

SCOPE: New Part 3 Proposal for the 2009 Provisions

PROPOSAL FOR CHANGE:

Add the following new Section in Part 3:

**Light-frame Wall Systems with
Wood Structural Panel Sheathing**

Introduction

This commentary section addresses seismic force-resisting systems using light-frame walls with wood structural panel sheathing, as recognized in ASCE 7 (ASCE 2007) Table 12.2-1, Items A13 and B23.

Global Strength, Drift, and Ductility

Wood and cold formed steel (CFS) light-frame structures using wood structural panel shear walls for seismic resistance rely on a significant level of ductility and displacement capacity in addition to strength. Vertical elements of the lateral force resisting system will often have strength-level (LRFD) calculated deflections of between 0.5% and 0.75% of the story height under the design based earthquake (DBE), implying an expected real inter-story drift (δ_u) on the order of 2% to 3% of the story height for the DBE, and more for the maximum considered earthquake (MCE). Nonlinear time-history analysis, laboratory shake table testing, and observed building performance suggest this level of drift can occur, but all three have also indicated that drifts can be substantially less (Cecotti & Karacabeyli 2002, Christovasilis et al. 2007, CUREE 2001a, CUREE 2001b, CUREE 2002a, CUREE 2002b, CUREE 2002c, Pryor et al. 2000,). This wide variation in response is primarily attributable to the added strength and stiffness of finish materials on both the interior and exterior of the structure. Several shake table studies have underscored this contribution of finish material (CUREE 2001b, Christovasilis et al. 2007). Testing and analysis have also shown a tendency for drifts to be concentrated in the lowest story of multi-story light-frame buildings which are typically the softest and weakest.

Elements in the Lateral Force Resisting System

In laboratory shake table testing and observed building performance to date, the primary seismic response of light-frame wood structural panel shear wall buildings has been the in-plane racking displacement of the wall elements, while the deformation demand on floor and roof diaphragms has remained small. The few recorded instances of diaphragm failures in light-frame buildings have involved irregularly shaped diaphragms with re-entrant corners (CUREE 2001a, Christovasilis et al. 2007). Consequently, the walls are the structural elements that determine the seismic response characteristics of light-frame shear wall buildings. Note that light-frame diaphragms can have a significant influence on the seismic behavior of concrete and masonry shear wall buildings, which are beyond the scope of this commentary section.

Wood structural panel sheathed shear wall systems are intended to 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. Structural assemblies in light-frame construction are formed by a system of closely spaced repetitive wood or cold-formed steel (CFS) floor, roof and wall framing members. In wood structural panel shear wall systems, sheathing is most commonly

1 installed in 4 foot by 8 to 10 foot sheets and fastened to wall framing with nails, screws, or similar small diameter
2 dowel-type fasteners.
3

4 Seismic design in accordance with ASCE 7 anticipates cyclic loading of vertical shear wall elements in the inelastic
5 range, often to or beyond peak capacity. The primary source of drift and energy dissipation of wood shear walls is
6 the bending and yielding of the shear wall sheathing to framing fasteners around the perimeter of each sheathing
7 panel, accompanied by slip between the sheathing and framing. In most wood frame shear walls, the racking of the
8 wall causes yielding of sheathing to framing fasteners, along with local crushing of the framing and sheathing due to
9 fastener bearing. In most CFS frame shear walls, the racking of the wall causes fastener-related bearing
10 deformations, along with dimpling of the sheathing material and/or framing members. The sheathing in both wood
11 and CFS walls rotates about its center of rigidity and the maximum number of fasteners is engaged in resisting the
12 lateral loads. Wood structural panel shear walls provide the necessary ductility and dependable overstrength to
13 warrant an R-factor of 6.5 or 7, provided that the shear wall is detailed to accommodate force and deformation
14 demands; see further discussion of shear wall element demands in NEHRP Provisions Part 3 (TS 6 & 7 white
15 paper).
16

