved this cycle. I am not submitting a negative because as a white paper it is understood to be an interim document with expectations of further revision/refinement.
RESPONSE: Non-persuasive, this will need to be addressed in future updates.

The document claims to define performance expectations for wood and steel with no information on steel provided. The commentary mentions that steel data is pending and Section 3.2 second paragraph states that “variation in the parameters might occur” for cold-formed steel systems from those tabulated in the paper. Information needed to draw conclusions for both materials is simply incomplete. The value of this document suffers greatly from the lack of data for steel.

RESPONSE: Non-persuasive, this will need to be addressed in future updates.

Because Table 3.2A is the substance of this document and is entirely new, the commentary needs to provide more specific information on how the parameters in Table 3.2A were developed. All details for the tested wall systems used in the dataset that affect the proposed performance parameters should be documented including segment aspect ratios, fastener sizes, spacing, framing lumber, presence of finishes, peak unit shear, etc. At this point, the paper only provides references to publications from which the dataset was compiled. The reader is left in limbo to figure out how the authors of the paper got from those references to the proposed parameters. It needs to be traceable and repeatable. Obviously this work has been already done to develop the proposed parameters. The paper would greatly benefit from documenting the process.
RESPONSE: Partially persuasive. The need for having the basis for parameters available was agreed upon. Explanation of parameters can currently be found in the AC322 task group report to ICC ES and in the AC322 task group data base. A Journal paper describing the parameters is currently being prepared by several individuals.

<u>Parameter 3</u>—Minimum Drift at 0.80Vu of 0.028h. According to the commentary, this parameter is driven by the need to sustain forces at deformation beyond the peak load. However, this commentary language does not reconcile with Parameter 4 that allows peak load to occur at drift of 0.04h, which exceeds the minimum post peak capacity drift requirement of 0.028h. Commentary language should be revised to be consistent with the proposed parameters.
<u>Parameter 5</u>—Minimum equivalent viscous damping (EVD). Although EVD is often calculated in research test papers for light-frame walls, to my knowledge this is the first attempt at codifying this parameter. EVD reflects specific pinching behavior of a system including reloading and unloading stiffnesses. The commentary is silent on the topic of pinching and appears to suggest that the EVD is included to eliminate systems with near elastic behavior. If this is the only reason for including the EVD, Parameter 2 dealing with drift ratios should be sufficient to ensure yielding. A better substantiation should be provided in the commentary with regard to the need for inclusion of this parameter, its effect of system performance, and the proposed minimum value of 15%.
RESPONSE: Partially persuasive. Edits by Phil Line were accepted by commenter as substantially addressing issues.

Gary Mochizuki (Y)
Typo on 7-2, page 10, line 13. It says "currently" but should say "current"
RESPONSE: Persuasive, editorial.

Jeff Ellis (Not voting member)
- Page 3 lines 13 to 22 & page 4 lines 1 to 6: The numbering needs to be corrected for these sections as Section 3.3.1 is repeated.
RESPONSE Persuasive, editorial.

- Page 6 line 5 to 12 and 44 to 46: The font is a different size than the other text on the other lines.
RESPONSE: Persuasive, editorial.
- Page 8 line 20: C3.3.1 commentary not provided.

RESPONSE: Non-persuasive. C3.3 addresses the topic. No additional commentary wording was submitted for consideration.

- Page 8 line 28: Revise to "Section 3.3.2 includes a reminder....." as it currently shows Section 3.3.1.

RESPONSE: Persuasive, editorial.

- Page 9 line 30: The greater than or equal to symbol overlaps with 6.0.

RESPONSE: Persuasive, editorial.

Phil Line (N- withdrawn 1/14/08)

(A)

3.1 Analysis model. ASCE 7 equivalent lateral force or simplified seismic design procedures <u>for determination of seismic resistance </u>are intended to be used with an analysis model that includes designated portions of the seismic-force resisting systems~~, which are to be~~ sheathed with wood structural panels.

Reason: The proposed revision is intended to clarify that the analysis model for determination of seismic resistance is based on the designated structural system so that evaluation of structure for presence of irregularities does not preclude designer consideration of effect of "non-structural" walls on triggering of irregularities such as a weak story or torsional irregularity. Prior language which might have addressed this issue better than my proposed edits are welcome.

RESPONSE: Persuasive. TS7 agreed to wording with "demand" substituted for "resistance." TS7 also agreed that the reason discussion should be incorporated into the commentary for this section.

