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- School – University of Utah (BS & MS)
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 - Current role: Director of Training and Research
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WOOD SHEAR WALLS

AN OVERVIEW AND COMPARISON OF DESIGN
METHODS, LIMIT STATES AND DETAILING



PHIL MILLER, SE, DUNN ASSOCIATES
FEBRUARY 21, 2024



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*Why do all structural engineers wear plaid?
It's like getting a warm hug from Microsoft excel.*

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SHEAR WALL DESIGN METHODS

INDIVIDUAL FULL HEIGHT SEGMENTS
FORCE TRANSFER AROUND OPENING WALLS
PERFORATED WALLS

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SHEAR WALL DESIGN METHODS

- SDPWS 2021
- Individual full height wall segments
- Force Transfer Around Openings (FTAO)
- Perforated walls
- Pros and cons of each



SDPWS 2021

- Adopted by IBC 2021, free at <https://awc.org/publications/2021-sdpws/>
- What's new that we will cover here:
 - Revisions to language and organization on shear wall types
 - Simplified tables for nominal capacity of diaphragms and shear walls for both wind and seismic. Values haven't changed.
 - For ASD capacity, divide nominal capacity by 2.8 (seismic) or 2.0 (wind)
 - For LRFD capacity, multiply nominal capacity by 0.5 (seismic) or 0.8 (wind)
 - Revised shear capacity adjustment factors for perforated shear wall design



SDPWS 2021

- What's new that we will not cover here:
 - Addition of deflection equations for cantilevered diaphragms
 - Addition of CLT shear wall and diaphragm provisions



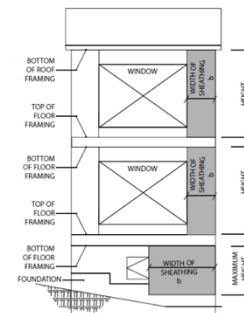
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INDIVIDUAL FULL HEIGHT WALL SEGMENTS

- Designed as shear walls between openings (openings are excluded from analysis)
- Hold downs typically at each end of each segment
- Aspect ratio $h/b < 2.0$, or $h/b < 3.5$ with penalty

Figure 4D Typical Individual Full-Height Wall Segments Height-to-Width Ratio



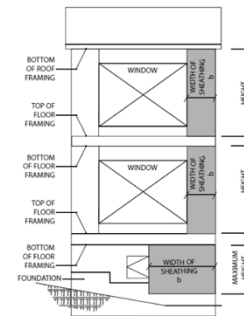
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INDIVIDUAL FULL HEIGHT WALL SEGMENTS

- **PROS:**
 - Simplest design method
 - Requires less coordination with exact opening sizes
 - Easier to communicate at early design stages
- **CONS:**
 - Most hold downs, and hold downs are typically heavier
 - Aspect ratio can be difficult to meet at exterior

Figure 4D Typical Individual Full-Height Wall Segments Height-to-Width Ratio



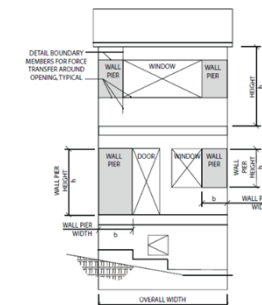
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FORCE TRANSFER AROUND OPENING WALLS (FTAO)

- Designed as shear walls including portions around openings (openings are included in analysis), with force-transfer straps & blocking
- Hold downs typically at ends of overall wall
- Wall and Pier aspect ratio $h/b < 3.5$

Figure 4E Typical Shear Wall Height-to-Width Ratio for Shear Walls Designed for Force Transfer Around Openings



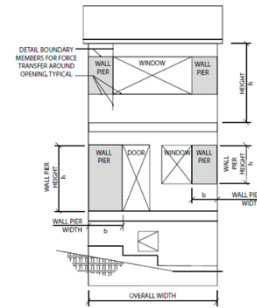
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FORCE TRANSFER AROUND OPENING WALLS (FTAO)

- **PROS:**
 - Fewer hold downs than FH segment design, also they are smaller
 - Aspect ratio is easier to achieve
- **CONS:**
 - More complex design method, difficult to run at early design stages
 - Coordination with exact opening sizes required. Architects may change your window!
 - Does not work around doors (only perforated can do that)

Figure 4E Typical Shear Wall Height-to-Width Ratio for Shear Walls Designed for Force Transfer Around Openings



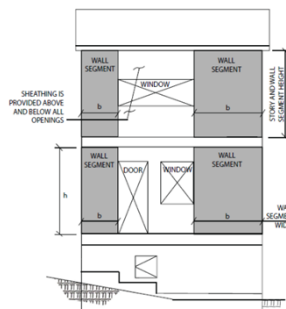
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PERFORATED WALLS

- Designed as shear walls including portions around openings (openings are included in analysis), no force transfer around openings
- Hold downs typically at ends of overall wall
- Wall Segment aspect ratio $h/b < 2.0$, or $h/b < 3.5$ with penalty

Figure 4F Typical Shear Wall Height-to-Width Ratio for Perforated Shear Walls



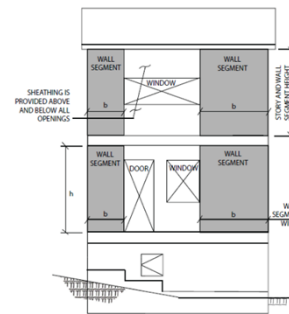
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PERFORATED WALLS

- **PROS:**
 - Fewer hold downs than FH segment design, but they may be larger
 - Aspect ratio is easier to achieve
 - Can work around doors
- **CONS:**
 - More complex design method, difficult to run at early design
 - Coordination with exact opening sizes required. Architects may change your window!
 - Must have parallel top and bottom plates (no sloping top chord: limited to 20' story height)
 - Upper bound on shear capacity limited by testing
 - Hold downs and anchor bolts in uplift

Figure 4F Typical Shear Wall Height-to-Width Ratio for Perforated Shear Walls



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*An indeterminate beam walks into a bar.
The bar tender asks: What do you need?
The beam replies: First, I need a moment..*

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SHEAR WALL LIMIT STATES

SHEAR DEMAND AND CAPACITY
OVERTURNING DEMAND AND CAPACITY
DEFLECTION AND STORY DRIFT



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SHEAR DEMAND SDPWS 4.3.5.5.1

- On a single wall, shear demand must be less than capacity per:
 - $V/b < v_n$ (plf) or
 - $V < v_n * b$ (lb) where
 - V (lb) = force on wall
 - b (ft) = length of wall
 - v_n (plf) = adjusted capacity of wall



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SHEAR DEMAND

SDPWS 4.3.5.5.1

4.3.5.5 Shear Walls in a Line: The provisions of this section are limited to shear distribution to individual shear walls in a shear wall line where the individual shear walls have the same sheathing materials.

