TECHNICAL NOTE $5.00
On Cold-Formed Steel Construction
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Changes from the 1997 UBC to the 2006 IBC for Lateral Design with Cold-Formed Steel Framing

Summary: The intent of this document is to highlight the changes to the cold-formed steel framing lateral design provisions in the 2006 International Building Code, which has adopted the AISI 2004 Standard for Cold-Formed Steel Framing - Lateral Design, in comparison to the cold-formed steel framing lateral provisions in the 1997 Uniform Building Code.

INTRODUCTION

The adoption of the 2006 International Building Code (IBC, Ref. 1) has brought about many changes in structural engineering design, perhaps most notably in California and Hawaii, which have been designing per the 1997 Uniform Building Code (UBC, Ref. 2) for the past ten years. In terms of cold-formed steel (CFS) design specifications, the 1997 UBC Chapter 22 adopted the 1986 version of the American Iron and Steel Institute (AISI) *Specification for Design of Cold-Formed Steel Structural Members* with the December 1989 Addendum (referred to as the AISI-ASD). Division VII of Chapter 22 covers amendments to AISI-ASD while Division VIII covers lateral resistance for steel stud wall systems.

The 2006 IBC, on the other hand, references the AISI 2001 *North American Specification for the Design of Cold-Formed Steel Structural Members* (NAS, Ref. 3), including the 2004 Supplement as well as separate AISI cold-formed steel framing standards covering general provisions, header design, lateral design, the prescriptive method for one- and two-family dwellings, truss design, and wall stud design. For lateral design, the AISI 2004 Standard for Cold-Formed Steel Framing - Lateral Design (AISI-Lateral, Ref. 4) incorporates several important design changes that engineers should be familiar with. This technical note covers clarification of the design method when R is equal to or less than 3, the safety factor for shearwalls used for wind resistance, additional framing member thicknesses for shear walls resisting seismic loads, shear walls sheathed with sheet steel, the deflection equation for Type I shear walls, higher permitted aspect ratio for shear walls, the design of Type II shear walls, the diaphragm table and design provisions, and the diaphragm deflection equation.

CLARIFICATION OF THE DESIGN METHOD WHEN R IS EQUAL TO OR LESS THAN 3

When the seismic response modification factor, R, for steel systems is taken as greater than 3, AISI-Lateral requires in Section C1.1, that the design must follow the provisions stated in Sections C5 (Walls) and D3 (Diaphragms). This is consistent with the requirements of ASCE 7-05 (Ref. 5) Design Coefficients and Factors for Seismic Force-Resisting Systems Table 12.2-1, which is adopted by reference by the 2006 IBC. Figures 1A and 1B show Table C1-1 of AISI-Lateral and AISI S213-07 which summarize ASCE7-02 Table 9.5.2.2 and ASCE7-05 Table 12.2-1, respectively, as they pertain to cold-formed steel shear walls and strap braced walls.

As is evident from the Table C1-1, designing with an R factor of less than 3 is not permitted for Seismic Design Categories E and F and is subject to height limitations for Seismic Design Category D. The ASCE7-02 and 7-05 Design Coefficients and Factors for Seismic Force-Resisting Systems tables both assign an R value of 3 for Structural Steel Systems Not Specifically Detailed for Seismic Resistance and this is permitted for designs in SDC A through C. The special seismic provisions of Sections C5 and D3 are not required to be followed when R values of 3 or less are used.

It should be noted that ASCE 7-05, adopted by reference by the 2006 IBC, assigns an R value of 6 ½ for light framed walls for bearing wall systems with wood or steel sheet sheathing, whereas ASCE7-02 Table 9.5.2.2 (Figure 1A) assigns an R value of 6, which would be required for designs under the 2000 and 2003 IBC.