(B)

3.2 Vertical shear wall element parameters. Table 3.2A documents envisioned load deflection behavior and related parameters in site-built wood structural panel shear walls. ~~If behavior varies from these parameters, the applicability of the Section 3.3 performance expectations should be further evaluated.~~

Reason: The deleted sentence offers only one option aimed at detailing changes; however, we have discussed other possible considerations. For example, if Table 3.2A performance is not achieved where Section 3.3 detailing is provided, 1) designation as R=6.5 may need to be revisited, or 2) Table 3.2A performance expectations need to be revised to better capture the range of performance intended to reflect R=6.5. Because standards committees may take different action and given the preliminary nature of the white paper, I would suggest that we keep the performance expectations as described and drop cursory guidance on what to do if there is variation from the expected performance.

RESPONSE: Persuasive. TS7 agreed to strike the noted sentence, but add in "Behaviors that vary from these parameters is outside the scope of this paper."

(C)

Parameters given in Table 3.2A are derived from wood structural panel sheathed shear walls with wood framing and tested using the CUREE protocol (CUREE, 2001a)<u> that exhibited yielding of sheathing to frame nails prior to onset of failure mechanisms such as sheathing tear-</u>

out, nail head pull through, wood frame member failure, and framing member connection failure. In particular, parameters one through three are taken from a database of 50 wood-frame shear walls assembled by the AC322 Seismic Equivalency Task Group (2007). Variation in the parameters ~~might~~ is known to occur within the broad range of wood structural panel shear walls based on differences in framing type, framing design, detailing, fasteners, and ~~steel framing, and~~ with testing protocols that vary from the CUREE protocol such as ~~using the~~ sequential phased displacement (SPD) (SEAOSC, 1997) or other protocols. See Commentary Section C3.2 for further discussion.

Reason: Further clarify that the performance parameters depicted are based on desired yielding of nails prior to onset of other less desirable wood failure mechanisms. Description of this performance trait is essential to our understanding of wood-frame wood structural panel behavior in the data base of walls and it also expands on discussion located in the commentary. Unless the data set of walls is specifically defined along with test protocol, variations in parameters are already known to occur. SPD testing will result in lower bound strengths than set by the current 2.5 level and ductility ratios which differ from the listed levels.

RESPONSE: partially persuasive. Modifications to first sentence were found non-persuasive. Modifications to third sentence were agreed to.

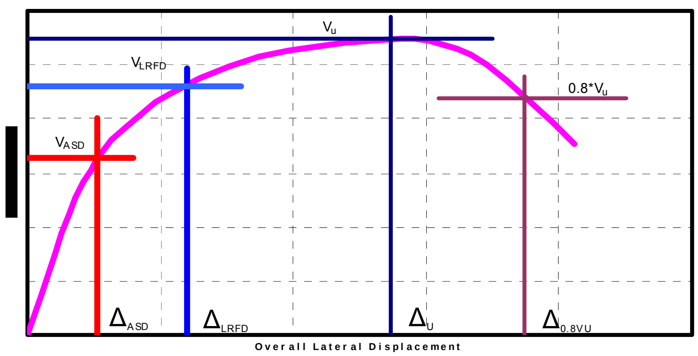

Figure 3.2-1 Vertical shear wall element parameters

(D)

Comment: Revise scale of Figure 3.2-1. V_ASD should be located lower on the curve to better show a minimum Vu/V_ASD = 2.5 per Table 3.2A. V_LRFD can then be shown at a level higher than V_ASD based on VLRFD/V_ASD = 1.4.

RESPONSE: Persuasive. Revisions will be made.

(E)

3.3 Vertical shear wall element performance expectations. The following recommendations for vertical shear wall elements are intended to allow development of ~~anticipated shear wall~~ of a yield mechanism in the sheathing to frame connection to enable performance as specified in Section 3.2:

Reason: Detailing is intended to enable the yield mechanism in the sheathing to frame connection. Proposed text expands on Commentary discussion and places it in the body of the White Paper.

RESPONSE: Persuasive.

3.3.1 Shear wall shear capacity is intended to act as the weak link in the shear wall assembly.

3.3.8 Vertical boundary member tension strength should not act as a weak link in the shear wall assembly. Boundary member design for tension should address reduced net sections and accumulated tension from multiple stories.

3.3.9 Vertical boundary member compression strength should not act as a weak link in the shear wall assembly. Boundary member design for compression should include consideration of member buckling, and transfer of compression loads in and out of compression members. ~~For CFS, both local and global member buckling should be addressed.~~

Reason: Not sure it is necessary to single out CFS in this case. Design may include consideration beyond local and global member buckling.