4.3.5.5.1 Shear distribution to individual shear walls in a shear wall line shall provide the same calculated deflection, δ_w , in each shear wall.

Exceptions:

1. Shear distribution to wood structural panel individual full-height shear walls in a shear wall line, including shear walls sheathed on opposing sides of common framing and assigned shear capacities in accordance with 4.3.5.4.1, shall be permitted to be taken as proportional to the design shear capacities of the individual full-height shear walls provided that the nominal shear capacities of all full-height shear walls with aspect ratios (h/b) greater than 2:1 are multiplied by $2b/h$. Where multiplied by $2b/h$, the nominal shear capacities need not be further reduced by the adjustment in 4.3.3.2.

- On a group of walls in a line shear demand is distributed to the individual walls in the line by:
 - Stiffness compatibility (deflection of individual walls must match). This requires iteration!
 - By exception, for lines with individual full height wall segments only, permitted to be taken as proportional to capacities of the walls in the line, adjusted by $2b/h$ for $h/b > 2$

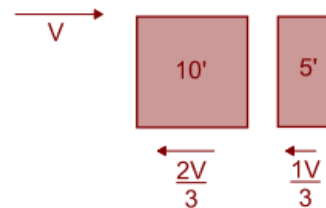


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SHEAR DEMAND

SDPWS 4.3.5.5.1

- Taking the exception is a bit faster, and commonly used for lines that are composed solely of full height segments. If you use the same shear wall assembly (sheathing thickness, nail size and spacing) on the line, the distribution is simply proportional to length. E.g. A wall line with a 10' long wall and a 5' long wall will have 2x the force in the 10' long wall vs. the 5' long wall.
 - $V \cdot 10/15 = 2/3V$ for 10' long wall
 - $V \cdot 5/15 = 1/3V$ for 5' long wall
 - If you take the exception, you must use $2b/h$ reduction on capacity for $h/b > 2$. You cannot use 1.25-0.125h/b from 4.3.3.2.



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SHEAR DEMAND SDPWS 4.3.5.5.1

- If your line of walls uses any FTAO or perforated walls, you must use the stiffness compatibility approach – iteration will be required to match deflection
- Example C4.3.1.1.1-1 in SDPWS commentary compares both approaches for a line of full height segments and achieves similar answers, within 5%.



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SHEAR CAPACITY

- Nominal shear capacity (plf) and stiffness G_a (kips/in) comes from SDPWS Tables 4.3A thru 4.3D.
 - 4.3A – **Wood-based Panels** (OSB or plywood sheathing, structural 1 panels, plywood siding, particleboard and fiberboard)
 - 4.3B – **Wood Structural Panels installed over 1/2" or 5/8" Gypsum Wallboard/Sheathing** (Useful for retrofit strengthening applications, OSB or plywood over drywall)
 - 4.3C – **Gypsum Board, Gypsum Lath and Plaster, and Portland Cement Plaster** (Useful for older construction, historic buildings)
 - 4.4D – **Lumber Shear Walls** (1x6 or 1x8 planks laid horizontally, diagonally (up to 2 layers) or vertically)



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SHEAR CAPACITY

- Nominal shear capacity to design capacity (SDPWS 4.1.4)
 - For ASD capacity, divide nominal capacity by 2.8 (seismic) or 2.0 (wind)
 - For LRFD capacity, multiply nominal capacity by 0.5 (seismic) or 0.8 (wind)
- Let's look at an excerpt of Table 4.3A



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Table 4.3A Nominal Unit Shear Capacities for Sheathed Wood-Frame Shear Walls ^{1,3,6}

Wood-based Panels ⁴															
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ _m (in.)	Nail Type & Size ⁹ Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.)											
				6			4			3			2		
				v _n (plf)	G _a (kips/in.)		v _n (plf)	G _a (kips/in.)		v _n (plf)	G _a (kips/in.)		v _n (plf)	G _a (kips/in.)	
				OSB	PLY		OSB	PLY		OSB	PLY		OSB	PLY	
Wood Structural Panels - Sheathing ^{4,5}	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) ⁸	505	13	9.5	755	18	12	980	24	14	1260	37	18
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17
	3/8 ²	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) ⁸	615	17	12	895	25	15	1150	31	17	1485	45	20
	7/16 ²			670	15	11	980	22	14	1260	28	17	1640	42	21
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) ^{8,10}	870	22	14	1290	30	17	1680	37	19	2155	52	23
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22

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Table 4.3A Nominal Unit Shear Capacities for Sheathed Wood-Frame Shear Walls ^{1,3,6}

Wood-based Panels ⁴																	
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ_m (in.)	Nail Type & Size ⁹ Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.)													
				6		4		3		2							
				v_n	G_a	v_n	G_a	v_n	G_a	v_n	G_a						
				(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)						
				ASD		EQ		W		OSB		PLY		OSB		PLY	
Wood Structural Panels - Sheathing ^{4,5}	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) ⁸	505	180	252	55	18	12	980	24	14	1260	37	18		
	3/8			560	200	280	40	15	11	1090	20	13	1430	32	17		
	3/8 ²	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) ⁸	615	220	307	95	25	15	1150	31	17	1485	45	20		
	7/16 ²			670	240	335	80	22	14	1260	28	17	1640	42	21		
	15/32			730	260	365	65	19	13	1370	25	15	1790	39	20		
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) ^{8,10}	870	310	435	290	30	17	1680	37	19	2155	52	23		
	19/32			950	340	475	430	26	16	1860	33	18	2435	48	22		

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- Nominal unit shear capacities shall be adjusted in accordance with 4.1.4 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.3.6. For specific requirements, see 4.3.7.1 for wood structural panel shear walls, 4.3.7.3 for particleboard shear walls, and 4.3.7.4 for fiberboard shear walls. See Appendix A for common and box nail dimensions.
- Nominal unit shear capacities are permitted to be increased to values shown for 15/32 inch (nominal) sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1 - (0.5 - G)]$, where G = Specific Gravity of the framing lumber from the NDS (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values, G_a , are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for shear walls constructed with either OSB or 3-ply plywood panels. When 4-ply, or 5-ply plywood panels or composite panels are used, G_a values shall be permitted to be increased by 1.2.
- Where moisture content of the framing is greater than 19% at time of fabrication, G_a values shall be multiplied by 0.5.
- Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.
- Galvanized nails shall be hot-dipped or mechanically deposited.
- Galvanized box nails shall be permitted to be substituted for the specified common nails as shown in the table below.

