Wood-Frame Shear Wall and Diaphragm Design

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Course Description

This course is intended for structural engineers and building designers seeking an overview of design steps, considerations and detailing best practices related to the wind- and seismic-resistive design of wood-frame diaphragms and shear walls. It provides an overview of relevant 2015 International Building Code (IBC) provisions and American Wood Council (AWC)-referenced standards, a discussion of common design errors, and guidance related to load path continuity. Discussion will cover diaphragm load paths, chords, collectors and openings, as well as shear wall components, construction options, overturning restraint systems and detailing considerations. Design examples will be used to illustrate key principles and code provisions.

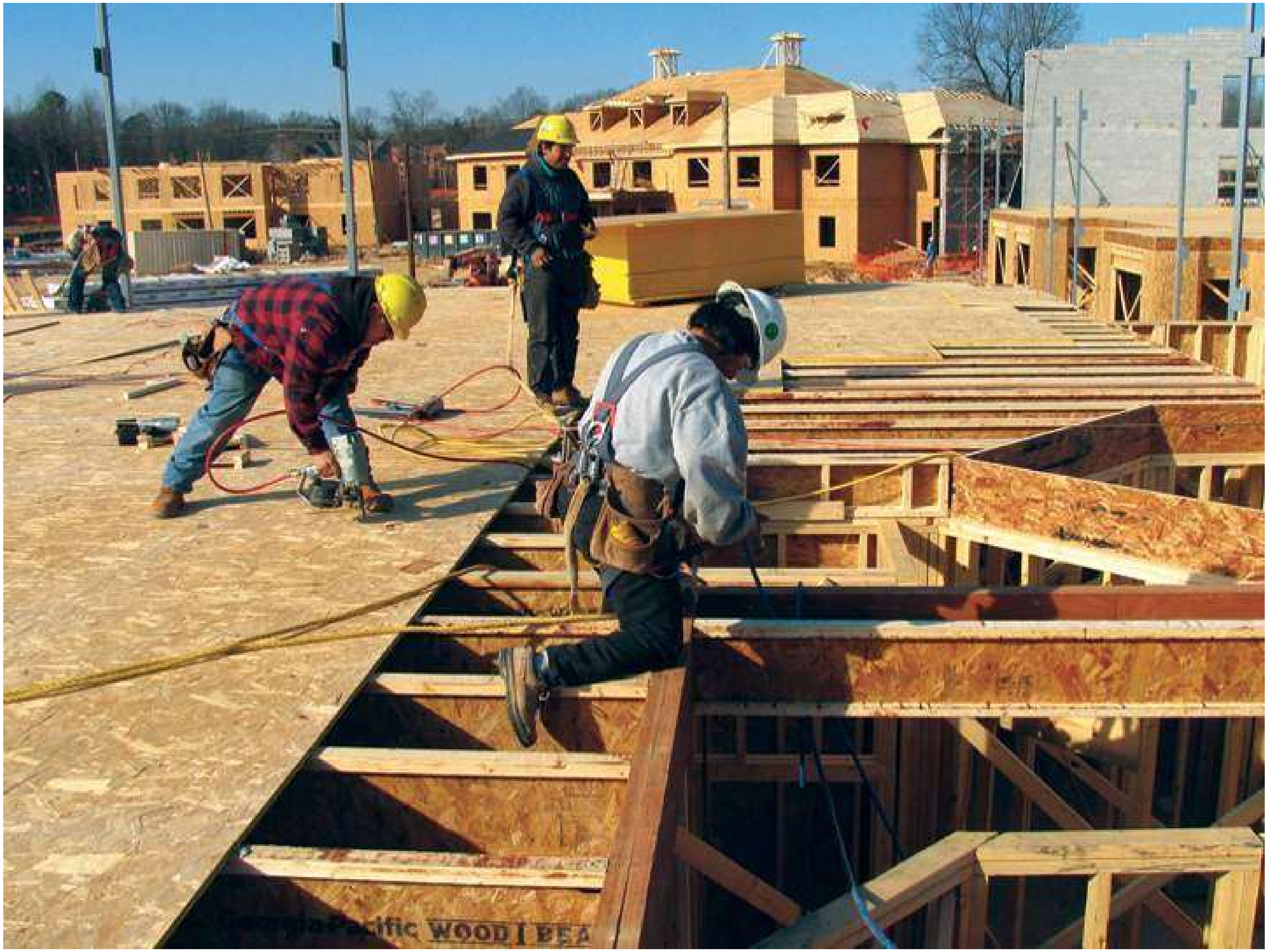
Learning Objectives

1. Review seismic and wind load paths in wood-frame structures.
2. Discuss relevant code and referenced standard provisions related to the design of shear walls and diaphragms.
3. Highlight methods for designing wood-frame diaphragms and related components including chords and collectors.
4. Explore shear wall design principles and highlight three code-compliant configuration options for solid walls vs. walls with openings.