17 This section of the commentary addresses the combinations of sheathing and fastening currently included in the
18 AF&PA and AISI standards, because the cycled load behaviors of these combinations are known to provide for
19 required inelastic behavior. New combinations of sheathing and fastening should be tested in reverse-cyclic loading;
20 some combinations of sheathing materials and methods of attachment (such as adhesive) have not shown similar
21 ductility levels in laboratory tests. This section of the commentary is also based on the behavior of shear walls with
22 overturning restraint (tie-downs or similar) provided at each end, as is common in an engineered structure;
23 laboratory testing of shear walls without overturning restraint has shown increases in deflection and reductions in
24 strength (CUREE 2004, Dolan & Heine 1997a, Dolan & Heine 1997b).
25
26

27 **Effects of Wall Finish Materials**

28
29 The affect of wall finish materials (gypsum wallboard, exterior stucco or siding, etc) on seismic performance
30 varies with the seismic demand level. Shake table testing of a full scale townhouse building as part of the
31 NEESWood benchmark (Christovasilis, 2007) test phases showed a significant influence of finish materials at a
32 ground motion level of roughly 50% of the DBE, with resulting average inter-story drift of approximately 0.75%. It
33 is anticipated that the influence of finish materials will be somewhat less for the full DBE with anticipated inter-
34 story drifts on the order of 3 to 4%. For structures near collapse, estimated at approximately 7% inter-story drift
35 based on structures tested on Japan's E-Defense table, it is thought that there is little contribution of finish materials
36 left in the structure. None the less, the inclusion of finish materials in analytical studies for ATC 63 (ATC, 2007)
37 did improve the collapse margin to an acceptable level, so influence of wall finish materials on collapse behavior
38 cannot be dismissed. More study of the influence of finish materials is needed.
39

40 Testing and analysis have demonstrated that finish materials can affect inter-story drift and performance. The
41 NEESWood benchmark testing showed that the reduction of drift due to finish materials was greatest for gypsum
42 wallboard applied to walls detailed as shear walls i.e. there was positive force transfer between the wall and the
43 diaphragm and overturning restraint was provided. When additional gypsum wallboard was applied to
44 nonstructural partition walls not detailed for force transfer from the diaphragms, the contribution was minimal.
45 Several studies have also shown that stucco can be very effective in reducing inter-story drift demand, while the
46 NEESWood benchmark test showed only a modest effect from the stucco exterior finish.
47

48 Further, when considering stucco as part of the seismic-force resisting system there are several limiting issues.
49 There is concern over the long-term performance of the stucco since stucco in combination with building
50 paper/barriers is the weather barrier for the structure. Staples are often used to attach the lath to the studs, but staple
51 legs do not have much thickness to resist long-term corrosion. Galvanized or stainless steel staples would be more
52 resistant to corrosion.
53

54 Finish materials such as gypsum wallboard and stucco have much lower displacement at peak capacity (around
55 0.5% to 1% of the story height) than the more ductile wood structural panel elements (1.5 to 5% of the story height).

1 This prevents a simple summation of capacities and complicates estimated performance. Although not directly
2 considered in the design, it seems clear that when a building is responding with inter-story drifts on the order of 2 to
3 3%, well beyond the peak capacity of the gypsum or stucco, those elements can provide important hysteretic
4 damping to help attenuate the movement of the structure. Similarly, CUREE shake table tests of a two-story wood
5 frame house examined the effects of waste wall sheathing (extra sheathing used in a segmented shear wall design
6 above and below the openings). The shake table tests of segmented walls without the horizontal segments had
7 increased wall displacements by a factor more than two, suggesting that the sheathing above the below the openings
8 has a significant effect on the performance of the structure (CUREE, 2001b) .
9

10 While not directly addressed in the Code, the affect of finishes should be evaluated qualitatively. In contrast to
11 moment frame construction, the mode shape of traditional light-framed multi-story buildings tends to degrade to a
12 single degree of freedom system as a story reaches its ultimate strength and deforms predominately in shear. As a
13 result, the majority of the displacement demand of the structure is concentrated in the weakest story. The other
14 stories tend to remain in the elastic range. Finishes, which tend to be more concentrated in upper stories, contribute
15 to this behavior, and tend to cause significant displacement demand in the first story. Detailing of shear wall
16 boundary members and anchorage connections is vital to adequate seismic performance in the soft or weak story of
17 a structure