Nail Type & Size Length (in.) x Shank diameter (in.) x Head diameter (in.)	
Common Nail 6d (2 x 0.113 x 0.266) 8d (2-1/2 x 0.131 x 0.281) 10d (3 x 0.148 x 0.312)	Galvanized Box Nail ⁷ 6d (2 x 0.099 x 0.266) 8d (2-1/2 x 0.113 x 0.297) 10d (3 x 0.128 x 0.312)

- Tabulated nominal unit shear capacities are applicable for carbon steel smooth shank nails of the specified type and size.
- Where tension force induced by shear wall overturning is resisted by a hold-down attached to the inside face of the end post, nominal unit shear capacity for shear walls using 10d common nails shall be multiplied by 0.92.

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SHEAR CAPACITY

Shear capacity adjustments:

- Table 4.3A Footnotes 2, 3 and 10. Note 10 is easy to miss!

- Nominal unit shear capacities are permitted to be increased to values shown for 15/32 inch (nominal) sheathing with same nailing provided (a) studs are spaced a maximum of 16 inches on center, or (b) panels are applied with long dimension across studs.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor = $[1 - (0.5 - G)]$, where G = Specific Gravity of the framing lumber from the NDS (Table 12.3.3A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Where tension force induced by shear wall overturning is resisted by a hold-down attached to the inside face of the end post, nominal unit shear capacity for shear walls using 10d common nails shall be multiplied by 0.92.



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SHEAR CAPACITY

Shear capacity adjustments, SDPWS 4.3.3 for aspect ratios, h/b :

- Blocked segmented walls with plywood/OSB sheathing and $2.0 < h/b < 3.5$:
 - Multiply capacity by 1.25-0.125 h/b , or by 2 b/h if using 4.3.5.5.1 exception
- Unblocked segmented walls with plywood/OSB are less common, but have reduced capacity for typical wall framing and $h/b < 2.0$. See SDPWS 4.3.5.3.

Table 4.3.3 Maximum Shear Wall Aspect Ratios

Sheathed Wood-Frame Shear Wall System	Maximum h/b Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1
Particleboard, blocked	2:1
Diagonally-sheathed lumber	2:1
Gypsum wallboard	2:1 ^{1,2}
Portland cement plaster	2:1 ¹
Structural Fiberboard	3.5:1

- Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.
- For wind design of blocked wood structural panel shear walls or structural fiberboard shear walls, with gypsum wallboard on the opposite side, maximum h/b for gypsum wallboard shall be permitted to be 3.5:1.



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SHEAR CAPACITY

- Shear capacity adjustments, SDPWS 4.3.3 for aspect ratios, h/b :
 - FTAO walls – no adjustments, just keep $h/b < 3.5$ for all wall piers
 - Perforated wall segments with $2.0 < h/b < 3.5$:
 - Multiply segment length by $2b/h$ for determining b and Σb .

Table 4.3.3 Maximum Shear Wall Aspect Ratios

Sheathed Wood-Frame Shear Wall System	Maximum h/b Ratio
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Gypsum wallboard	2:1 ^{1,2}
Portland cement plaster	2:1 ¹
Structural Fiberboard	3.5:1

- Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.
- For wind design of blocked wood structural panel shear walls or structural fiberboard shear walls, with gypsum wallboard on the opposite side, maximum h/b for gypsum wallboard shall be permitted to be 3.5:1.



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SHEAR CAPACITY

- Perforated wall capacity

$$V_n = v_n C_o \Sigma b_i \quad (4.3-5)$$

where:

v_n = nominal unit shear capacity, lbs/ft

C_o = shear capacity adjustment factor from Table 4.3.5.6 or calculated using the following equation:

$$C_o = A_{wall} / (3A_o + A_{fhs}) \leq 1.0 \quad (4.3-6)$$

A_{fhs} = total area sheathed with full-height sheathing, ft^2 , regardless of whether individual wall segments meet the aspect ratio limits in 4.3.3.4.

A_o = total area of openings in the perforated shear wall where individual opening areas are calculated as the opening width times the clear opening height, ft^2 .

A_{wall} = total area of a perforated shear wall equal to the length of the perforated shear wall times its height, ft^2 .

Σb_i = sum of perforated shear wall segment lengths b_i , ft. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with 4.3.3.4.



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SHEAR CAPACITY

- Perforated wall capacity

Table 4.3.5.6 Shear Capacity Adjustment Factor, C_o ¹

Percent Full-Height Sheathing (A_{fhs}/A_{wall})	Percentage Wall Area Openings (A_o / A_{wall})									
	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%
	Shear Capacity Ratio, C_o									
10%	1.00	1.00	1.00	1.00	0.77	0.63	0.53	0.45	0.40	0.36
20%	1.00	1.00	1.00	0.91	0.71	0.59	0.50	0.43	0.38	-
30%	1.00	1.00	1.00	0.83	0.67	0.56	0.48	0.42	-	-
40%	1.00	1.00	1.00	0.77	0.63	0.53	0.45	-	-	-
50%	1.00	1.00	0.91	0.71	0.59	0.50	-	-	-	-
60%	1.00	1.00	0.83	0.67	0.56	-	-	-	-	-
70%	1.00	1.00	0.77	0.63	-	-	-	-	-	-
80%	1.00	0.91	0.71	-	-	-	-	-	-	-
90%	1.00	0.83	-	-	-	-	-	-	-	-
100%	1.00	-	-	-	-	-	-	-	-	-



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OVERTURNING DEMAND

4.3.6.1.2 Tension and Compression Chords: Tension force, T , and a compression force, C , resulting from shear wall overturning forces at each story level shall be calculated in accordance with the following:

$$T = C = V(h)/b_{eff} \quad (4.3-7)$$

where:

C = compression force, lbs

h = shear wall height, ft

T = tension force, lbs

V = shear force induced by the design load, lbs

b_{eff} = effective length (e.g. moment arm) for calculation of T and C force, ft

- Overturning loads are commonly resolved into tension and compression at the end of the wall, which is resisted by chords

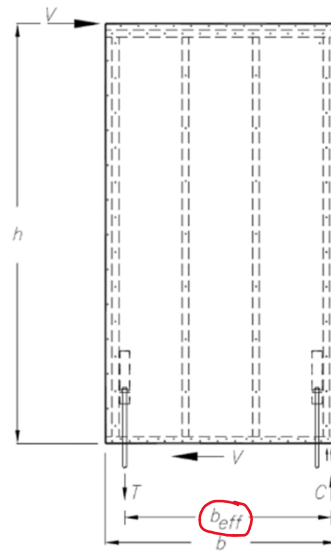
- Note use of b_{eff} in denominator is new to 2021. Previous equation was $T=C=V(h)/L = vh$
- b_{eff} is the moment arm between the compression and tension chords, and should account for the distance from the end of the wall to the center of the tension rod / hold down / tie strap.