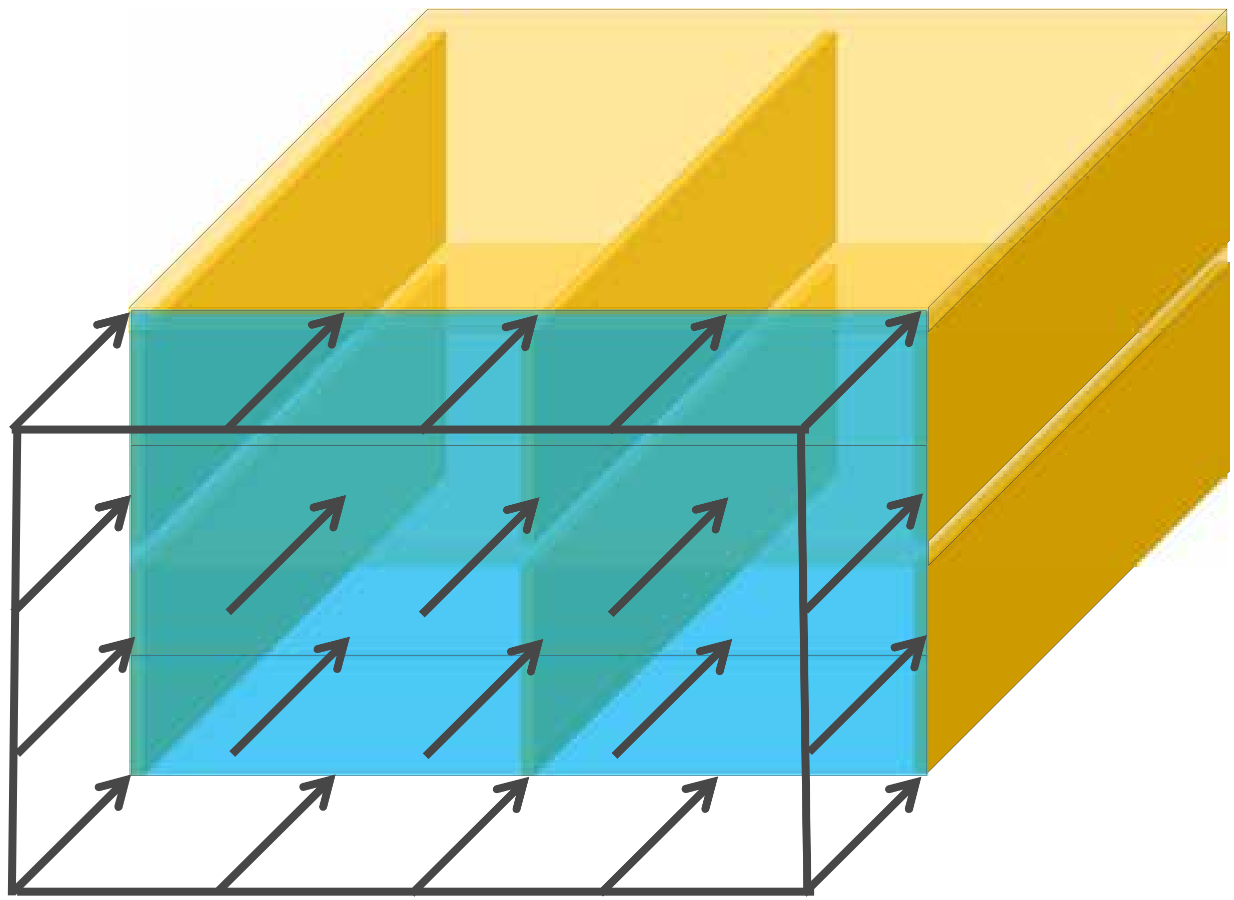
Overview

- Diaphragms
- Shear Walls

Diaphragm Design

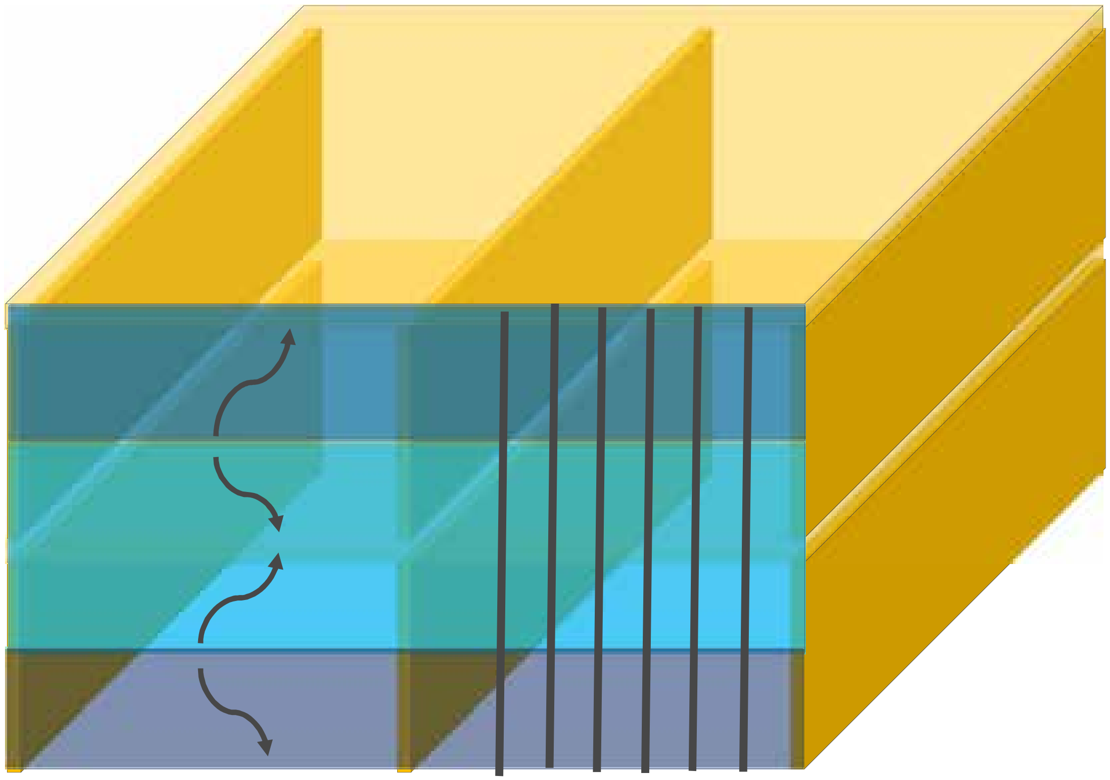


Multi-Story Wind Load Design



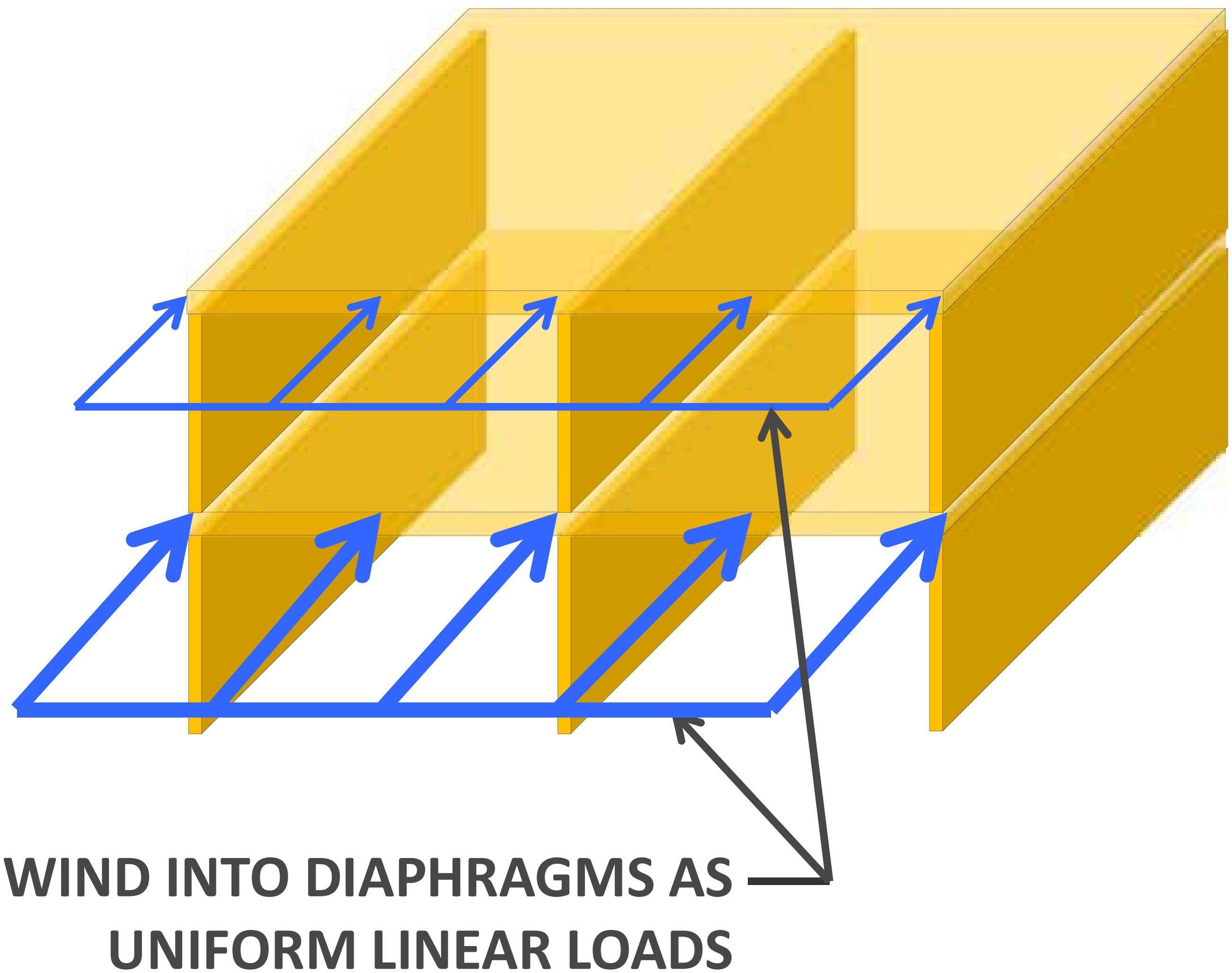
WIND SURFACE
LOADS ON WALLS

Multi-Story Wind Load Design

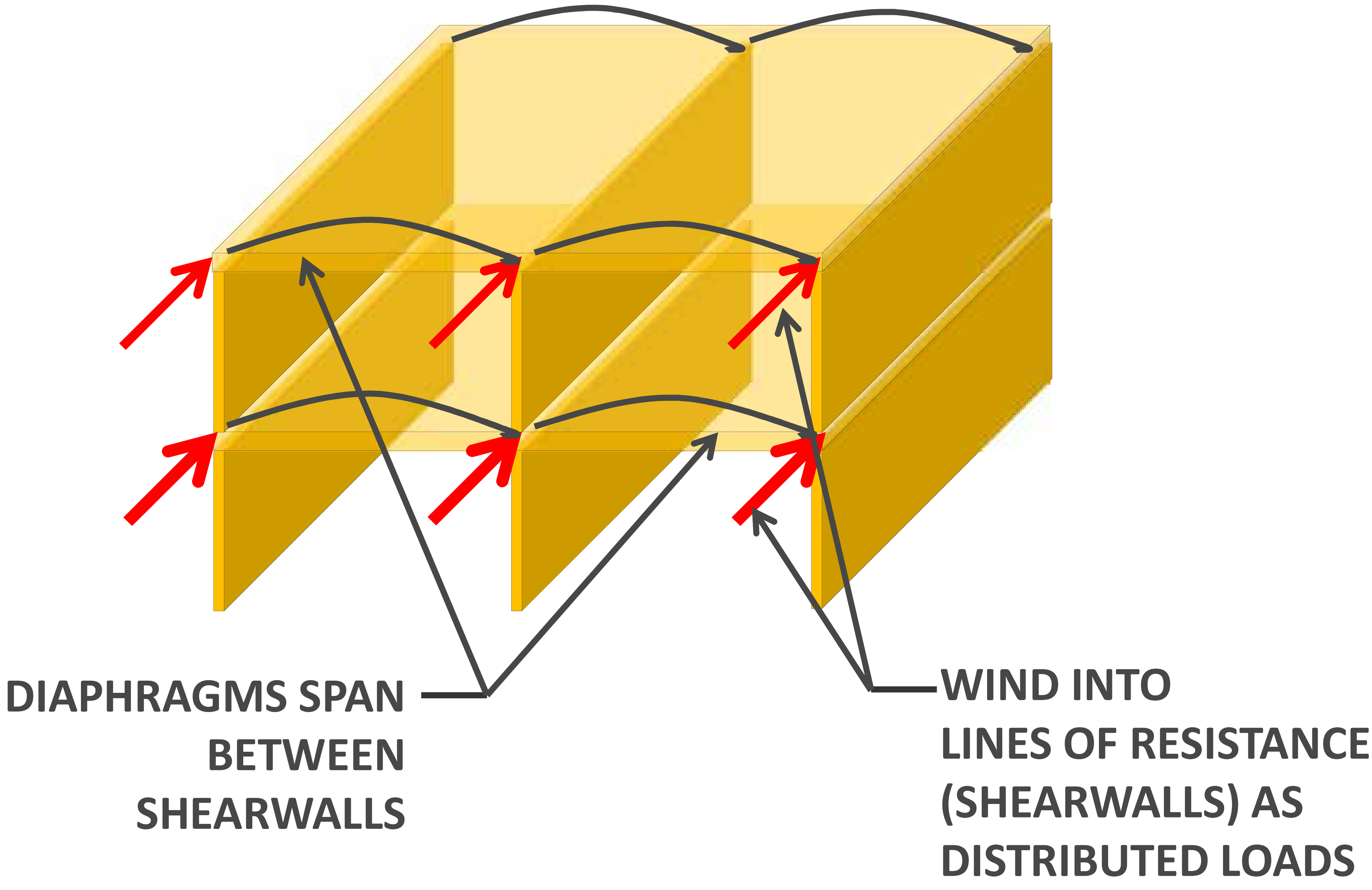