18 **Analytical Modeling**

19 In order to evaluate the distribution of design forces and drifts, a building analytical model in accordance with
20 ASCE 7, Section 12.7.3 is needed. Generally, the analysis uses either the simplified alternative structural design
21 criteria (ASCE 7, Section 12.14) or equivalent lateral force procedure (ASCE 7, Section 12.8). The primary analysis
22 model includes only those elements designated as structural shear walls and diaphragms.
23
24

25 Vertical distribution of forces should be in accordance with ASCE 7, which dictates a triangular (first mode)
26 distribution for buildings under the equivalent lateral force procedure and a rectangular distribution under the
27 simplified alternative structural design criteria, with a slight increase in design base shear.
28

29 Analytical modeling of the horizontal distribution of seismic forces at a theoretical level is based on the relative
30 stiffness of the diaphragms, shear walls, and any other force resisting elements. Horizontal distribution should be in
31 accordance with ASCE 7, which identifies three categories: diaphragms which are clearly very flexible relative to
32 the vertical elements; diaphragms that are clearly very rigid relative to the vertical elements; and everything else.
33 For everything else, it is required that a semi-rigid diaphragm model be used for distribution of horizontal seismic
34 forces.
35
36

37 ASCE 7 includes an exception permitting one- and two-family residential buildings with diaphragms of wood
38 structural panel or untopped steel decking to be categorized as flexible. The simplified alternative structural design
39 criteria of ASCE 7, Section 12.14 also permits assumption of a flexible diaphragm force distribution for wood
40 structural panel diaphragms, untopped steel decking, or similar panelized construction (Section 12.4.5), with a small
41 increase in base shear for multi-story buildings.
42

43 Other buildings braced by wood and CFS light-frame shear walls fall into the “everything else” category. For these
44 other buildings, one approach to meeting this requirement is evaluating force distribution using both rigid and
45 flexible diaphragm models and designing each shear wall and diaphragm for the worst case force. For this approach,
46 the flexible diaphragm model should be solved first, followed by an iterative rigid diaphragm distribution analysis.
47

48 Where non-uniform distribution of finish materials might trigger significant torsional behavior in the structure,
49 additional analytical study, including the effects of finish materials should be considered.
50

51 **Adhesive Attachment of Sheathing in Wood & CFS Light-frame Buildings**

52 Ductility in wood-frame shear walls with wood structural panel sheathing is provided by the fasteners used to attach
53 the sheathing to the framing members. Testing has shown brittle failure when adhesives are used for this
54
55

1 attachment in place of mechanical fasteners. This is discussed further in the “Recommended Lateral Force
2 Requirements and Commentary” (SEAOC, 1999), Section C804.3. For this reason, the 2006 edition of the IBC
3 Section 2305.3.10 states "Adhesive attachment of shear wall sheathing is not permitted as a substitute for
4 mechanical fasteners, and shall not be used in shear wall strength calculations alone, or in combination with
5 mechanical fasteners in Seismic Design Category D, E or F." The 2005 edition of AF&PA Special Design
6 Provisions for Wind and Seismic (SDPWS) Section 4.3.6.3.1 states, "Adhesive attachment of shear wall sheathing
7 shall not be used alone, or in combination with mechanical fasteners." Both of these provisions address adhesive
8 attachment in wood-frame shear walls.

9
10 To date, adhesives are not known to have been used for structural attachment of wood structural panel sheathing to
11 CFS stud walls. There is no discussion of adhesive attachment of CFS frame shear walls as of yet in either the IBC
12 or AISI S213 (2007); however, a commentary note was added to AISI S213. Limited testing to date (Serrette, 2006)
13 suggests some similarities to the brittle behavior seen in wood-frame shear walls with adhesives. For this reason,
14 adhesive attachment to CFS studs should not be undertaken without adequate study of seismic behavior.