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Figure C4.3.6.1.2 Tension and Compression Force Couple due to Shear Wall Overturning Moment



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OVERTURNING DEMAND

- For perforated walls, overturning loads are increased by $1/C_o$

$$T = C = \frac{Vh}{C_o \sum b_i} \quad (4.3-8)$$

where:

C_o = shear capacity adjustment factor from Table 4.3.5.6

V = shear force induced by the design load in the perforated shear wall, lbs

$\sum b_i$ = sum of perforated shear wall segment lengths, b_i , ft. Lengths of perforated shear wall segments with aspect ratios greater than 2:1 shall be adjusted in accordance with 4.3.3.4.

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OVERTURNING DEMAND

Shear Walls: The maximum unit shear force induced by the design load, v_{max} , transmitted into the top of a perforated shear wall, out of the base of the perforated shear wall at full height sheathing, and into collectors connecting shear wall segments, shall be calculated in accordance with the following:

$$v_{max} = \frac{V}{C_o \sum b_l} \quad (4.3-9)$$

4.3.6.4.2 Uplift Anchorage at Shear Wall Ends: Where the dead load stabilizing moment is not sufficient to prevent uplift due to overturning moments on the wall (from 4.3.6.1.2 or 4.3.6.1.3), an anchoring device shall be provided at the end of each shear wall.

4.3.6.4.2.1 Uplift Anchorage for Perforated Shear Walls: In addition to the requirements of 4.3.6.4.2, perforated shear wall bottom plates at full height sheathing shall be anchored for a uniform uplift force, t , equal to the maximum unit shear force induced by the design load, v_{max} , determined in 4.3.6.4.1.1, or calculated by rational analysis.

- Code allows compression from dead load moment to offset tension from overturning
- For perforated walls, you need to design for uplift on the wall segments in addition to the chord anchorage per 4.3.6.4.2.1.
 - Foundation has anchor bolts and plate washers, which helps resist uplift, but upper floors may need straps, screws or overlapping plywood on the rim.



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OVERTURNING DEMAND

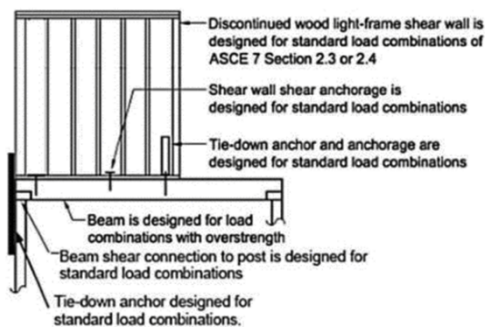


FIGURE C12.3-5 Discontinued Wood Light-Frame Shear Wall

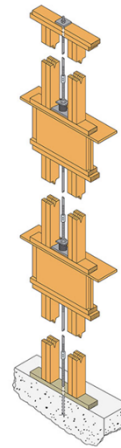
- ASCE 7 requires overturning forces to be amplified by Ω_0 (typically 2.5 or 3) for the design of the element supporting the discontinuous hold down, but not necessarily for the hold down itself or its anchorage. Foundations are exempt also. Examples:
 - Steel transfer beams
 - Transfer slabs in 2-stage construction. Are you modeling your PT slab with hold down loads at Ω_0 ?
- A little help from load combinations since full seismic is not concurrent with full live.
- $1.2D + 1.6L$ vs $1.2D + (0.5 \text{ or } 1.0)L + \Omega_0 E_h + E_v$



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OVERTURNING CAPACITY

- Tensile capacity is provided by straps, hold downs, or ATS rod assemblies
 - Common suppliers: Simpson, Mitek/USP
 - Straps and hold downs can be doubled up for more capacity. They can be shared at corners of intersecting shear walls. ATS rod assemblies should not be shared at corners.

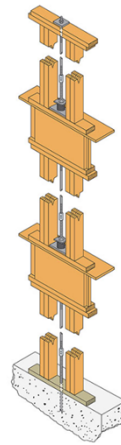
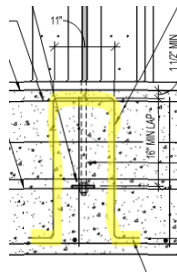
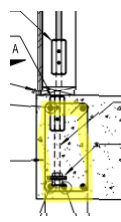
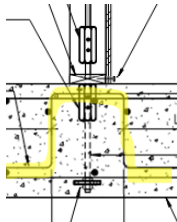


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OVERTURNING CAPACITY

- ATS rod assemblies (typically a deferred submittal)
 - Studs and rod size increase from top to bottom per increased T/C forces provided by EOR
 - Pay careful attention to anchorage due to high forces. Use rebar to restrain concrete break out per ACI.
 - Compression stud packs can get wide at the base – watch out for MEP coordination in these areas

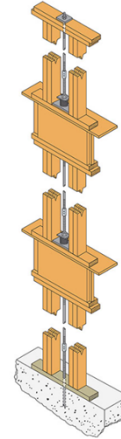


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OVERTURNING CAPACITY

- ATS rod assemblies (typically a deferred submittal)
 - Shrinkage compensation devices should be used at each floor to ensure rod is fully engaged after wood building shrinks (typically at least 1/8" to 1/4" per floor, possibly more)
 - Watch out for high-strength rod. 50ksi rod is stronger than 36ksi rod, but since deflection is based on PL/AE, the smaller area of the high strength rod will give you more stretch and deflection



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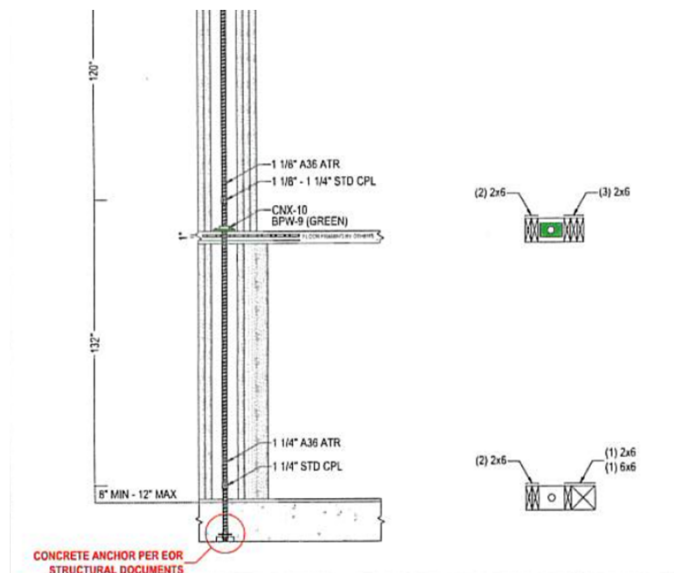
37