STUDS RESIST SURFACE LOADS IN BENDING, STUD REACTIONS DISTRIBUTE SURFACE LOADS TO DIAPHRAGMS

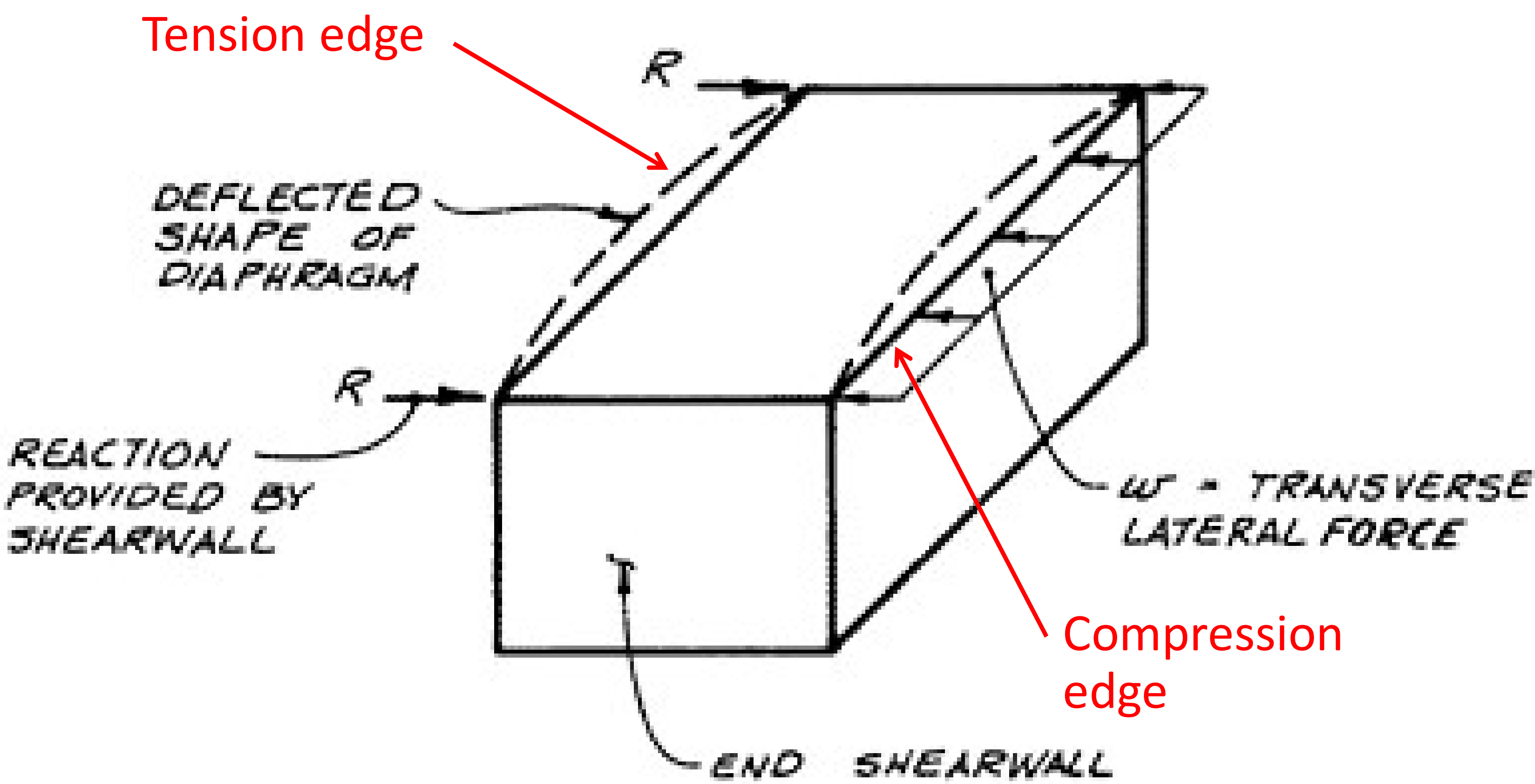
Multi-Story Wind Load Design



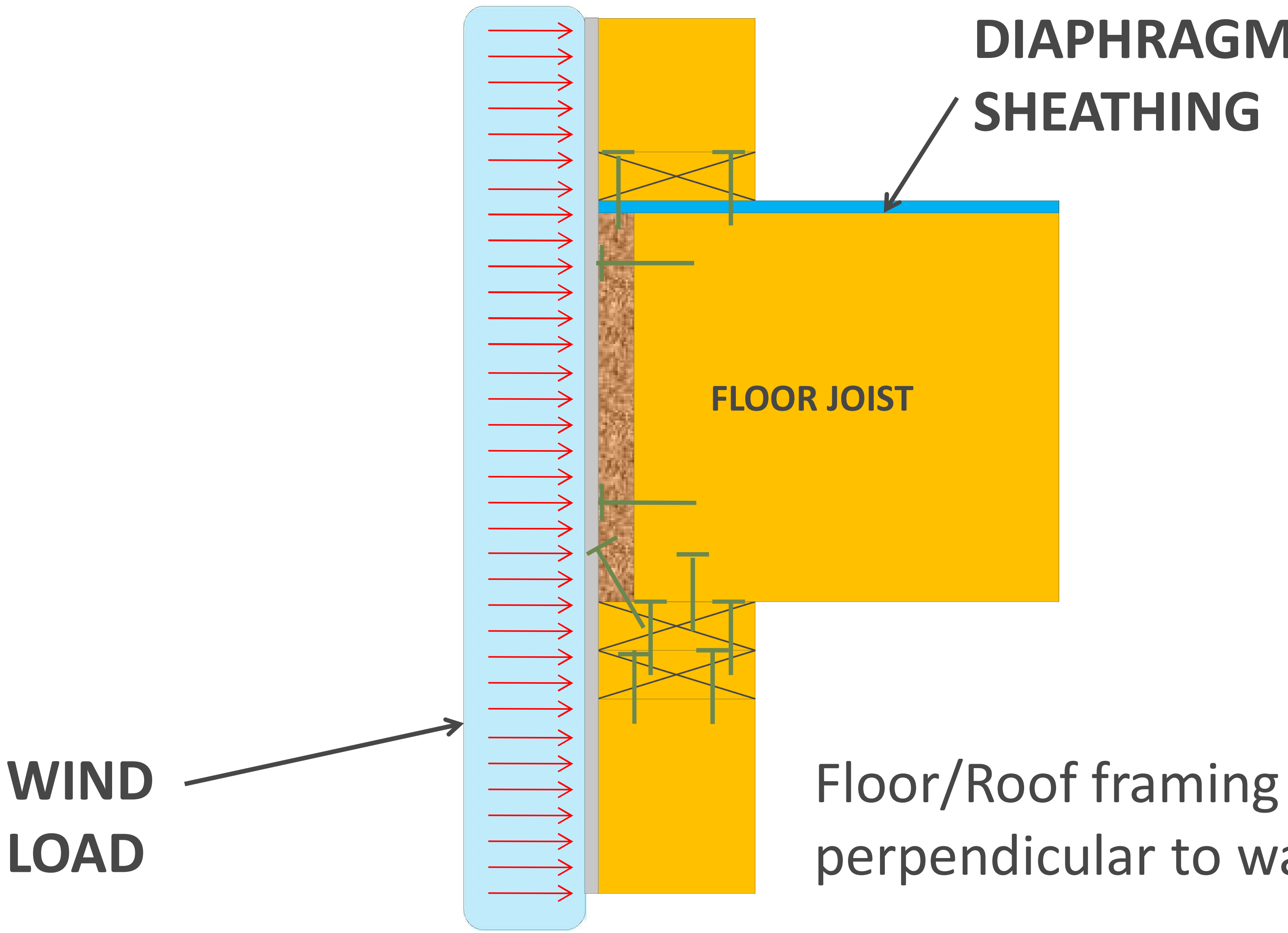
Multi-Story Wind Load Design



Diaphragm – Bending Member

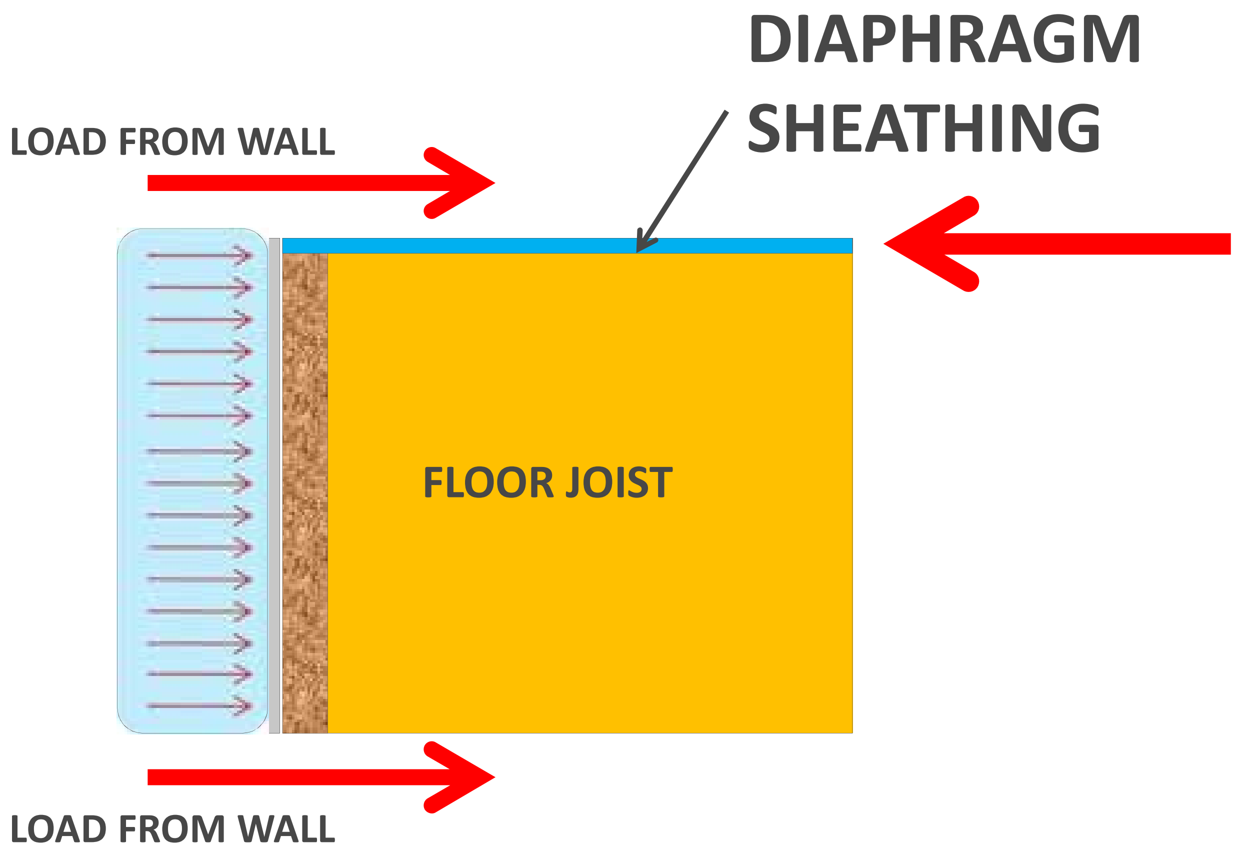


Stud to Diaphragm