RESPONSE: Persuasive.

(F)

4.1 Analysis model. ASCE 7 equivalent lateral force or simplified seismic design procedures for determination of seismic resistance are intended to be used with an analysis that includes designated portions of the seismic-force resisting system. All vertical elements considered in the analysis model are intended to meet the aspect ratio, design and detailing requirements of the applicable AF&PA or AISI standard.

Reason: Revision clarifies that the analysis model for seismic resistance is based on the designated structural system so that evaluation of structure for presence of irregularities does not preclude consideration of effect of "non-structural" walls on triggering of irregularities such as a weak story or torsional irregularity.

RESPONSE: Persuasive. Modifications to match comment A.

(G)

4.2 Vertical shear wall element parameters. Table 4.2A documents envisioned load deflection behavior and related parameters in site-built light-frame shear walls sheathed with other than wood structural panels (and other than sheet steel). ~~If behavior varies from these parameters, the applicability of the Section 4.3 performance expectations should be further evaluated.~~ Parameters are given in Table 4.2A for both the CUREE protocol and the SPD protocol; modification of parameters may be required for other protocols.

Reason: If performance varies then consideration of whether the parameters are correct or R = 2.5 is applicable are also likely options. Given that standards committees may take different action and due to the preliminary nature of the parameters given, suggest dropping cursory guidance on what is needed if performance parameters are not met.

RESPONSE: Persuasive. Modifications to match comment B.

(H)

4.3 Vertical shear wall element performance expectations. Adequate seismic performance of buildings designed using shear walls sheathed with other than wood structural panel shear walls (or steel sheets, also an R=6.5 system) is almost entirely dependent on shear wall element strength, rather than ductility. Building drift demands are anticipated to be significantly less than those for R = 6.5 or 7 buildings. To accommodate this behavior, it is recommended that the nominal strength of the following be adequate to match or exceed the peak capacity, V_U, of the

shear wall sheathing:
Reason: Clarify intent to use nominal strength.
RESPONSE: Persuasive.

Commentary

(I)
C3 R = 6.5 & 7.0 Systems
…….
The typical wood light-frame building responds to a seismic event by racking the wall elements while the floor and roof diaphragms remain close to elastic. Consequently, the walls largely determine the seismic response characteristics of light-frame construction. In R = 6.5 & 7.0 wood structural panel shear wall systems, sheathing is most commonly installed in 4 foot by 8 to 10 foot sheets and fastened to wall framing with nails (for wood frame)~~,~~ and screws (for CFS)~~, or similar small diameter dowel-type fasteners~~. The primary source of seismic drift and energy dissipation of wood frame shear walls is the bending and yielding of the shear wall sheathing to framing nails ~~fasteners around the perimeter of each sheathing panel,~~ accompanied by slip between the sheathing and framing. In CFS frame shear walls, drift and energy dissipation are generally related to the tilting (rotation) of the sheathing fasteners as well as bearing deformations in the wood structural panel or steel adjacent to the connections~~; again the deformations at the sheathing fasteners are~~ accompanied by slip between the sheathing and framing.
Reason: More specifically describe yield mechanisms for wood frame and CFS to avoid confusion over whether screws are predominantly used in wood frame and nails are used in CFS. I'm not aware of other similar small diameter fasteners covered by the underlying design standards. Note that the term "slip" is often used to describe load-deformation response of nailed-connections in wood versus those with screws.
RESPONSE: Persuasive. Parenthetical phrases to be added. Balance of section to follow Steve priors recommended edits.

(J)
C3.1 Analysis model. Virtually all wood or CFS shear wall buildings are designed for seismic loads using the ASCE 7 equivalent lateral force method or simplified method. Analysis for these systems includes vertical wall elements that are designated to be part of the seismic-force resisting system. For R= 6.5 & 7 systems, all designated shear walls will be sheathed with wood structural panel sheathing. Recent seismic studies have affirmed that, for these buildings, the strength and stiffness contribution of finish materials and partition walls plays a significant role in the ~~acceptable~~ seismic performance of these buildings. ~~See further discussion in NEHRP Provisions Part 2 commentary~~. Despite this understanding, analysis models used to evaluate and distribute seismic demand are intended to only include designated wood structural panel shear walls.
Reason: Mostly editorial and to avoid referencing Part 2 commentary at this time as its contents are not yet known.
RESPONSE: Persuasive.