OVERTURNING CAPACITY

- Compressive capacity is provided by studs or posts at the ends of the shear wall
 - For typical floor heights and DF-L #2, a 2x4 is good for 2 kips compression (buckling controlled), a 2x6 is good for 5 kips compression (controlled by crushing of sill plate at 625psi).
 - Larger posts may be required to be compatible with selected tension hardware (must fit strap or tested width of hold down)
 - For ATS systems, the final stud layout will be provided with the deferred submittal

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MIDPOINT QUIZ

- LIMERICK FILL-IN-THE-BLANK
- I READ THE LIMERICK, AND YOU RAISE YOUR HAND IF YOU KNOW THE MISSING LAST WORD. (IT WILL RHYME WITH FIRST 2 LINES)

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MIDPOINT QUIZ

LIMERICK 1

With windows and doors decorated
 Empirical design-related
 My panels and piers
 Have unequal shear
 The shear wall is _____



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MIDPOINT QUIZ

LIMERICK 2

From the high roof I run to the ground
 Against overturning I'm no clown
 My hardware and plates
 Wood shrink compensate
 My rod and my studs make a _____



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DEFLECTION/STORY DRIFT

- 3 term equation

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$

- 4 term equation (commentary)

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{G_v t_v} + 0.75he_n + \frac{h}{b}\Delta_a$$

(bending, end post deformation excluding anchorage slip)
 (shear, panel shear deformation)
 (shear, panel nail slip)
 (bending, wall anchorage deformation)

$G_v t_v$ = shear stiffness, lb/in. of panel depth. See Table C4.2.3A or C4.2.3B.

e_n = nail slip, in. See Table C4.2.3D.

where:

b = shear wall length, ft

Δ_a = vertical deformation of the wall overturning anchorage system (including but not limited to fastener slip, device elongation, rod elongation, and uncompensated shrinkage) plus the vertical compression deformation, the effects of which are measured at the ends of the shear wall and associated with the unit shear force induced by the design load in the shear wall, in.

E = modulus of elasticity of end posts, psi

A = area of end post cross-section, in.²

G_a = apparent shear wall shear stiffness from nail slip and panel shear deformation, kips/in. (Tables 4.3A, 4.3B, 4.3C, or 4.3D)

h = shear wall height, ft

v = unit shear force induced by the design load, lbs/ft

δ_{sw} = maximum shear wall deflection determined by elastic analysis, in.



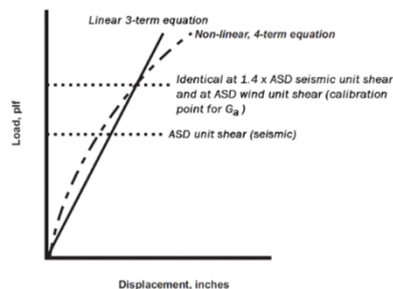
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DEFLECTION/STORY DRIFT

- From SDPWS commentary

Figure C4.3.4 Comparison of 4-Term and 3-Term Deflection Equations



In SDPWS Equation 4.3-1, deflection due to panel shear and nail slip are accounted for by a single apparent shear stiffness term, G_a . **Calculated deflection, using either the 4-term (Equation C4.3.4-1) or 3-term equation (SDPWS Equation 4.3-1 and Equation C4.3.4-2), are identical at 1.4 times the allowable shear value for seismic (see Figure C4.3.4) to facilitate use for checking seismic drift at strength level forces in accordance with ASCE 7.** This calibration point also coincides with ASD unit shear capacities for wind and is also suitable for calculating deflections from wind load induced unit shears up to the ASD unit shear capacity. **Below the calibration point, calculation using the 3-term equation overestimates deflection relative to the 4-term equation but such overestimations are generally negligible for design purposes.** These small differences, however, can influence load distribution assumptions based on relative stiffness if both deflection calculation methods are used in a design. **For consistency and to minimize calculation-based differences, either the 4-term equation or 3-term equation should be used for all shear walls in a line.**



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DEFLECTION/STORY DRIFT

- Don't forget to multiply v (ASD) by 1.4 to get v (LRFD) to get elastic deflection at strength level
- Multiply elastic deflection by C_d/I_e per ASCE 7 and compare with allowable story drift, Δ_a

The deflection at level x (δ_x) (in. or mm) used to compute the design story drift, Δ , shall be determined in accordance with the following equation:

$$\delta_x = \frac{C_d \delta_{se}}{I_e} < \Delta_a \quad (12.8-15)$$

where

C_d = deflection amplification factor in Table 12.2-1;

δ_{se} = deflection at the location required by this section determined by an elastic analysis; and

I_e = Importance Factor determined in accordance with Section 11.5.1.

$$\Delta_a = 0.02h \text{ (Cat. II)}$$

$$0.015h \text{ (Cat. III)}$$

$$0.010h \text{ (Cat. IV)}$$



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DEFLECTION/STORY DRIFT

Table 12.12-1 Allowable Story Drift, $\Delta_a^{a,b}$

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}^c$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures ^d	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

^a h_{sx} is the story height below level x .

^bFor seismic force-resisting systems solely comprising moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.

^cThere shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

^dStructures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.



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DEFLECTION/STORY DRIFT

- For non-slender aspect ratios, shear stiffness will dominate the behavior
- For slender aspect ratios, anchorage slip and flexure terms will become dominant, so upsizing hold downs can be effective to limit deflection. You can scale published value by stress ratio to sharpen your pencil.