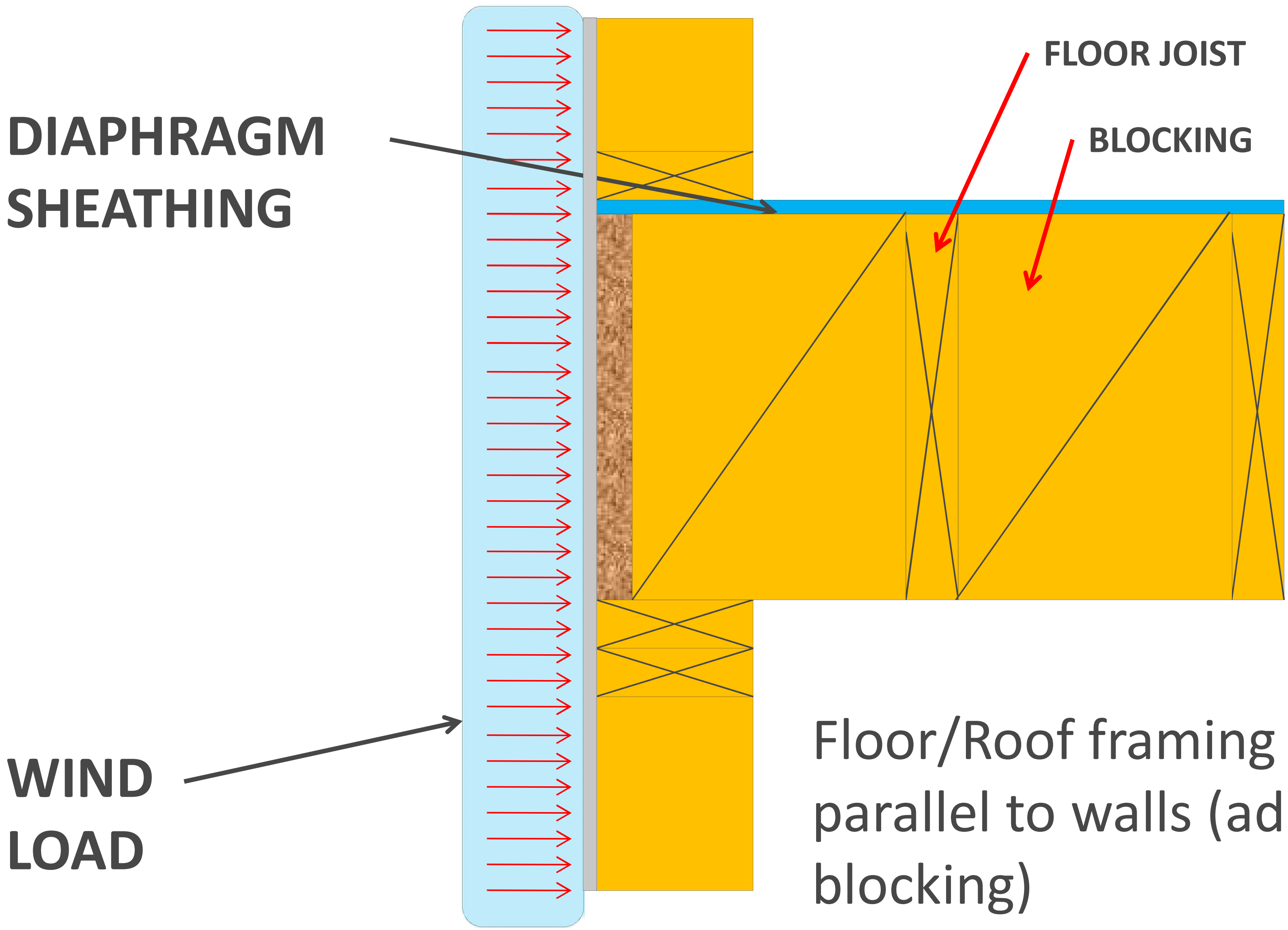
Floor/Roof framing perpendicular to walls

Stud to Diaphragm



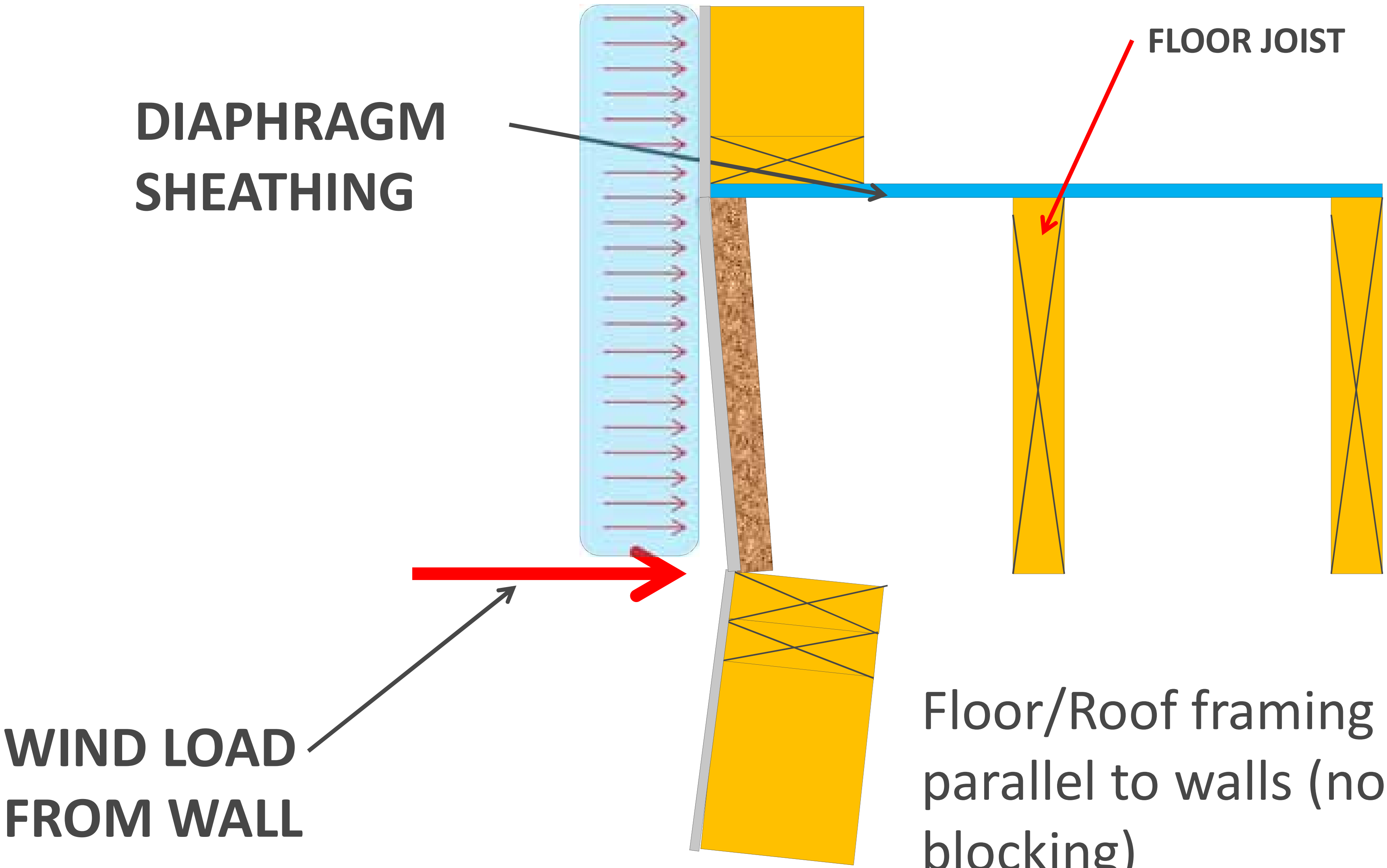
Floor/Roof framing
perpendicular to walls

Stud to Diaphragm



Floor/Roof framing parallel to walls (add blocking)

Stud to Diaphragm



Floor/Roof framing parallel to walls (no blocking)