(K)

C3.2 Vertical shear wall element parameters. The parameters addressed in this section are intended to allow discussion of hysteretic behavior for site-built wood structural panel shear walls, from which performance expectations and detailing recommendations can follow. The parameters in Table 3.2 were accepted by the authors of this white paper for wood structural panel shear walls with wood framing; they were accepted as interim values for wood structural panel shear walls with CFS framing until a data base of testing with CFS members is developed.

The parameters are not intended to be used to assign R-factors to vertical shear wall systems, nor are they intended to address prefabricated shear wall elements. ~~Vertical elements whose parameters do not conform to those described in Table 3.2A require careful examination of the applicability of detailing performance expectations described in Section 3.3.~~
RESPONSE: Persuasive – will be modified to correspond to comment B.

> Table 3.2A, Item 1. Provision of overstrength beyond ASD design capacity is understood to be fundamental to earthquake performance of buildings braced with wood structural panel shear walls. The values in Table 3.2 mirror values recommended by the AC322 Seismic Equivalency Task Group (AC322, 2007). The values are derived from a group of 50 wood ~~light-~~frame shear walls sheathed with wood structural panel sheathing nailed to the wood frame and tested using the CUREE protocol. ~~The tabulated numbers are the average values plus and minus one standard deviation.~~
> *Reason: Clarify use of nailed wood structural panel shear walls versus stapled or attached by other means. Overstrength is one parameter where the upper threshold did not follow the plus and minus one standard deviation target due to known presence of greater overstrength in narrow walls and peforated shear walls.*
> **RESPONSE: Persuasive** – discussion of AC322 data will be reorganized, discussion of values revised.
>
> Table 3.2, Item 2. ~~Similar to item 1, provision of d~~Deformation capacity at peak strength well beyond deformation capacity at design level is understood to be fundamental to earthquake performance of buildings braced with wood structural panel shear walls. ~~The source of Item 2 values is the same as Item 1.~~
>
> Table 3.2, Item 3. Testing and analysis suggests that wood structural panel sheathed shear walls ~~need to be~~ are capable of supporting post-peak loading. ~~Item 3 indicates that at~~ At a post peak capacity of 80% of peak, the shear wall element drift ~~shall not be~~ of not less than 0.028h is expected.
>
> Table 3.2, Item 4. The range of vertical element drift recognizes both the allowable story drift permitted by ASCE 7, and some variation of vertical shear wall elements above and below this drift.
>
> Table 3.2, Item 5. ~~It is intended that wood structural panel shear walls will have started softening and dissipating energy prior to reaching peak strength. In order to assure this behavior, a requirement for energy dissipation is included.~~ The criterion looks at one cycle in which the shear wall element reaches peak strength. If the CUREE protocol is used, this

would be anticipated to be in the range of the 6th to 8th loading cycle. The areas within the curve for both positive and negative excursions are intended to be summed and an equivalent viscous damping ratio calculated. See Filiatrault et al. (2003) for details of equivalent viscous damping calculation. ~~This criterion is intended to eliminate systems that exhibit near to linear elastic behavior, as these systems are not addressed in the performance expectations.~~
Reason: Items 1 through 5 are revised to reflect my understanding that the intended purpose of Items 1 through 5 was to simply state performance expectations versus identify requirements based on judgment or modeling.
RESPONSE: Persuasive for modifications to Items 3 & 5.

(L)
C3.3 Vertical Shear Wall Element Performance Expectations. Performance expectations in this section are intended to support the sheathing to framing fastening as the primary sources of inelastic behavior and source of energy dissipation in the vertical shear wall elements. Failure of the boundary members or connections addressed in items 3.3.2 to 3.3.6 could cause a more critical and possibly sudden and brittle failure of the vertical shear wall element. In general, further study is needed to determine whether or not the desired behavior requires detailing provisions beyond those currently required by ASCE 7, SDPWS, and AISI S213.

C3.3.2 Vertical boundary member tension design. Tension boundary members should be sized such that they are not the weak link in the shear wall assembly. For wood boundary members, it is recognized that most members will be stronger than the calculated nominal capacity due to the 5% basis of reference wood member strength properties in underlying design and product standards. It is also recognized that tension post capacity in use will be sensitive to placement of knots and other characteristics because the strength controlling characteristic of the wood member is not always located in the area of ~~relative to~~ maximum tension force~~ locations~~. ~~Recommendations considering these effects of wood are not available at this time.~~

C3.3.7 Boundary member and connection deformation. Excessive ~~D~~deformation of boundary members and their connections can lead to the premature failure of sheathing to framing fastening due to large imposed deformations. See discussion in commentary sections 12.2.3.11 & 12.2.3.12 of the 2003 NEHRP Provisions (FEMA, 2003).
Reason: Clarification.
RESPONSE: Persuasive.