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DEFLECTION/STORY DRIFT

- Remember to be consistent with deflection checks along a shear wall line:
 - Don't mix 3-term and 4-term equation. Shear distribution is proportional to stiffness.
 - If you take the exception to 4.3.5.5.1, you must use $2b/h$ reduction on capacity for $h/b > 2$. You cannot use $1.25-0.125h/b$ from 4.3.3.2.
- Watch out for G_a for plywood vs. OSB. OSB is 1.2x to 2.3x stiffer than plywood, but since OSB cannot be fire treated, you're stuck using plywood in Type 3 buildings at the exterior walls or where FRT is required (typically 5 wood stories)



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Table 4.3A Nominal Unit Shear Capacities for Sheathed Wood-Frame Shear Walls ^{1,3,6}

Wood-based Panels ⁴																			
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Nail Bearing Length in Framing Member or Blocking, ℓ _m (in.)	Nail Type & Size ⁹ Length (in.) x Shank diameter (in.) x Head diameter (in.)	Panel Edge Nail Spacing (in.)															
				6				4				3				2			
				v _n		G _a		v _n		G _a		v _n		G _a		v _n		G _a	
				(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)	(plf)	(kips/in.)				
				OSB		PLY		OSB		PLY		OSB		PLY					
Wood Structural Panels - Sheathing ^{4,5}	5/16	1-1/4	6d common nail (2 x 0.113 x 0.266) ⁸	505	13	9.5	755	18	12	980	24	14	1260	37	18				
	3/8			560	11	8.5	840	15	11	1090	20	13	1430	32	17				
	3/8 ²	1-3/8	8d common nail (2-1/2 x 0.131 x 0.281) ⁸	615	17	12	895	25	15	1150	31	17	1485	45	20				
	7/16 ²			670	15	11	980	22	14	1260	28	17	1640	42	21				
	15/32			730	13	10	1065	19	13	1370	25	15	1790	39	20				
	15/32	1-1/2	10d common nail (3 x 0.148 x 0.312) ^{8,10}	870	22	14	1290	30	17	1680	37	19	2155	52	23				
	19/32			950	19	13	1430	26	16	1860	33	18	2435	48	22				
					G _a (OSB)		G _a (PLY)		1.2-1.6x		1.3-1.8x		1.4-1.9x		1.6-2.3x				

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DEFLECTION/STORY DRIFT

4.3.4.2 Deflection of Perforated Shear Walls: The deflection of a perforated shear wall shall be calculated in accordance with 4.3.4, where v in equation 4.3-1 is equal to v_{max} obtained in equation 4.3-9 and b is taken as Σb_i .

$$v_{max} = \frac{V}{C_o \Sigma b_i}$$

$$\delta_{sw} = \frac{8vh^3}{EA(\Sigma b_i)} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$



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DEFLECTION/STORY DRIFT

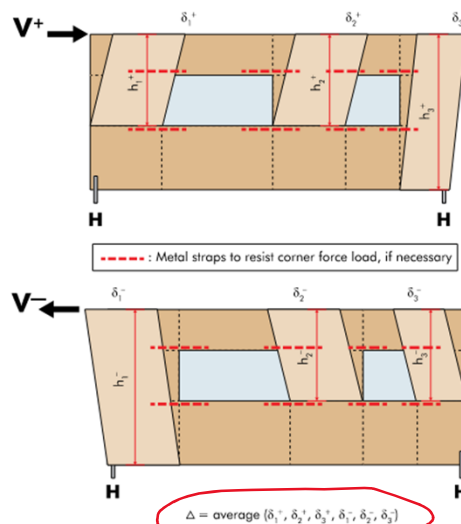
- FTAO walls – recommend using FTAO wall calculator at APA website
<https://www.apawood.org/ftao>
- What rational analysis do they use? (see technical note T555)

Deflection of wood structural panel shear walls shall be permitted to be calculated using approved alternate procedures that follow a rational analysis where apparent shear stiffness accounts for panel shear deformation and non-linear nail slip in the sheathing to framing connection.

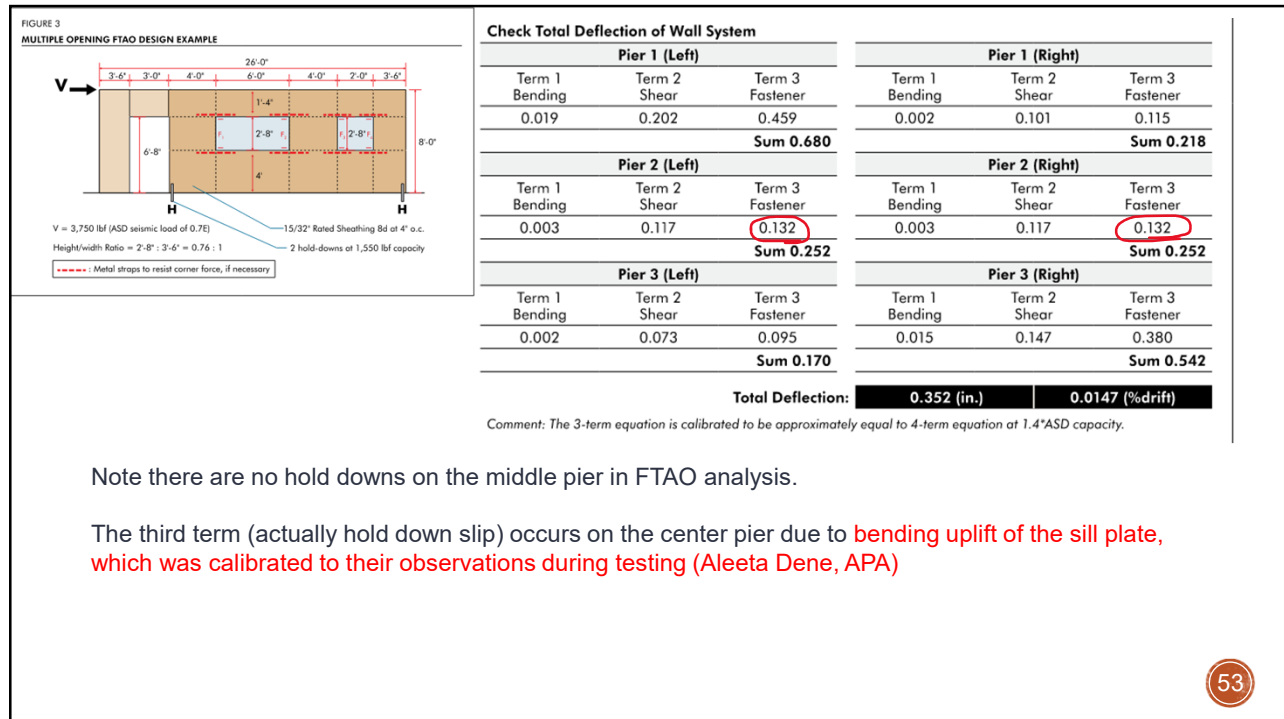


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FIGURE 2
CONCEPTUAL METHODOLOGY FOR ESTIMATING DEFLECTION OF WALLS WITH OPENINGS