15 16 17 **Sheathing Fasteners for Wood & CFS Light-frame Buildings**

18
19 *Over-Driven Fasteners.* Fasteners that are driven past the top of the sheathing have reduced bearing area on the
20 sheathing. Consequently, this can lead to a shear load reduction of sheathed assemblies. APA Technical Note #TT-
21 012 states that no reduction is required for wood sheathed shear walls or diaphragms when the fasteners are
22 overdriven up to 1/16" under dry conditions. In addition, no reduction is necessary if no more than 20% of the
23 perimeter fasteners are driven between 1/16" and up to 1/8". Also, an article entitled “Capacity of Oriented Strand
24 Board Shear Walls with Overdriven Sheathing Nails” reports on a testing program of overdriven OSB sheathing
25 fasteners (Jones and Fonseca, 2002). The Canadian code CSA (CSA, 2001) O86 Cl. 9.5.3.4 states, “Nails shall be
26 firmly driven into framing members but shall not be over-driven into sheathing. For structural wood-based
27 sheathing, nails shall not be over-driven more than 15% of the panel thickness.”

28
29 Significant reductions in strength have been observed in laboratory tests of CFS frame shear wall assemblies when
30 the screw sheathing fasteners are overdriven. This may be due to the countersunk screw head removing or
31 damaging more of the wood sheathing than an overdriven common sheathing nail. Additional testing is required to
32 determine what affect overdriven screw fasteners may have on shear strength for CFS frame shear wall assemblies.

33
34 *Fastener Locations.* The location of the sheathing fasteners affects the performance of shear wall assemblies. If the
35 sheathing fasteners are located too close to the panel edge, they may pull through the sheathing panel prior to the
36 shear wall assembly obtaining its expected capacity. In addition, if the fasteners penetrate into the framing member
37 too close to the framing member’s edge or if the fasteners are spaced too close together, they may cause the wood
38 framing member to split prior to the assembly obtaining the expected load.

39
40 *Wood Framing Considerations:* The AF&PA NDS Commentary (AF&PA, 2005b) includes spacing and edge
41 distance recommendations for fasteners less than 1/4" diameter used for wood framing. In addition, the IBC Section
42 2305.1.2.1 (ICC 2006) requires that fasteners not be placed less than 3/8" from the panel edge. The IBC and
43 AF&PA SDPWS wood frame shear wall table require a minimum of a 3" nominal framing member at panel edges
44 and staggered placement of fasteners when fasteners are spaced at 2 1/2" on center or closer or for 3" on center
45 spacing when 10d commons penetrate in the framing members more than 1 1/2".

46
47 *CFS Framing Considerations:* AISI S100 (AISI, 2007b) includes spacing and edge distance requirements for
48 fasteners used for steel-to-steel connections. AISI S213, Section C2.2 requires that fasteners be 3/8" from the panel
49 edge, for all of the sheathing panels shown in the standard, as well. The 2007 edition of AISI S213 will incorporate
50 a Canadian provision requiring minimum of 1/2" edge distance for wood sheathed, CFS frame shear wall assemblies
51 based on edge distance used in the testing.

52
53 Salenikovich (2000) tested walls with different edge distances for the sheathing fasteners and found that the
54 strength/capacity does not increase very much with increased edge distance, but the displacement capacity
55 significantly increased and the building as a whole would be toughened. The edge distance of the fasters along the

1 bottom of the wall (between the sheathing and the bottom plate of the wall framing) is especially important to
2 achieve the toughening effect.
3
4

5 **Hold-Down Connector Slip for Wood & CFS Light-frame Buildings**
6

7 *Performance.* The aspect ratio of the shear wall affects how much horizontal wall drift is due to vertical deflection
8 of the hold-down connection. Higher aspect ratio shear wall assemblies are more sensitive to hold-down connection
9 slip or deformation. This slip/deformation related drift is addressed in one part of the light-frame shear wall
10 deflection equation that determines the horizontal wall drift due to the vertical deflection of the hold-down
11 connection. Hold-down deflection may be comprised of fastener slip, device elongation/movement, and/or anchor
12 bolt elongation.
13
14