(M)
C4 R=2.0 & 2.5 Systems
……
Adequate seismic performance of buildings designed using shear walls sheathed with other than wood structural panels (or sheet steel) is almost entirely dependent on shear wall element strength rather than ductility. As ductility is replaced with strength, reliability of the bracing system becomes highly dependant on adequate capacity, adequate detailing, and adequate understanding of seismic demand. For the materials currently included in the AISI and AF&PA standards, there is a level of comfort with design for R=2 or 2.5 systems. This comes both from a

history of design of ~~these~~ wood-frame systems using R=4.5 under the Uniform Building Code (ICBO, various) and recent analytical studies suggesting generally adequate performance with R=2 design (ATC, 2007).
RESPONSE: Non-persuasive. Also applies to CFS framed shear walls.

~~For low R-factor systems, however, there has been a history of poorly performing buildings across the range of building materials; inadequate detailing of systems has often been key to poor performance. Notable examples include major failures of precast concrete mid-rise buildings in Armenia. For light-frame buildings, case studies of poorly performing buildings following the 1994 Northridge Earthquake (CUREE, 2001b) identified a variety of sources including lack of adequate detailing of vertical shear wall elements.~~
Reason: Comparison to other building materials and reference to CUREE for this section on R=2.0 systems does not seem to add to the discussion. I thought we had previously decided to drop this paragraph.
RESPONSE: Persuasive.

Tom Skaggs (YR)
1. Even though Table 3.2A was based on a fairly large data base, I'm somewhat concerned about the implications of tested walls, which are constructed following detailing required by reference documents, not meeting these performance parameters. Although C3.2 clearly state this table is not intended for the assignment of R-values, I think that is the natural outcome of this table – i.e. either qualifying as R = 6.5 or dropping the assembly down to R = 2.0.
RESPONSE: Non-persuasive. This danger is understood, however it does not outweigh the benefit of the white paper going forward at this time.

2. Although (Figure 3.2-1) drawing is NTS, above criteria 1 and 2 are clearly not met by this drawing.
REPSONSE: Persuasive. Figure will be revised.

3. The section (3.4) relating test data using SPD loading protocol and CUREE protocol make it appear that it's as easy as applying 20% to ultimate strength. My personal opinion is that the relationships are very complicated, much more than a simple linear relationship. Based on my test experience, criteria 2, 3 and perhaps 4 would not be met with wood frame walls tested to SPD protocol. A completely different column in Table 3.2A should be developed based on SPD testing.
RESPONSE: Non-persuasive. The 20% adjustment is only intended to be applied to strength, allowing an estimate of peak demand on the shear wall. Miss-use of the criteria will need to be identified and corrected.

Steve Pryor (Y)
1. Figure #.2-1 should reflect the requirements of Table 3.2A Items 1 and 2.
RESPONSE: persuasive.

2. Section C3, second paragraph. The typical wood light-frame building responds to a seismic event by racking the wall elements while the floor and roof diaphragms remain close to elastic. Consequently, the walls largely determine the seismic response characteristics of light-frame

construction. In R = 6.5 & 7.0 wood structural panel shear wall systems, sheathing is most commonly installed in 4 foot by 8 to 10 foot sheets and fastened to wall framing with nails, screws, or similar small diameter dowel-type fasteners. ~~The primary source of seismic drift and energy dissipation of wood shear walls is the bending and yielding of the shear wall sheathing to framing fasteners around the perimeter of each sheathing panel, accompanied by slip between the sheathing and framing.~~ Seismic drift and energy dissipation in wood shear walls is primarily due to bearing deformations in the wood framing and sheathing adjacent to the fasteners around the perimeter of each sheathing panel. Bending and yielding of these fasteners contributes to a lesser extent, but both allow slip between the sheathing and framing. In CFS frame shear walls, drift and energy dissipation are generally related to the tilting (rotation) of the sheathing fasteners as well as bearing deformations in the wood structural panel or steel adjacent to the connections; again the deformations at the sheathing fasteners are accompanied by slip between the sheathing and framing.

RESPONSE: Persuasive. Suggested revision was accepted with revisions.