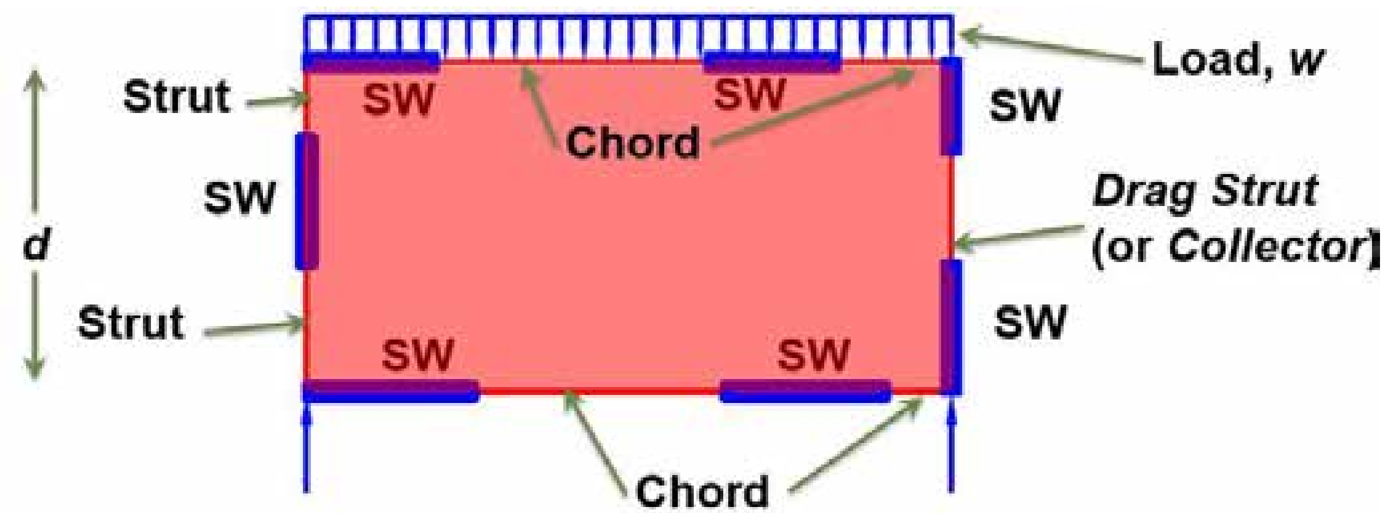
WSP Diaphragm Capacity

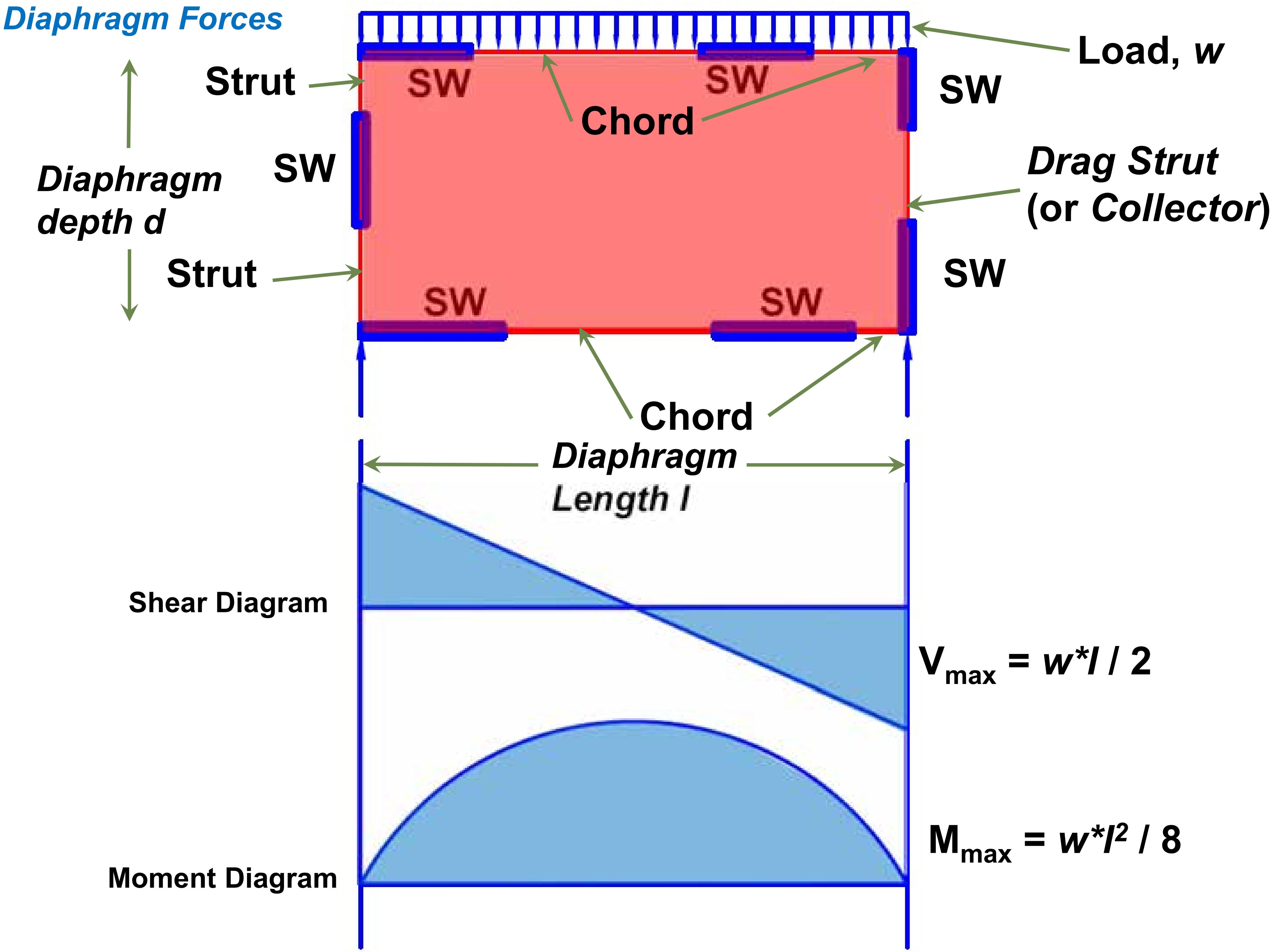
- Capacities listed in AWC's Special Design Provisions for Wind and Seismic (SDPWS)
- WSP diaphragms most common. Can also use single-layer horizontal and diagonal lumber sheathing, and double-layer diagonal sheathing.
- Note that capacities are given as nominal: must be adjusted by a reduction or resistance factor to determine allowable unit shear capacity (ASD) or factored unit shear resistance (LRFD)

Diaphragm Boundary

All edges of a diaphragm shall be supported by a boundary element. (SDPWS 4.1.4; 4.2.6.1)

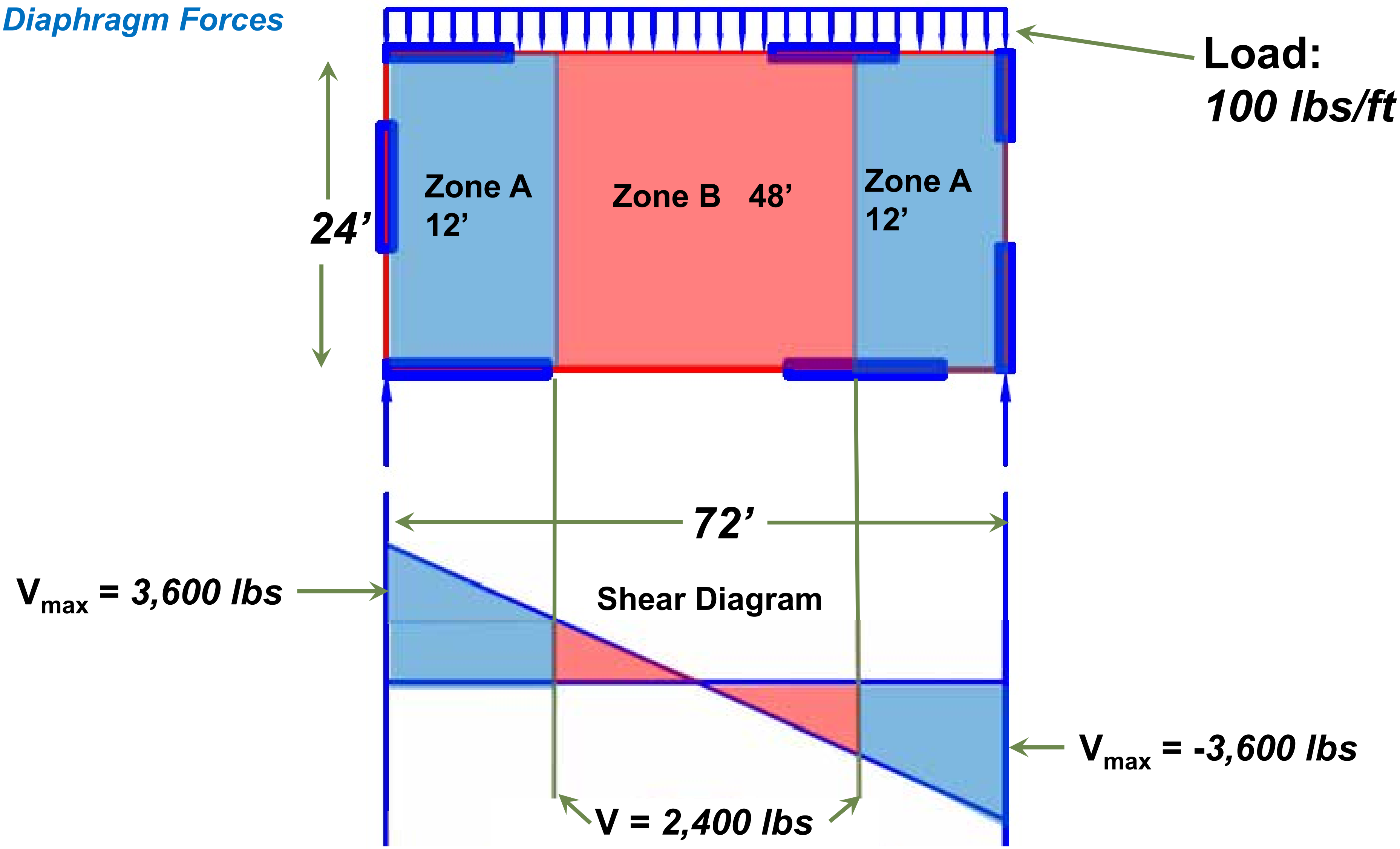
- **<u>Diaphragm Boundary Elements:</u>**
  - **Chords, drag struts, collectors, Shear walls, frames**
  - **Boundary member locations:**
    - **Diaphragm and shear wall perimeters**
    - **Interior openings**
    - **Areas of discontinuity**
    - **Re-entrant corners.**





Diaphragm Forces
Load, w
Strut
SW
SW
Chord
SW
Drag Strut
(or Collector)
Diaphragm
depth d
SW
Strut
SW
SW
SW
Chord
Diaphragm
Length l
Shear Diagram
$V_{max} = w*l / 2$
Moment Diagram
$M_{max} = w*l^2 / 8$

Diaphragm Forces



Diaphragm Fastener Schedule

- *Zone A: Nailing Pattern 1*
- *Zone B: Nailing Pattern 2*

Diaphragm Capacities in AWC SDPWS

- Capacities in SDPWS are **Nominal** values. Not ASD

Divide Nominal Values by 2.0 for ASD Capacity

Multiply Nominal Values by 0.8 for LRFD Capacity

- Capacity is reduced for species with Specific Gravity < 0.5
- For Spruce Pine Fir multiply by 0.92

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)
Structural I	6d	1-1/4	5/16	2
				3
	8d	1-3/8	3/8	2
				3
	10d	1-1/2	15/32	2
				3

Blocked

SDPWS Table 4.2A

B WIND			
Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
6	4	2-1/2	2
Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
6	6	4	3
v_w (plf)	v_w (plf)	v_w (plf)	v_w (plf)
520	700	1050	1175
590	785	1175	1330
755	1010	1485	1680
840	1120	1680	1890
895	1190	1790	2045
1010	1345	2015	2295