15 **Combination of Materials for Wood & CFS Light-frame Buildings**
16

17 Combining shear walls that have different sheathing materials is not permitted by either the IBC or ASCE 7. It is
18 permitted to have one side opposite the wood or steel sheet sheathed side to be sheathed with gypsum board and
19 there are load values for that combination of sheathing materials on opposite side of the same shear wall assembly.
20 This prohibition is due to the difference in stiffness, strength and performance of shear walls with different
21 sheathing materials that, when combined, may result in unexpected load distribution and failure. However, if the
22 gypsum is to be used to resist the lateral forces in the design, the walls included in the design must have the gypsum
23 attached to the top and bottom plates of the wall framing. This is not the typical attachment used by most drywall
24 installers. The concept of floating corners has been recommended by most drywall manufacturers to prevent
25 cracking of the taped joint in the drywall at the top of the wall to ceiling. If floating corners is used, the gypsum is
26 totally ineffective in resisting any lateral loads unless the wall deformation causes the gypsum panel to bear on an
27 adjacent structural elements such as in a corner.
28
29

30 **Aspect Ratio for Wood & CFS Light-frame Buildings**
31

32 Shear wall aspect ratio also plays a role in the performance of light-frame shear wall assemblies. Although similar
33 strength may be observed for shear walls of different aspect ratios, with ratios over 2:1 flexural bending of the wall
34 becomes more dominant than shear/fastener deformation. Testing showed that these high aspect ratio walls are not
35 only more flexible, but that they will not satisfy the required seismic drift limit performance objective without a
36 reduction in design capacity and corresponding design level drift. Therefore, a reduction factor, $2w/h$, was
37 implemented in the AF&PA standard for those shear wall assemblies exceeding 2:1 but not exceeding 3.5:1. The
38 same reduction is applied in the AISI standard for shear wall assemblies with aspect ratios not exceeding 4:1. Also,
39 it should be noted that as walls get narrower, it is crucial to ensure proper installation of the shear wall components
40 (such as hold-down connections), since poor installation will lower the performance significantly more (i.e.:
41 excessive top of wall drift) than would be seen by a lower aspect ratio wall.
42
43

44 **Wood Moisture Issues**
45

46 *Changes in Moisture Content.* Significant changes in moisture content (>8%) can effect physical wood properties
47 by swelling or shrinking wood fibers which in turn can effect wood connections. Testing by APA shows that wood
48 framed shear walls do not lose capacity due to moisture changes, however, nail slip is increased and thus shear wall
49 deflections increase.(APA, 2002). To minimize any negative effects of increased shear wall deflections, the designer
50 should specify dry lumber or engineered wood products (which are typically dry) for highly optimized structures.
51
52

1 **Wood Framing Forces and Connectivity**

2
3 *Tension Failures of Wood Posts.* It is unusual for tension posts in walls to fail in tension if the only tension force is
4 the induced force associated with the racking and overturning of the wall segment itself. However, if the load path
5 from upper stories causes significant additional tension forces in the post, failure can occur through three possible
6 mechanisms: 1) net section failure, 2) combined bending and tension failure, and 3) connection failure. Tension
7 posts should be designed with checks for all three failure mechanisms.

8
9 *Compression Failure of Wood Posts.* Compression failures in wood posts are typically associated with high
10 compression load due to accumulation of load due to load path considerations. The possible failure mechanisms
11 are: 1) traditional buckling and 2) beam column action.

12
13 Buckling of the post is usually restricted to the out-of-plane direction of the wall because of the continuous support
14 provided by the sheathing in the plane of the wall. The designer must account for the entire load being supplied to
15 the compression post, including the compression induced by the racking and overturning of the wall element itself
16 plus the compression forces being transferred to the post by the stories above and any headers attached to the post.
17 Buckling of isolated, stand-alone posts must be checked in both directions.

18
19 *Shear Failure for Wood Studs* Shear failures in studs have been observed in wall tests and field studies where strong
20 sheathing materials, such as stucco, fail at connections along the bottom of the wall. This causes the next row of
21 fasteners from the bottom of the wall to transfer all of the lateral loads into the studs, thus loading the studs in
22 bending about their weak axis. Typically, the failure occurs at about 1/3 the height of the stud.