In addition, footnote g to the ASCE7 Design Coefficients and Factors for Seismic Force-Resisting Systems tables permits the designer to subtract 0.5 from the tabulated overstrength factor if the diaphragm is considered flexible (i.e.; 3.0 - 0.5 = 2.5), but the value may not be reduced below 2. This serves to reduce the force that the connections, boundary elements, and overturning restraint devices (hold-downs) must be

Table C1-1
Design Coefficients and Factors for Basic Seismic Force-Resisting Systems

Basic Seismic Force-Resisting System	Seismic Response Modification Coefficient, R	System Over-strength Factor, Ω_o	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (ft) Limitations [a] — Seismic Design Category: A&B	C	D	E	F
Bearing Wall Systems								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6	3	4	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2	2 ½	2	NL	NL	35	NP	NP
Light-framed wall systems using flat strap bracing	4	2	3 ½	NL	NL	65	65	65
Building Frame Systems								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6 ½	2 ½	4 ½	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2 ½	2 ½	2 ½	NL	NL	35	NP	NP

[a] NL = Not Limited and NP = Not Permitted.
For SI: 1 foot = 0.305 m

FIGURE 1A: AISI-LATERAL TABLE C1-1 DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS (ASCE7-02)

Table C1-1
United States and Mexico
Design Coefficients and Factors for Basic Seismic Force-Resisting Systems

Basic Seismic Force-Resisting System	Seismic Response Modification Coefficient, R	System Over-strength Factor, Ω_o	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (ft) Limitations [a] — Seismic Design Category: A&B	C	D	E	F
Bearing Wall Systems								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	6 ½	3	4	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2	2 ½	2	NL	NL	35	NP	NP
Light-framed wall systems using flat strap bracing	4	2	3 ½	NL	NL	65	65	65
Building Frame Systems								
Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	7	2 ½	4 ½	NL	NL	65	65	65
Light-framed walls with shear panels of all other materials	2 ½	2 ½	2 ½	NL	NL	35	NP	NP

[a] NL = Not Limited and NP = Not Permitted.
For SI: 1 foot = 0.305 m

FIGURE 1B: AISI S213-07 TABLE C1-1 DESIGN COEFFICIENTS AND FACTORS FOR BASIC SEISMIC FORCE-RESISTING SYSTEMS (ASCE7-05)

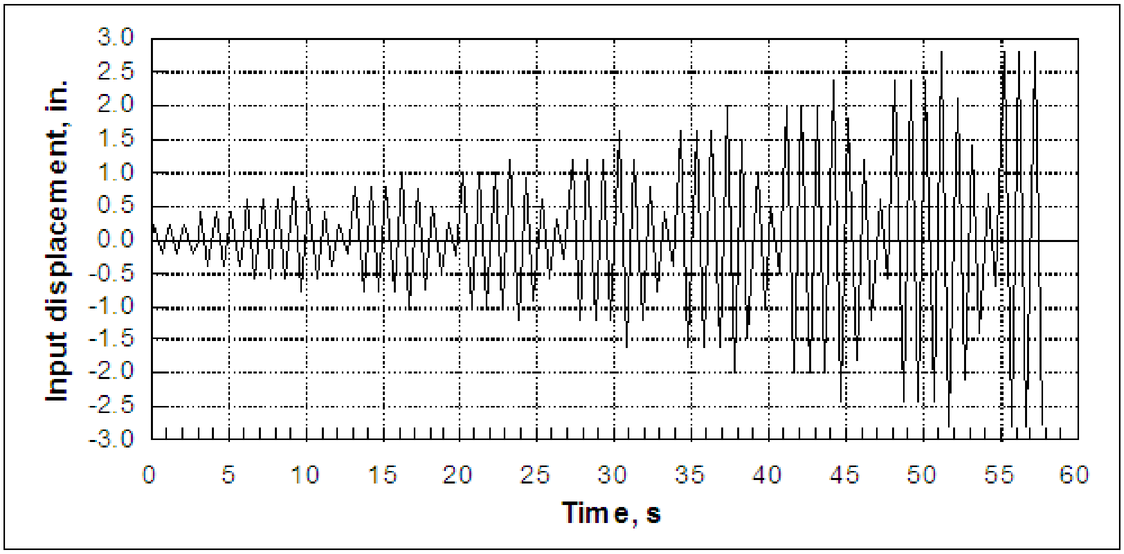

FIGURE 2: AISI-LATERAL FIGURE C2-2 - REVERSE CYCLIC TEST PROTOCOL (1.0 HZ)

designed to resist in accordance with Section C5 when using an R value greater than 3.