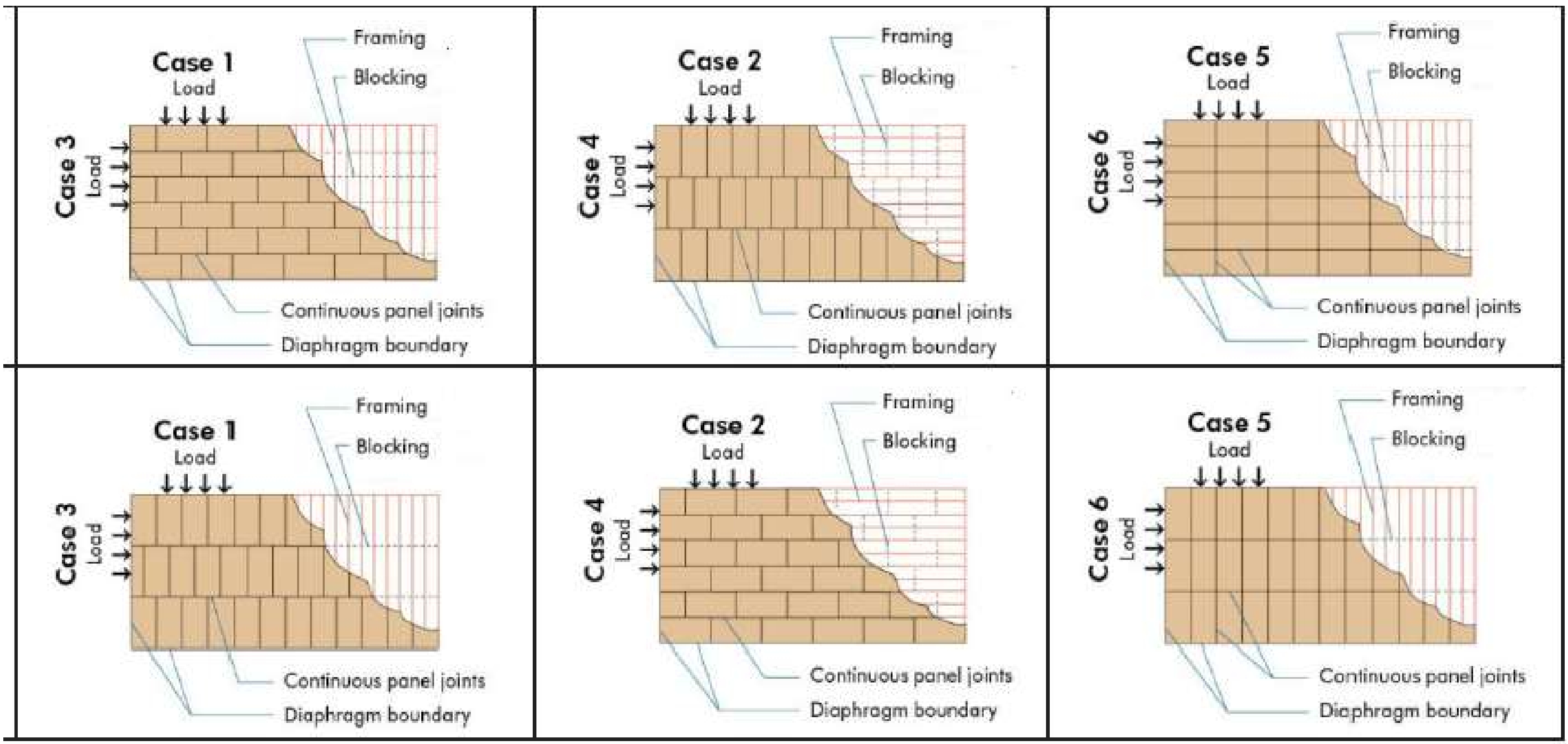
Unblocked

SDPWS Table 4.2C

B WIND	
6 in. Nail Spacing at diaphragm boundaries and supported panel edges	
Case 1	Cases 2,3,4,5,6
v_w (plf)	v_w (plf)
460	350
520	390
670	505
740	560
800	600
895	670

Diaphragm Types

SDPWS Tables 4.2A & B



Example: Retail Restaurant

- Assume Basic Wind Speed = 115 mph Ultimate; Exposure B
- Spruce Pine-Fir
- 10’ wall height

Diaphragm Design

- Capacity

Shearwall Design

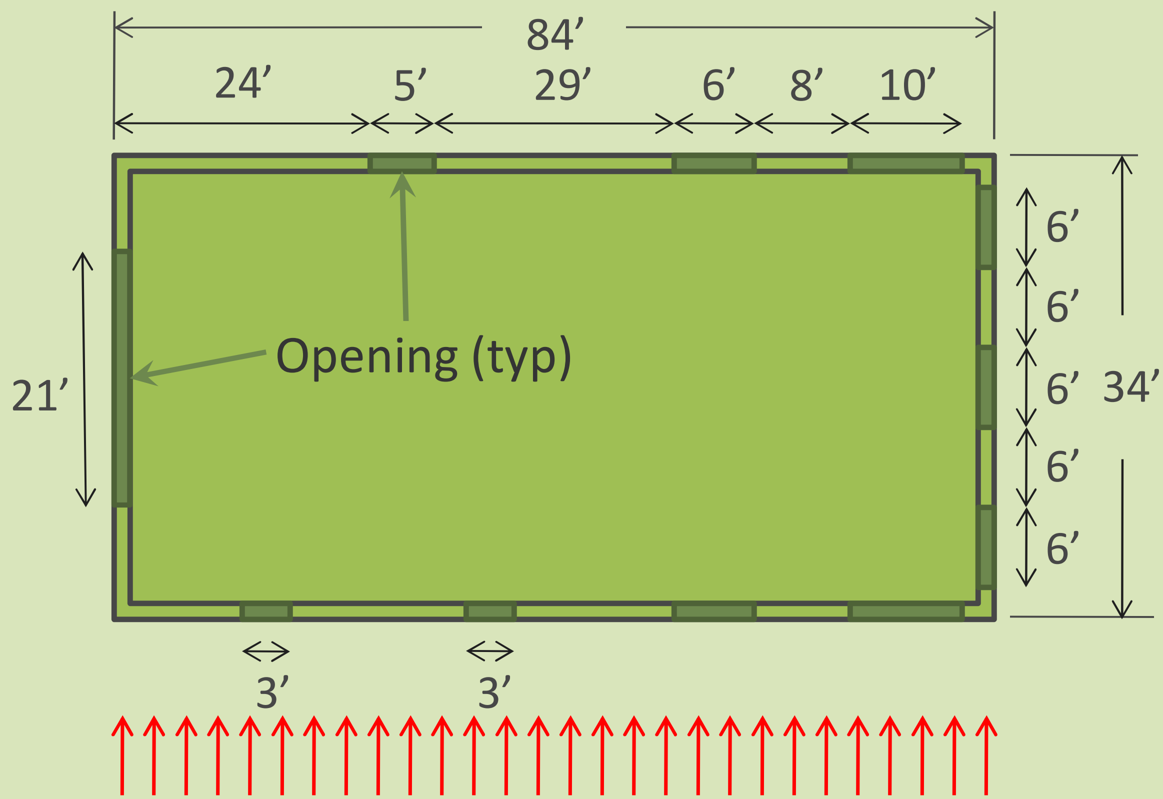
(SDPWS 4.3.5)

- Conventional
- Force Transfer Around Opening
- Perforated Shearwall

Retail Restaurant – Diaphragm Design

Critical Shearwall at front of building

Check Diaphragm for wind loads on 84’ wall



Diaphragm Aspect Ratios

SDPWS TABLE 4.2.4

TYPE - MAXIMUM LENGTH/WIDTH RATIO

Wood structural panel, unblocked	3:1
Wood structural panel, blocked	4:1
Single-layer straight lumber sheathing	2:1
Single-layer diagonal lumber sheathing	3:1
Double-layer diagonal lumber sheathing	4:1

For an 84 x 34 diaphragm the aspect ratio is 2.5 < 3.

Diaphragm aspect ratio is OK.

Calculating MWFRS Wind Loads

Calculate wind pressure using Directional Method (ASCE 7 Chpt 27)

$q_h = 0.00256K_zK_{zt}K_dV^2$ (ASCE 27.3-1)

$q_h = 0.00256*0.57*1.0*0.85*115^2 = 16.4$ psf

$p = q_h[(GC_{pf})-(GC_{pi})]$

$GC_{pf} = 0.85*[0.8 – (-0.3)] = 0.935$

$GC_{pi} = 0.18 - 0.18 = 0$

p = (16.4 psf)(0.935) = 15.34psf

0.6*W = 0.6*15.34 = 9.2 psf on walls

(0.6 from ASD Load Combo. See ASCE 7: 2.4.1)

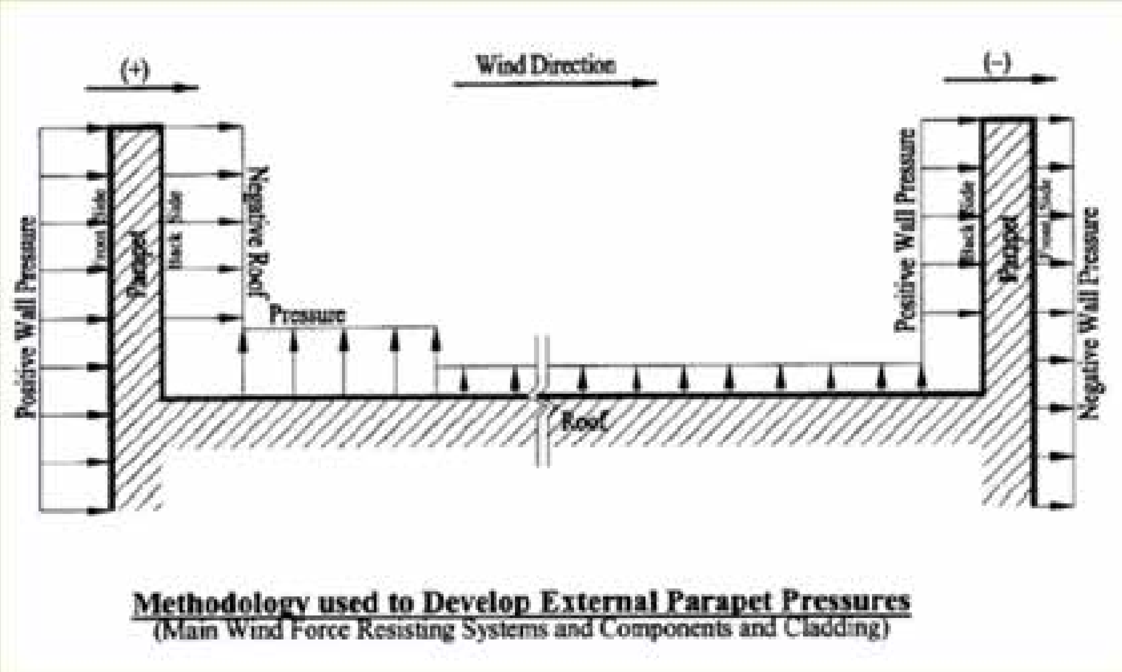
Use min 9.6 psf per ASCE 27.1.5

Wall Pressure Coefficients, C_p

Surface	L/B	C_p
Windward Wall	All values	0.8
Leeward Wall	0-1	-0.5
	2	-0.3
	≥4	-0.2

ASCE 7-10 Figure 27.4-1

Parapet Design – Figure 27.6-2



At parapets windward and leeward pressures occur on each parapet.