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QUESTION MINI-BREAK

- 1 OR 2 QUESTIONS

54

*Why did the engineer cross the road?
That's how it was done on the last project.*

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COMPARISON OF METHOD RESULTS

INDIVIDUAL FULL HEIGHT SEGMENTS
FORCE TRANSFER AROUND OPENING WALLS
PERFORATED WALLS

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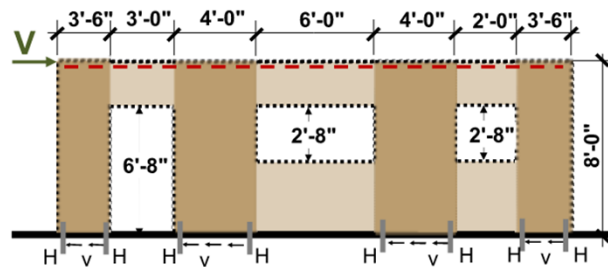
COMPARISON

- Courtesy of "Shear Exhilaration" by Aleeta Dene, APA
- For complete presentation go to <https://www.apawood.org/shear-exhilaration> or google "shear exhilaration APA"



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Segmented Approach



$V = 3,750 \text{ lbs}$

Code Limitation

height/width Ratio = 8:3.5

$1.25 - 0.125h/b = 1.25 - 0.125(8/3.5) = 0.964$

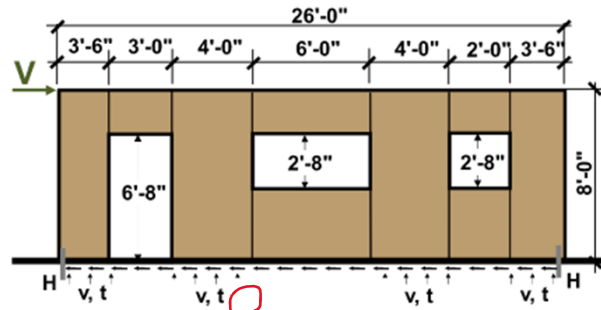
OR

$2b/h = 2(3.5)/8 = .875$



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Perforated Shear Wall Approach



$V = 3,750 \text{ lbs}$

Height/width ratio = 8:3.5

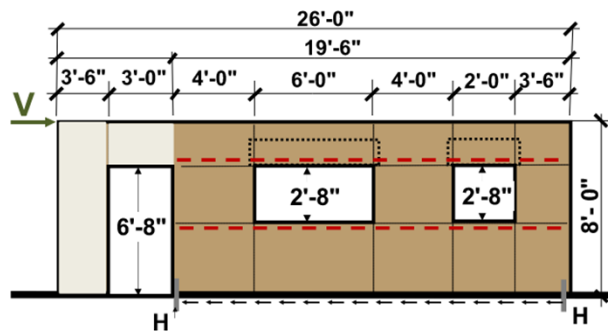
$2b/h = (2)(3.5)/8 = 0.875$

Note the uplift on full height segments of perforated design



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FTAO Approach



$V = 3,750 \text{ lbs}$

Note the pier left of the door is excluded

Height/width ratio =
 $2'-8'' : 3'-6'' = 0.76:1$



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Comparison

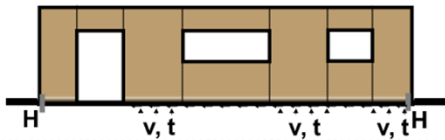
Segmented Approach



15/32 Category Rated Sheathing

8d @ 4" o.c. (3'-6" walls)
 8d @ 6" o.c. (4' walls)
 8 – hold-downs @ 2,000+ lb capacity

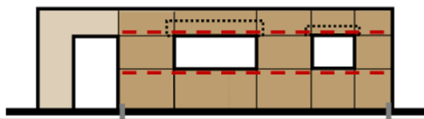
Perforated



15/32 Category Rated Sheathing

8d @ 4" o.c.
 2 – hold-downs @ 2,435 lb capacity
 extensive plate anchorage

Force Transfer



15/32 Category Rated Sheathing

8d @ 4" o.c.
 2 – hold-downs @ 1,550 lb capacity
 2 Straps – 865 lb



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	SEGMENTED	FTAO
PROS	-Simplest design method -Requires less coordination with exact opening sizes -Easier to communicate at early design stages	-Fewer hold downs than segmented design, also they are smaller -Aspect ratio is easier to achieve
CONS	-Most hold downs, and hold downs are typically heavier -Aspect ratio can be difficult to meet at exterior	-More complex design method, difficult to run at early design stages -Coordination with exact opening sizes required. Architects may change your window! -Does not work around doors (only perforated can do that)

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	PERFORATED
PROS	-Fewer hold downs than segmented design, but they may be larger -Aspect ratio is easier to achieve -Can work around doors
CONS	-More complex design method, difficult to run at early design stages -Coordination with exact opening sizes required. Architects may change your window! -Must have parallel top and bottom plates (no sloping top chords), limited to 20' story height -Penalties on shear capacity, hold downs and anchor bolts (uplift), max capacity limited by testing, so double sided walls don't really help

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*Live in the $wL^2/8$
Limit your Mc/I
Do things for the $wL/2$ joy of it*

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OTHER CONSIDERATIONS

LOAD PATH
DETAILING
RIM BOARD SHEAR TRANSFER

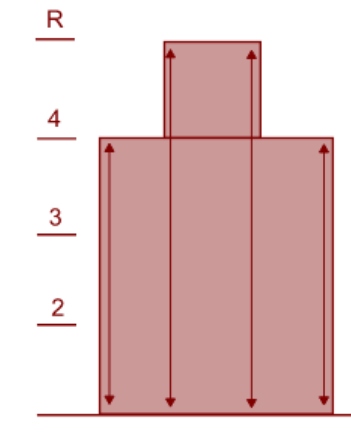
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LOAD PATH

4.3.6.4.4 Load Path: A load path to the foundation shall be provided for uplift, shear, and compression forces. Elements resisting shear wall forces contributed by multiple stories shall be designed for the sum of forces contributed by each story.