SAFETY FACTOR FOR SHEAR WALLS USED FOR WIND RESISTANCE

The safety factor required for the allowable strength design of shear walls for wind resistance has been revised from 3.0 (from Section 2219.3 of the 1997 UBC) to 2.0 (Section C2.1 of the AISI-Lateral). The wind load table values are based upon monotonic tests. Typically, the available strengths of shearwall assemblies used to resist wind loads are determined through monotonic tests per ASTM E564, Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings (Ref. 6). The seismic load table values are based upon the Sequential Phase Displacement protocol (SPD, Ref. 7) reversed cyclic test protocol, shown in Figure 2, and degraded wall strength envelope responses, shown in Figure 3, in both the 1997 UBC and the AISI-Lateral shear wall tables.

It should be noted that wood sheathed, CFS framed shear wall assemblies have been observed to have up to 20% more strength when tested using the CUREE cyclic test protocol (Ref. 8) compared to the SPD cyclic test protocol (Ref. 9). Also, the degraded wall strength envelope may result in up to a 10% decrease in load when compared to the peak load envelope. Therefore, designers should multiply the seismic shear wall table load values for wood sheathed, CFS framed assemblies by a factor of 1.3 when determining the "loads that the system can deliver" as noted in Section C5 of AISI-Lateral.

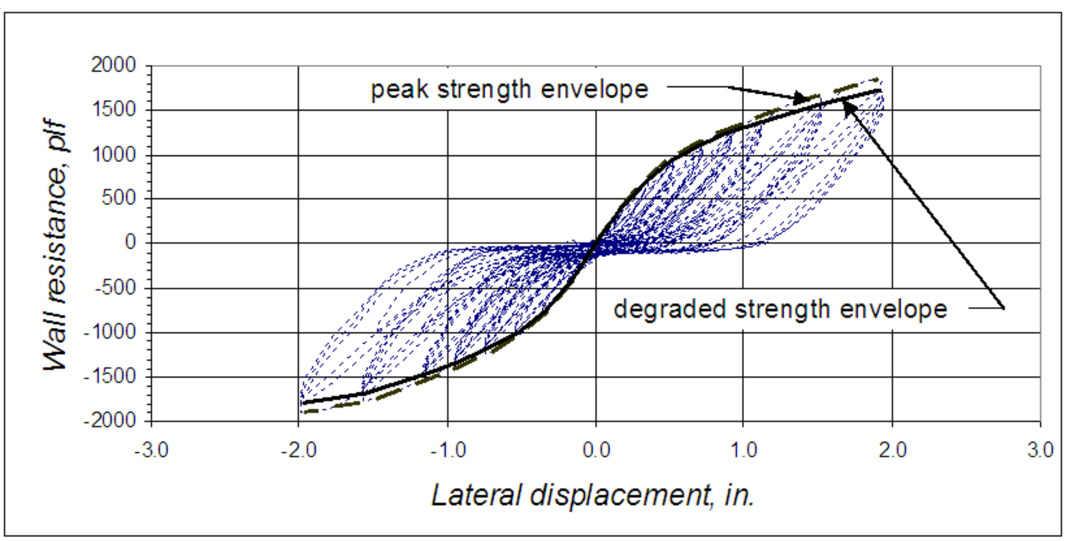

FIGURE 3: AISI-LATERAL FIGURE C2-3 - HYSTERETIC RESPONSE PLOT SHOWING PEAK AND DEGRADED STRENGTH ENVELOPES

ADDITIONAL FRAMING MEMBER THICKNESSES FOR SHEAR WALLS RESISTING SEISMIC LOADS

The 1997 UBC restricted the member thicknesses for wall studs and tracks that could be used for framing shear walls resisting seismic loads. Table 22-VIII-C of the 1997 UBC states in footnote 2 that studs and track shall have a minimum uncoated base metal thickness of 0.033 inch and shall not have a base metal thickness greater than 0.043 inch (20ga. and 18 ga., respectively).