Section 27.4.5: $p_p = q(GC_{pn})$

GC_{pn} = 1.5 Windward parapet, -1.0 Leeward parapet

Windward Parapet GC_{pf} is 1.5: 16.4*1.5*0.6 = 14.76 psf

Leeward Parapet GC_{pf} is 1.0: 16.4*1.0*0.6 = 9.84 psf

Net Parapet = 14.76 + 9.84 = <u>24.6 psf</u>

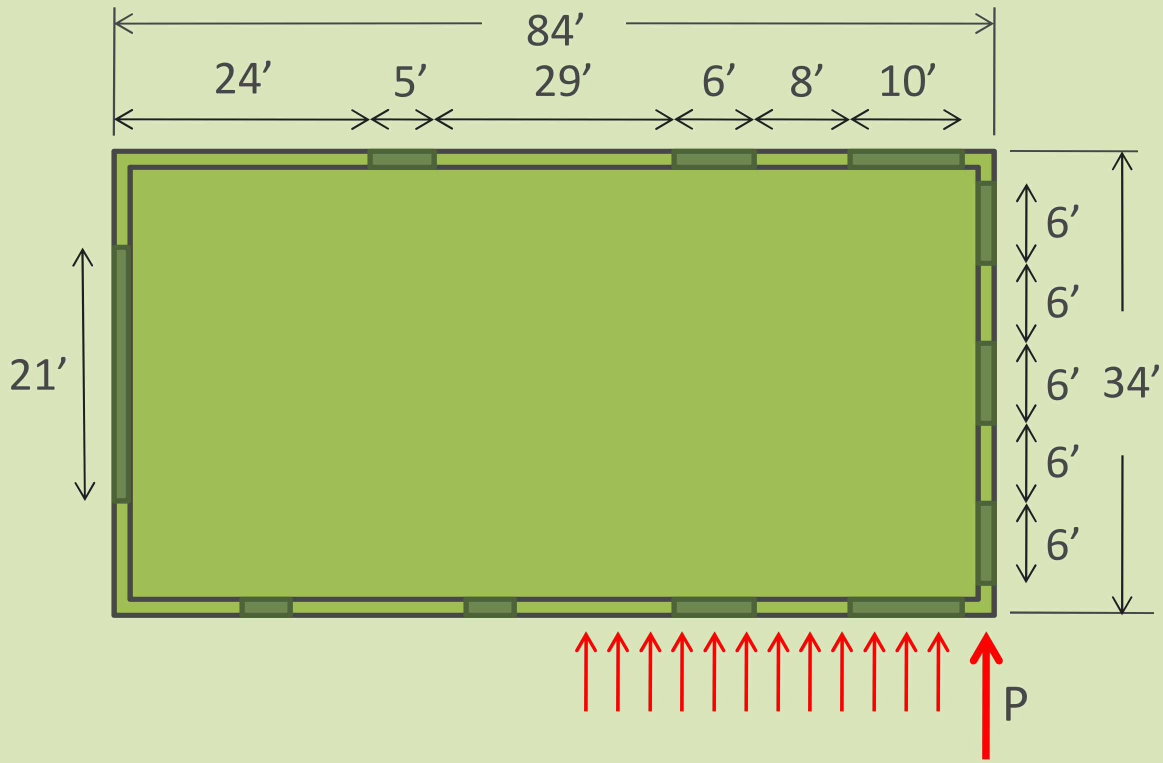
Retail Restaurant – Diaphragm Design

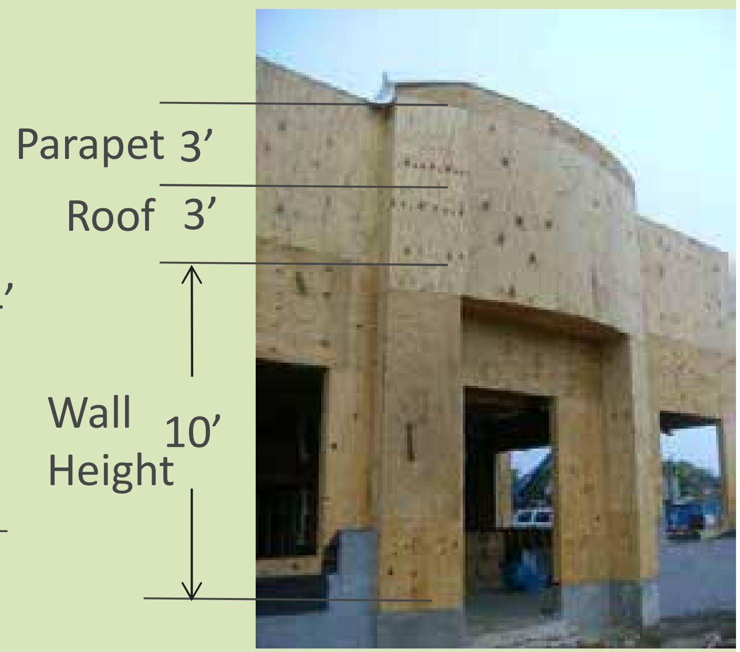
W = (9.6psf*(5'+3')+(24.6)*3') = 150.6 plf

V = (150.6 plf)*(84'/2) = 6,325 lb

M = (150.6 plf)*(84'^{2})/8 = 132,829 lb*ft

T = C = (132,829 lb*ft)/(34 ft) = 3,907 lb





$v_{diaphragm}$ = 6,325 lb/34' = 186 plf

Diaphragm Capacity: SDPWS Table 4.2C

Table 4.2C Nominal Unit Shear Capacities for Wood-Frame Diaphragms

Unblocked Wood Structural Panel Diaphragms[1,2,3,4,5]

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Supported Edges and Boundaries (in.)	A SEISMIC: 6 in. Nail Spacing at diaphragm boundaries and supported panel edges						B WIND: 6 in. Nail Spacing at diaphragm boundaries and supported panel edges	
					Case 1			Cases 2,3,4,5,6			Case 1	Cases 2,3,4,5,6
					v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_s (plf)	G_a (kips/in.) OSB	G_a (kips/in.) PLY	v_w (plf)	v_w (plf)
Structural I	6d	1-1/4	5/16	2	330	9.0	7.0	250	6.0	4.5	460	350
				3	370	7.0	6.0	280	4.5	4.0	520	390
	8d	1-3/8	3/8	2	480	8.5	7.0	360	6.0	4.5	670	505
				3	530	7.5	6.0	400	5.0	4.0	740	560
	10d	1-1/2	15/32	2	570	14	10	430	9.5	7.0	800	600
				3	640	12	9.0	480	8.0	6.0	895	670
Sheathing and Single-Floor	6d	1-1/4	5/16	2	300	9.0	6.5	220	6.0	4.0	420	310
				3	340	7.0	5.5	250	5.0	3.5	475	350
			3/8	2	330	7.5	5.5	250	5.0	4.0	460	350
				3	370	6.0	4.5	280	4.0	3.0	520	390
	8d	1-3/8	3/8	2	430	9.0	6.5	320	6.0	4.5	600	450
				3	480	7.5	5.5	360	5.0	3.5	670	505
			7/16	2	460	8.5	6.0	340	5.5	4.0	645	475
				3	510	7.0	5.5	380	4.5	3.5	715	530
			15/32	2	480	7.5	5.5	360	5.0	4.0	670	505
				3	530	6.5	5.0	400	4.0	3.5	740	560
	10d	1-1/2	15/32	2	510	15	9.0	380	10	6.0	715	530
				3	580	12	8.0	430	8.0	5.5	810	600
			19/32	2	570	13	8.5	430	8.5	5.5	800	600
				3	640	10	7.5	480	7.0	5.0	895	670

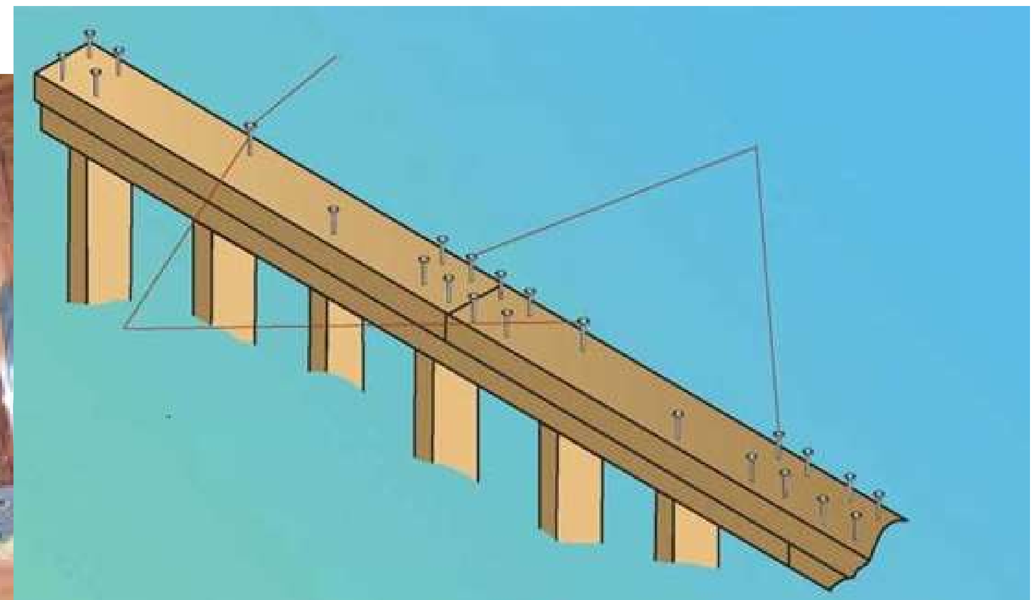
Capacity is reduced for species with Specific Gravity < 0.5.
For Spruce Pine Fir multiply by 0.92