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LOAD PATH

- Non-stacking walls - Ω_o for beam or transfer slab design. Other items designs for standard load combinations
- ASCE 7-16 Commentary

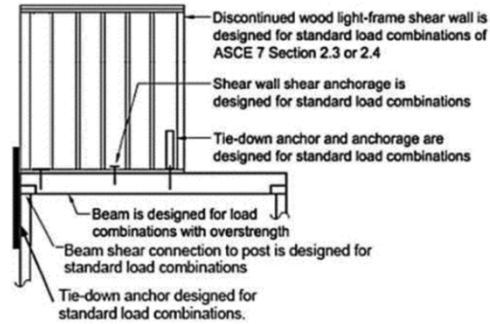


FIGURE C12.3-5 Discontinued Wood Light-Frame Shear Wall

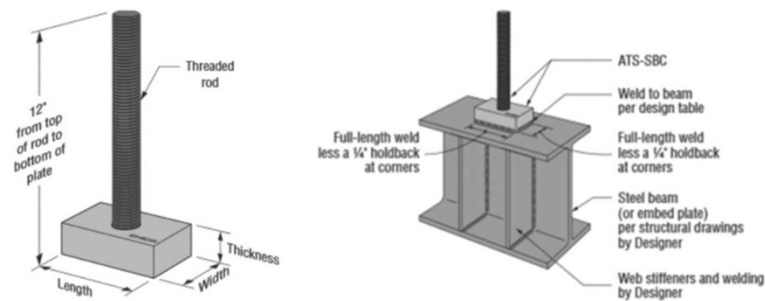


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LOAD PATH

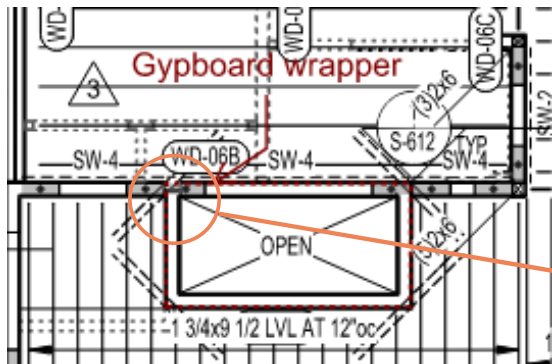
- Hold downs on steel beams. Simpson ATS-SBC is a useful attachment



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DETAILING



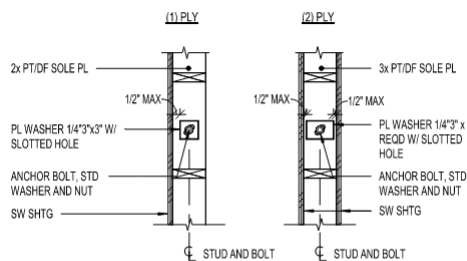
- Shaft walls meeting non-shaft walls – gypboard interrupts the line, so design as separate shear walls



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DETAILING



TYPICAL ANCHOR BOLTS AT SHEAR WALLS (PLAN VIEW)
(SEE NOTES 11 & 12)

- Plate washer placement around anchor bolts – within 1/2" of sheathed face
- Double sided difficulty
 - Inspections and trades access

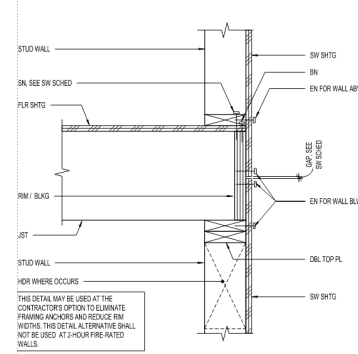
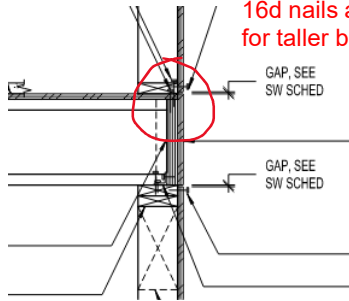
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RIM BOARD SHEAR TRANSFER

Several options

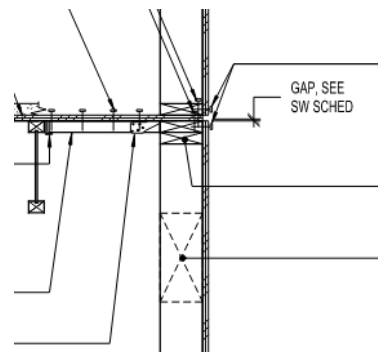
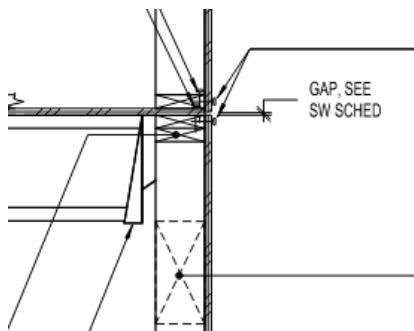
Check LVL manufacturer for min rim width – need 1 ¾" LVL for (2) rows of 16d nails at 6"oc, which isn't enough for taller buildings



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RIM BOARD SHEAR TRANSFER

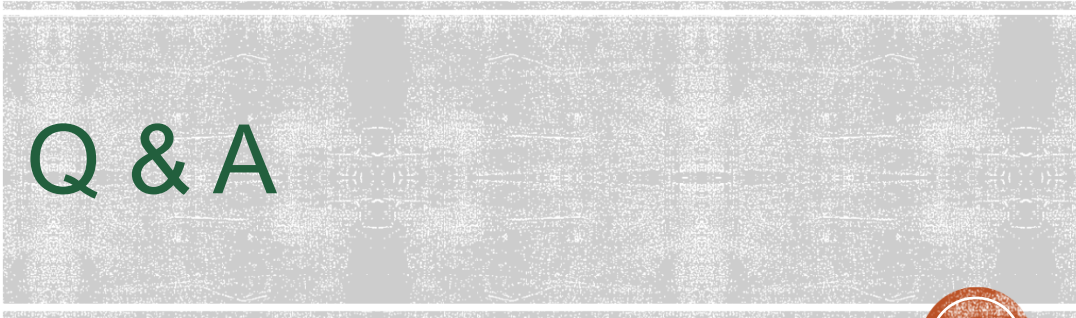


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*To the optimist the glass half-full
To the pessimist half-empty
To the engineer the glass is twice as large as need be*

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Q & A

ASK ANYTHING ABOUT WHAT WE COVERED

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FINAL QUIZ

▪ LIMERICK FILL-IN-THE-BLANK PART 2



FINAL QUIZ

LIMERICK 3

Between segments long straps I prefer
 Fewer hold downs at piers you infer
 I can't handle doors
 But windows galore
 The wall utilizes _____



FINAL QUIZ

LIMERICK 4

After this class I'm now a wood whiz
And with confidence I pass the quiz
The shear walls I know
Will help design flow
I'm learning to like the new _____



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