AISI-Lateral, however, expands the allowable framing member thicknesses to thicker than 43 mil (18 ga.), provided that a higher grade steel is utilized. Footnote 6 of Table C2.1-3 of AISI-Lateral, which details the nominal shear strengths for seismic loads for shear walls, states that walls studs and track shall be of ASTM A1003 Grade 33 Type H steel for members with a designation thickness of 33 and 43 mil, and A1003 Grade 50 Type H steel for members with a designation thickness equal to or greater than 54 mils.

However, AISI-Lateral also states that unless the seismic shear wall table framing indicates a minimum framing thickness (only indicated for sheet steel sheathed assemblies), use of a different framing thickness than what's shown in the seismic shear wall table is not permitted. This provision is to try to preclude shear failure of the screw fasteners to help ensure ductile performance of the shear wall assembly.

SHEAR WALLS SHEATHED WITH SHEET STEEL

The 1997 UBC provided nominal shear values for shear walls framed with cold-formed steel studs and faced with ½" gypsum wallboard each side, 15/32" Structural I plywood sheathing one side, or 7/16" OSB one side. AISI-Lateral introduces nominal shear strength values for shear walls resisting seismic loads for two thicknesses of steel sheet sheathing, 0.018" and 0.027". All the UBC and AISI-Lateral shear wall table values are based upon tests and analysis conducted by Serrette (Ref. 10,11,12)

Sheet steel sheathing values have also been added to the nominal shear strength for wind load table as well in Table C2.1-1. In addition, AISI-Lateral permits an increase in available strength when a panel is used on both sides of the wall. However, AISI-Lateral Section C2.1 states that the available strength is not cumulative if different sheathing material or fastener spacing is used on the same side of the wall, that one is to use twice the strength of the weaker sheathing material or only the strength of the stronger sheathing material if different sheathing is used on either side of the wall, and that the strength is not cumulative for dissimilar sheathing material applied to the same wall line. Table C2.1-1, Nominal Shear Strength for Wind Loads, does permit a 30% strength increase when the opposite side of the tabulated assembly is sheathed with gypsum board with screw spacing at 7" on center at the edges and field, but this is only for a couple wood sheathed assemblies.

DEFLECTION EQUATION FOR TYPE I WOOD AND SHEET STEEL SHEATHED, CFS FRAMED SHEAR WALLS

A challenge with the 1997 UBC is the calculation of shear wall deflections. Without any specific method provided, the engineer is forced to estimate these values, which may affect the final design solution and lead to potentially significant costs or in-service loading issues. The 2006 IBC, through AISI-Lateral, provides a means to estimate deflection for wood and steel sheet sheathed Type I CFS framed shear walls based on a simple mechanical model empirically corrected to match shear wall tests of CFS framed shear wall assemblies.

This new deflection equation in AISI-Lateral Section C2.1.1, shown below, is the deflection equation for blocked wood and steel sheet sheathed Type I shear walls, based on work performed by Serette (Ref. 13). Equations C2.1-1 and C2.1-2 (SI) can be used to calculate the approximate deflection of CFS framed shear walls to determine if they comply with the seismic story drift limitations of ASCE 7-05 Section 12.12.

$$\delta = \frac{8vh^3}{E_s A_c b} + \omega_1 \omega_2 \frac{vh}{\rho G t_{sheathing}} + \omega_1^{5/4} \omega_2 \omega_3 \omega_4 \left(\frac{v}{\beta}\right)^2 + \delta_a$$

(Eq. C2.1-1)

where:

A_C = Gross cross-sectional area of *chord* member, in square inches (mm²)
b = Width of the *shear wall*, in feet (mm)
E_s = Modulus of elasticity of steel = 29,500,000 psi (203,000 MPa)
G = Shear modulus of sheathing material, in pounds per square inch (MPa)
h = Wall height, in feet (mm)
s = Maximum fastener spacing at panel edges, in inches (mm)
$t_{sheathing}$ = Nominal panel thickness, in inches (mm)
t_{stud} = Framing designation thickness, in inches (mm)
v = Shear demand (V/b), in pounds per linear foot (N/mm)
V = Total lateral load applied to the *shear wall*, in pounds (N)
β = 810 for plywood and 660 for OSB
= 500 ($t_{sheathing}$/0.018) for sheet steel (for $t_{sheathing}$ in inches)
= 500 ($t_{sheathing}$/0.457) for sheet steel (for $t_{sheathing}$ in mm)
δ = Calculated deflection, in inches (mm)
δ_a = Deflection due to anchorage/attachment details, in inches (mm)
ρ = 1.85 for plywood and 1.05 for OSB
= 0.075($t_{sheathing}$/0.018) for sheet steel (for $t_{sheathing}$ in inches)
= 0.075($t_{sheathing}$/0.457) for sheet steel (for $t_{sheathing}$ in mm)
ω_1 = s/6 (for s in inches) and s/152.4 (for s in mm)
ω_2 = 0.033/t_{stud} (for t_{stud} in inches) and 0.838/t_{stud} (for t_{stud} in mm)

$$\omega_3 = \sqrt{\frac{\left(h/b\right)}{2}}$$

ω_4 = 1 for wood structural panels

$$= \sqrt{\frac{33}{F_y}} \text{ (for } F_y \text{ in ksi) and } = \sqrt{\frac{227.5}{F_y}} \text{ (for } F_y \text{ in MPa) for sheet steel}$$

Examples of a Type I shear walls are shown in Figure 4. A Type I shear wall is required to have hold-downs at each end of each wall segment and design for force transfer around openings where they occur. It should be noted that these equations are only applicable to the nominal shear values given in AISI-Lateral. The equation is composed of four terms which individually contribute to the lateral deflection, d, of the wall: linear elastic cantilever bending (boundary member contribution), linear elastic sheathing shear, a contribution for overall nonlinear effects (incorporates an empirical factor b to account for inelastic behavior), and a lateral contribution from anchorage/hold-down deformation. Figure 5 illustrates the anchorage/hold-down contribution to the horizontal displacement at the top of the wall.

HIGHER ASPECT RATIOS PERMITTED

AISI-Lateral now permits the use of up to a 4:1 aspect ratio for some wood and steel sheathed shear wall assemblies, as identified in Tables C2.1 through C2.1-3. This is an increase from the 2:1 aspect ratio limit found in the 1997 UBC. When the aspect ratio exceeds 2:1 and is less or equal to 4:1, a 2w/h reduction factor on the nominal shear strength of the shear wall must be taken. The reduced strength values were determined through testing by Serrette on 4:1 aspect ratio shear wall assemblies (Ref. 12).