23
24 *Wood Cross-Grain Bending.* Cross-grain bending of wood members leads to failure at relatively low loads. Of
25 particular concern are locations where ledger boards are loaded perpendicular to their length and at shear wall
26 foundation sill plates. For ledgers, the connection between the diaphragm and the masonry or concrete wall should
27 connect the longitudinal diaphragm framing member (or a series of blocked joists) directly to the wall rather than to
28 the ledger. In shear walls, wide steel plate washers and stiff hold-down connectors or straps should be used to
29 reduce the cross-grain bending action.

30
31 *Bearing failure of wood plate.* The failure of the bottom plate of the wall due to compression perpendicular to grain
32 causes damage to the wood member holding sheathing fasteners along the bottom of the wall. Added deformation of
33 walls with wood framing loaded perpendicular to grain at wood floor levels compared to walls located on slabs-on-
34 grade has been observed in testing and should be accounted for in analysis of drift.

35
36
37 **Cold Formed Steel Member Considerations**

38
39 *Local Damage of Framing.* The thin-walled nature, i.e. small thickness, of CFS framing members makes them
40 vulnerable to physical damage that may have an adverse effect on the structural performance. (Ref. AISI S200-07).
41 Full damage assessment is not within the scope of the AISI standards and, consequently, when damage alters the
42 cross-section geometry of a framing member beyond the specified tolerances, the designer should be consulted.
43 (AISI, 2007c)

44
45 *Web Holes.* As with any material, large holes may affect the structural performance of CFS framing members.
46 Therefore, CFS framing standards require that “*Holes in webs of studs, joists and tracks shall be in conformance*
47 *with an approved design, AISI S100, or an approved design standard. Webs with holes not conforming to the above*
48 *shall be reinforced or patched in accordance with an approved design or approved design standard.*” (Ref. AISI
49 S200-07)

50
51 In CFS framing members, a “punch-out” is defined as a hole made during the manufacturing process in the web of a
52 steel framing member (Ref. AISI S200-07). Suitable dimensions and locations of standard punch-outs are further
53 defined. (Ref. AISI S201-07) (AISI, 2007d).

54
55 Nominal shear strengths for shear walls that have been included in industry design standards are based on tests with

1 studs with 1.5-inch (38 mm) x 4-inch (100 mm) punch-outs at a center-to-center spacing of 24 inches (600 mm)
2 and, as a result, the use of studs with standard punch-outs is permitted when using these values (Ref. AISI S213-
3 07).

4
5 *Local Buckling of CFS Framing Members.* CFS framing members are susceptible to local buckling when loaded in
6 compression. Consequently, local buckling is a necessary design consideration for CFS members. To that end, CFS
7 framing standards specify that “*The proportioning, design and detailing of cold-formed steel light-frame systems,*
8 *members, connections and connectors ... be in accordance with AISI S100.*” (Re. AISI S213-07)

9
10 AISI S100-07 considers local buckling of individual elements of CFS members as a major design criterion. As such,
11 AISI S100-07 requires that the design of such members provide sufficient safety against the failure mode, with due
12 consideration to postbuckling strength. Postbuckling strength of plate elements was discovered experimentally in
13 1928. In order to utilize the postbuckling strength for design purposes, AISI has used the effective design width
14 approach to determine the section properties since 1946.

15
16 The designer should pay particular attention to the buckling performance for collectors and headers that are acting
17 as drag struts. These members can have relatively high compression forces and must be designed for cyclic loading.
18 The local transfer of forces into wall or diaphragm elements also becomes an area where localized buckling can
19 occur.

20
21 *Bending Failure of CFS Framing Members.* The difference in local buckling behavior between stiffened and
22 unstiffened elements results in a significant difference in the weak axis bending strength between stud and track
23 members. Weak axis bending strength of a track member is typically lower than a stud. This makes proper design
24 and detailing of wall anchorage important in order to minimize bending loads on track members. Connection of the
25 anchorage device directly to the chord stud is a common detail, which effectively eliminates bending loads on track
26 members.