AISI-Lateral Section C2.1 states that "Where a height to

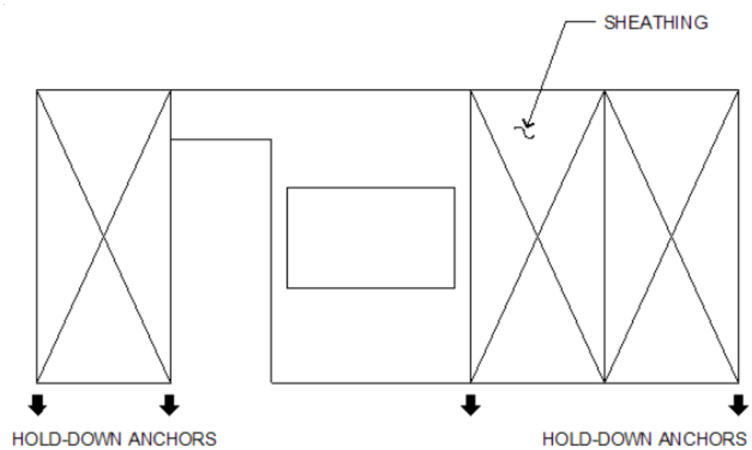

FIGURE 4: AISI-LATERAL FIGURE C2-1 TYPE I SHEAR WALLS

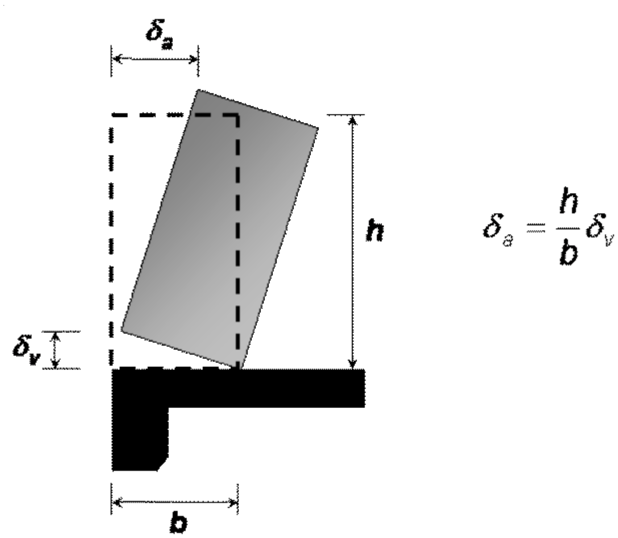

FIGURE 5: AISI-LATERAL FIGURE C2-4 LATERAL CONTIBUTION FROM ANCHORAGE/HOLD-DOWN DEFORMATION

width aspect ratio (h/w) of a shear wall segment is greater than 2:1, as permitted in Tables C2.1-1, C2.1-2 and C2.1-3, the available shear strength shall be multiplied by 2w/h, but in no case shall the height to width aspect ratio (h/w) exceed 4:1." The increased aspect ratio is permitted in the following cases:

- 7/16" OSB, one side, wind loads
- 0.027" steel sheet, one side, wind loads
- 15/32" Structural I sheathing (4-ply), one side, 33 or 43 mils stud and track thickness, seismic loads
- 7/16" OSB, one side, 33, 43, or 54 mils stud & track thickness, seismic loads

TYPE II (PERFORATED) SHEAR WALL DESIGN PROVISIONS

AISI-Lateral Section C3 includes provisions for Type II shear walls, which were not addressed in the 1997 UBC. This is an empirical methodology based on full scale testing.

Type II shear walls are permitted to have openings between the ends of the wall as the tabulated Type I shear wall strength values are reduced. In SDC B through F, Type II shear wall strengths are to be based on the strength of a Type I shear wall with screw spacing at 4" or 6" on center. Type II walls are required to have hold-downs located at each end of the wall line, shear anchorage along the bottom of the wall, as well as uniform uplift anchorage between the wall ends.

Another Type II requirement is that the Type II shear wall segment at each end of a Type II shear wall must comply with the aspect ratio limitations and if they exceed 2:1, where permitted by the shear wall tables, and are less than 4:1, a reduction of 2w/h is required. A Type II shear wall deflection equation has not yet been determined.

WOOD-SHEATHED, CFS-FRAMED DIAPHRAGM TABLES & DESIGN PROVISIONS

AISI-Lateral includes diaphragm assembly strengths and detailing provisions in Section D. Section D2.2 includes a nominal load table for CFS framed diaphragms which was not shown in the 1997 UBC.

The minimum framing member thickness is required to be 33-mil, the minimum screw size is to be a #8, when blocking is required it is to be a minimum of a 1 ½" x 33-mil strap, and the maximum aspect ratio (length / width) is not to exceed 4:1 and 3:1 for blocked and unblocked assemblies, respectively. As in the shear wall tables, the tabulated shear strengths are nominal and are to be divided by a safety factor or multiplied by a resistance factor to obtain ASD or LRFD strengths, respectively. The safety factor for those assemblies resisting wind loads is 2 and for those resisting seismic loads is 2.5.

There are special seismic provisions when the R value used to determine the lateral forces is greater than 3. The aspect ratio is limited to 4:1 when all edges of the wood sheathing are attached to framing or intermittent blocking members. The aspect ratio is reduced to 3:1 when blocking is not provided. The minimum panel width is required to be not less than 24". In addition, there are provisions for open front structures with rigid wood diaphragms.

WOOD-SHEATHED, CFS-FRAMED DIAPHRAGM DEFLECTION EQUATION

AISI-Lateral Section D2.1.1 includes a diaphragm deflection equation D2.1-1, shown below, which was developed after a review of the equations used for wood framed shear walls and diaphragms and performance similarities

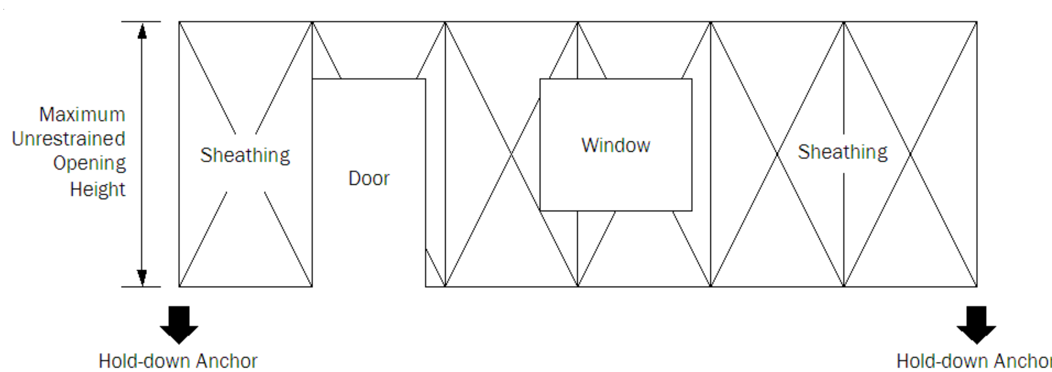

FIGURE 6: AISI S213-07 FIGURE C3-1 TYPE II SHEAR WALLS

Table D2-1

NOMINAL SHEAR STRENGTH FOR DIAPHRAGMS WITH WOOD SHEATHING [1]

(Pounds Per Foot)

Membrane Material	Screw Size	Thickness (in)	Blocked				Unblocked	
			Screw spacing at diaphragm boundary edges and at all continuous panel edges				Screws spaced maximum of 6" on all supported edges	
			6	4	2.5	2	Load perpendicular to unblocked edges and continuous panel joints	All other configurations
			Screw spacing at all other panel edges					
			6	6	4	3		
Structural I	See note 2	3/8	768	1022	1660	2045	685	510
		7/16	768	1127	1800	2255	755	565
		15/32	925	1232	1970	2465	825	615
C-D, C-C and other graded wood structural panels in DOC PS-1 and PS-2	See note 2	3/8	690	920	1470	1840	615	460
		7/16	760	1015	1620	2030	680	505
		15/32	832	1110	1770	2215	740	555

1 For SI: 1" = 25.4 mm, 1 foot = 0.305 m, 1 lb = 4.45 N

FIGURE 6: AISI-LATERAL TABLE D2-1 NOMINAL SHEAR STRENGTH FOR DIAPHRAGMS WITH WOOD SHEATHING

between wood and CFS framed shear wall assemblies. The deflection is to be multiplied by a factor of 2.5 when unblocked diaphragms are used.