27
28 *Bending Deformation of CFS Track Web.* CFS framing members offer limited strength and stiffness to resist
29 transverse concentrated loads, such as would be applied by an anchor bolt with standard cut washer through the
30 track web if used to resist wall uplift forces. This is similar to the concern regarding cross grain bending in a wood
31 bottom plate with a similar anchorage detail.

32
33 Consequently, the AISI CFS framing standards require that “*Studs or other vertical boundary members at the ends*
34 *of wall segments, that resist seismic loads, braced with either sheathing or diagonal braces, ... be anchored such*
35 *that the bottom track is not required to resist uplift by bending of the track web.*” (Ref. AISI S213-07)

36
37 Connection of the anchorage device directly to the chord stud is the most common detail, which effectively
38 eliminates transverse concentrated loads on track members. Use of a C-shape section (i.e., stud) reinforcement or a
39 plate washer at anchor bolts are other ways to distribute transverse concentrated loads and may avoid this issue.

40
41 *CFS Sheathing Connectors.* As with wood frame shear walls, in CFS shear walls inelastic behavior is primarily
42 concentrated at the fasteners connecting the sheathing to the wall-framing members. However, the behavior of
43 sheathing screw fasteners in CFS framing members is different than the behavior of sheathing nails in wood
44 framing.

45
46 “*In general, racking of the wall resulted in the screw fasteners rocking (tilting) about the plane of the stud flange*
47 *material immediately around the screw. This behavior resulted in permanent lateral deflection of the wall and*
48 *appears to be the main source of energy dissipation in the walls. As the lateral displacement of the wall increased,*
49 *the panel pulled over the screw heads and became unzipped.*” (Ref. Nguyen, Hall and Serrette 1996)

50
51 Screw size must be sufficient to preclude the screw shear failure mode. Thus, the screw size needs to be properly
52 matched with framing member thickness to assure the desired behavior. The AISI CFS framing standards require
53 that “*Unless noted as (min.), substitution of a stud or track of a different designation thickness is not permitted.*”
54 (Ref. AISI S213-07)

1 *End Gap in CFS Stud-to-Track Connection.* In wood frame shear walls, “all wood columns and posts shall be
2 framed to provide full end bearing. Alternatively, column and post end connections shall be designed to resist the
3 full compressive loads, neglecting all end bearing capacity.” (Ref. ASCE 7-05 Section 14.5.2)

4
5 In CFS framing, “Ends of structural wall studs shall have square end cuts and shall be seated tight against the
6 tracks. For the purpose of this section, seated tight shall mean that a maximum gap tolerance of 1/8 inch (3.2 mm)
7 will be acceptable between the end of wall framing member and the track.” (Ref. AISI S200-07) The CFS framing
8 end gap does not adversely affect strength, but it is likely to contribute to system flexibility and shear wall
9 deflection.

10
11
12 Testing has shown that a smaller gap tolerance may be suitable in some situations. For instance, testing of thicker
13 materials (greater than 0.054 inches) “showed that the relative movement between the stud and track could result in
14 shear failure of the screws.” In these cases, a smaller gap tolerance of 1/16 inch (1.6 mm) may be more appropriate.
15 Additionally, “a smaller gap tolerance may also be desirable in multi-story structures where the accumulation of
16 these gap closures may become significant. Depending on track radius, it may be necessary to oversize the depth of
17 the track to assure that the stud flanges do not prematurely engage the track radius and result in an excessive gap.”
18 (Ref. AISI S200-07)

19
20 The designer should take the above into consideration when calculating shear wall deflections, by considering the
21 effect of the end gap in the fourth term of the deflection equation (i.e., lateral contribution from anchorage/hold-
22 down deformation). (Ref. AISI S213-07)

23
24 *Pull-Out Resistance of CFS Screw Fasteners.* The pull-out resistance of screw fasteners may be reduced when the
25 fasteners are cyclically loaded (Mahendran and Maharaachchi, 2000). Consequently, CFS framing standards
26 require that “The pull-out resistance of screws ... not be used to resist seismic forces.” (AISI S213-07)

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