Diaphragm deflection is used to determine whether a diaphragm is flexible or rigid and also used to determine if the deflection will adversely affect the walls attached to the diaphragm. There are several cases in which the 2006 IBC and ASCE7-05 permit wood structural panel diaphragms to be idealized as flexible and these cases are described in 2006 IBC Section 1613.6.1 and ASEC7-05 Section 12.3.1.1.

ASCE7-05 Section 12.3.1.3 states that those diaphragms not satisfying the conditions in the aforementioned sections may be idealized as flexible if the calculated in-plane diaphragm deflection is more than two times the average story drift of the vertical elements of the seismic force-resisting system (i.e.; shear walls). For those cases in which they are not permitted to be idealized as flexible, one must determine if they are either flexible or rigid per ASCE7-05.

ASCE7-05 Section 12.14.8.3.1 requires that, when a diaphragm is flexible, the seismic story shear be distributed to the vertical elements of the lateral force resisting system using the tributary area method. ASCE7-05 Section 12.14.8.3.2 requires, that when a diaphragm is not flexible, the seismic story shear be distributed to the vertical elements of the lateral force resisting system based on the relative stiffness of the vertical elements and diaphragm.

Two approaches have typically been used to estimate whether the in-plane diaphragm displacement is excessive causing potential out-of-plane issues with the attached walls. The most typical approach is to comply with the code aspect ratio limits for diaphragms assuming that if one complies, the deflection will not be excessive. The other is to calculate the deflection and compare to the out-of-plane displacement limit for the attached walls. The Applied Technology Council has a publication (Ref. 14) in which they suggest deflection criteria for diaphragms supporting concrete or masonry walls.

$$\delta = \frac{5vL^3}{8E_s A_c b} + \omega_1\omega_2 \frac{vL}{\rho G t_{sheathing}} + \omega_1^{1.25}\omega_2(\alpha)\left(\frac{v}{2\beta}\right)^2 + \frac{\sum_{j=1}^{n}\Delta_{ci}X_i}{2b}$$

(Eq. D2.1-1)

A_c = Gross cross-sectional area of chord member, in square inches
b = Diaphragm depth parallel to direction of load, in feet
Es = Modulus of elasticity of steel = 29,500,000psi
G = Shear modulus of sheathing material, in pounds per square inch
L = Diaphragm length perpendicular to direction of load, in feet
n = Number of chord splices in the diaphragm (considering bothdiaphragm chords)
s = Maximum fastener spacing at panel edges, in inches
$t_{sheathing}$ = Nominal panel thickness, in inches
t_{stud} = Nominal framing thickness, in inches
v = Shear demand (V/2b), in pounds per linear foot
V = Total lateral load applied to the diaphragm, in pounds
X_i = Distance between the "ith" chord-splice and the nearest support (braced wall line), in feet
α = Ratio of the average load per nail based on a non-uniform nail pattern to the average load per nail based on a uniform nail pattern (=1 for a uniformly fastened diaphragm)
β = 810 for plywood and 660 for OSB
δ = Calculated deflection, in inches
Δ_{ci} = Deformation value associated with "ith" chord splice, in inches
ρ = 1.85 for plywood and 1.05 for OSB
ω_1 = s/6 (for s in inches)
ω_2 = 0.33/tstud (for tstud in inches)

SUMMARY

This technical note has highlighted the changes for cold-formed steel framing lateral design for the 2006 IBC compared to the 1997 UBC. These changes included revision to the shear wall resisting wind forces safety factor, additions to shear wall types, expansion of the framing thicknesses that may be used, a shear wall deflection equation, a Type II shear wall design provisions, and added diaphragm deflection equation.

It should be noted that the 2007 AISI Lateral Design standard (AISI S213-07, Ref. 15) has recently been published. It is now a North American standard as it has incorporated provisions for Canada and Mexico. It includes some additional shear wall provisions and clarifications, added provisions and clarifications for strap braced wall assemblies, and added provisions for seismic forces contributed by masonry and concrete walls as well as other concrete or masonry components. This new standard is referenced by the new 2007 AISI North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100-07) and will be discussed in a forthcoming CFSEI Technical Note.

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