Capacity = (645 plf)(0.92)/2 = 297 plf

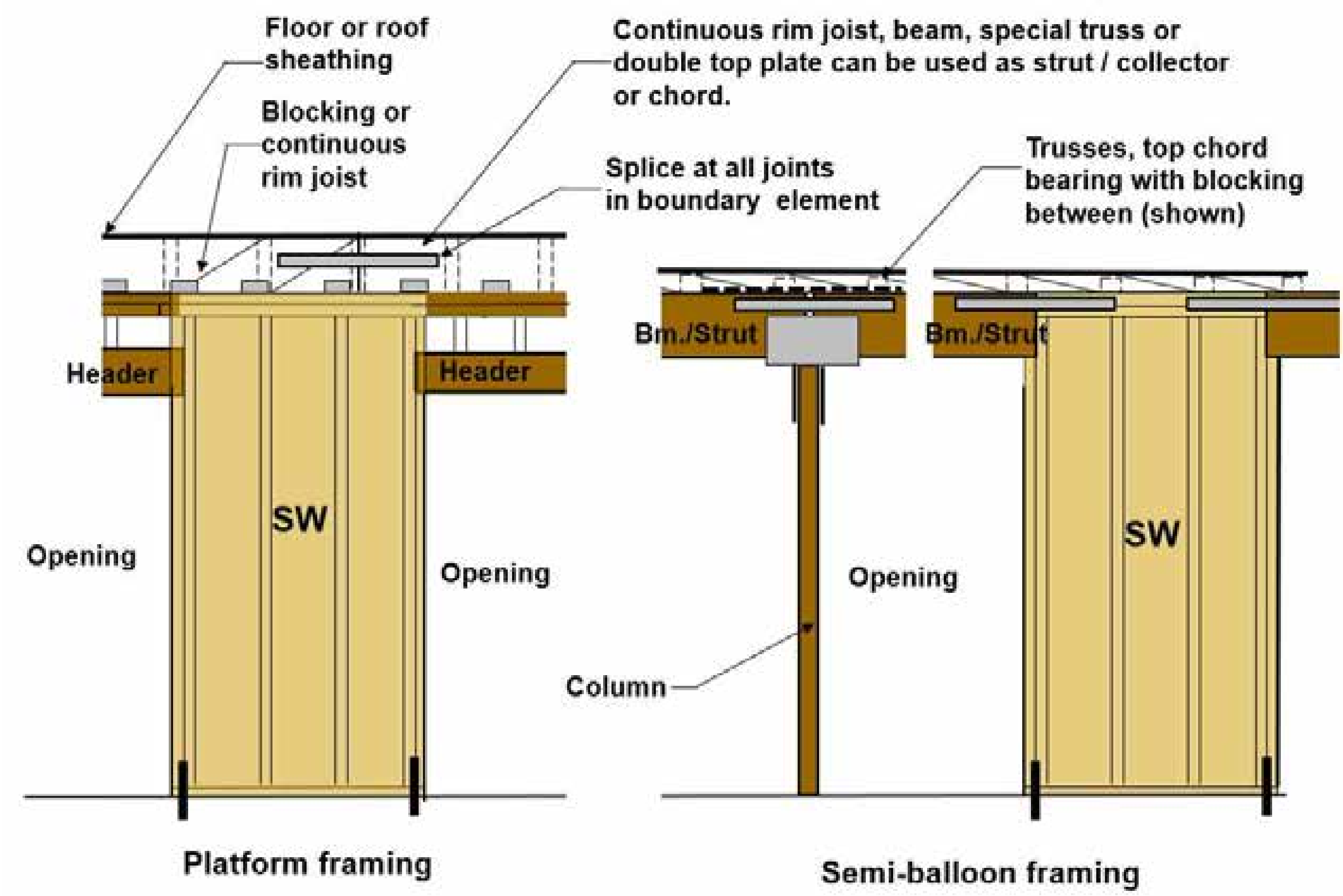
297 plf > 186 plf, diaphragm is adequate with sheathing & fastening as shown above

Diaphragm Chords

Wall Top Plates Typically Function as Both Diaphragm Chords and Drag Struts/Collectors

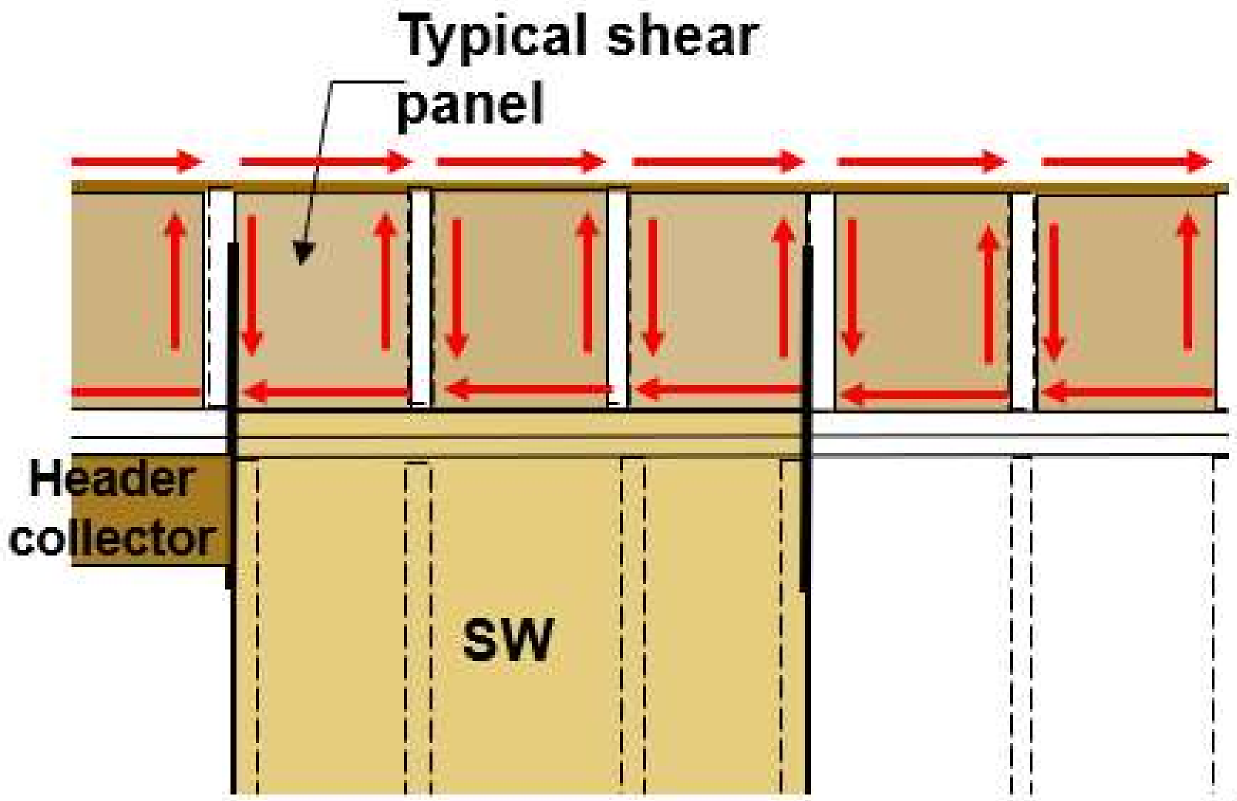


Load Path



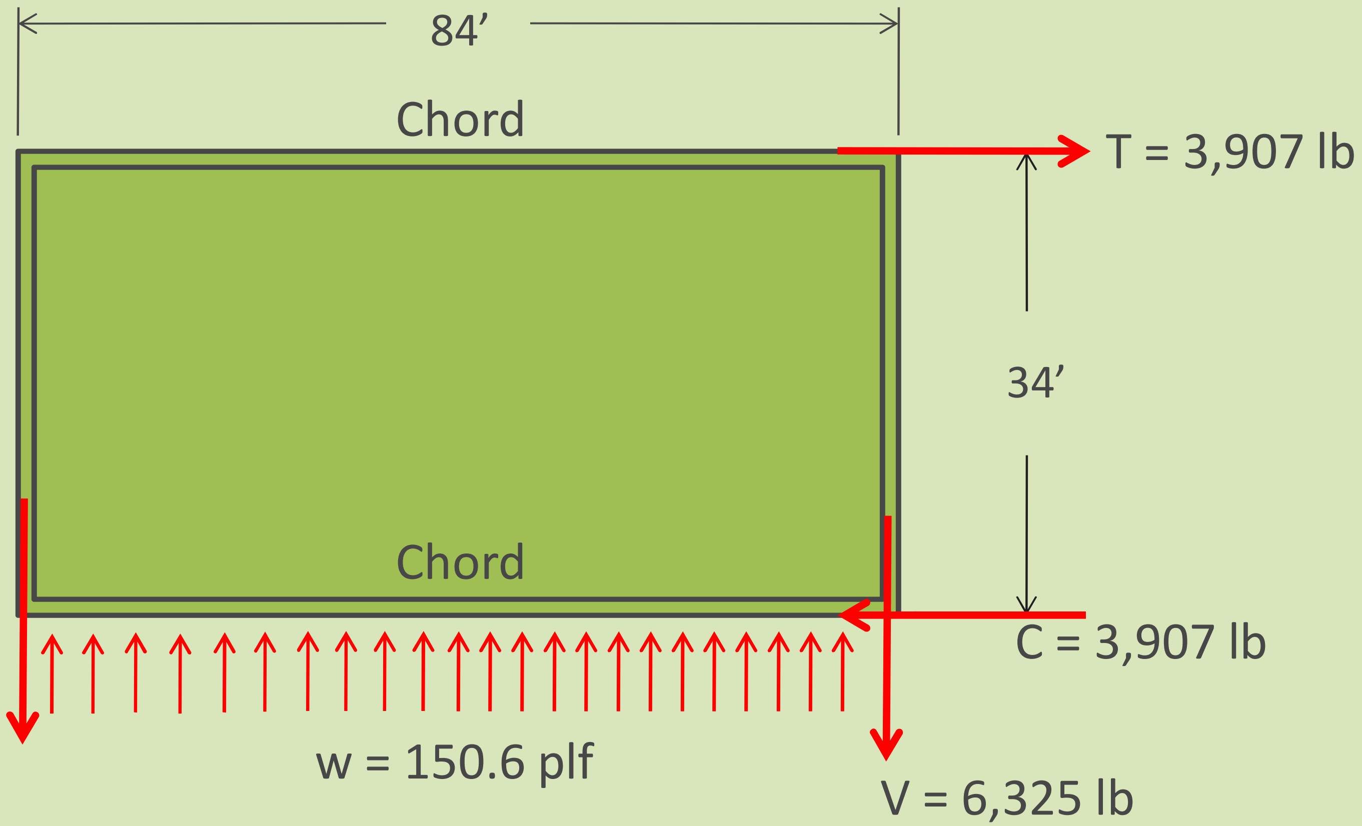
Floor or roof sheathing
Continuous rim joist, beam, special truss or double top plate can be used as strut / collector or chord.
Blocking or continuous rim joist
Splice at all joints in boundary element
Trusses, top chord bearing with blocking between (shown)
Header
Header
Bm./Strut
Bm./Strut
SW
SW
Opening
Opening
Opening
Column
Platform framing
Semi-balloon framing

Load Path



Diaphragm Design – Chords

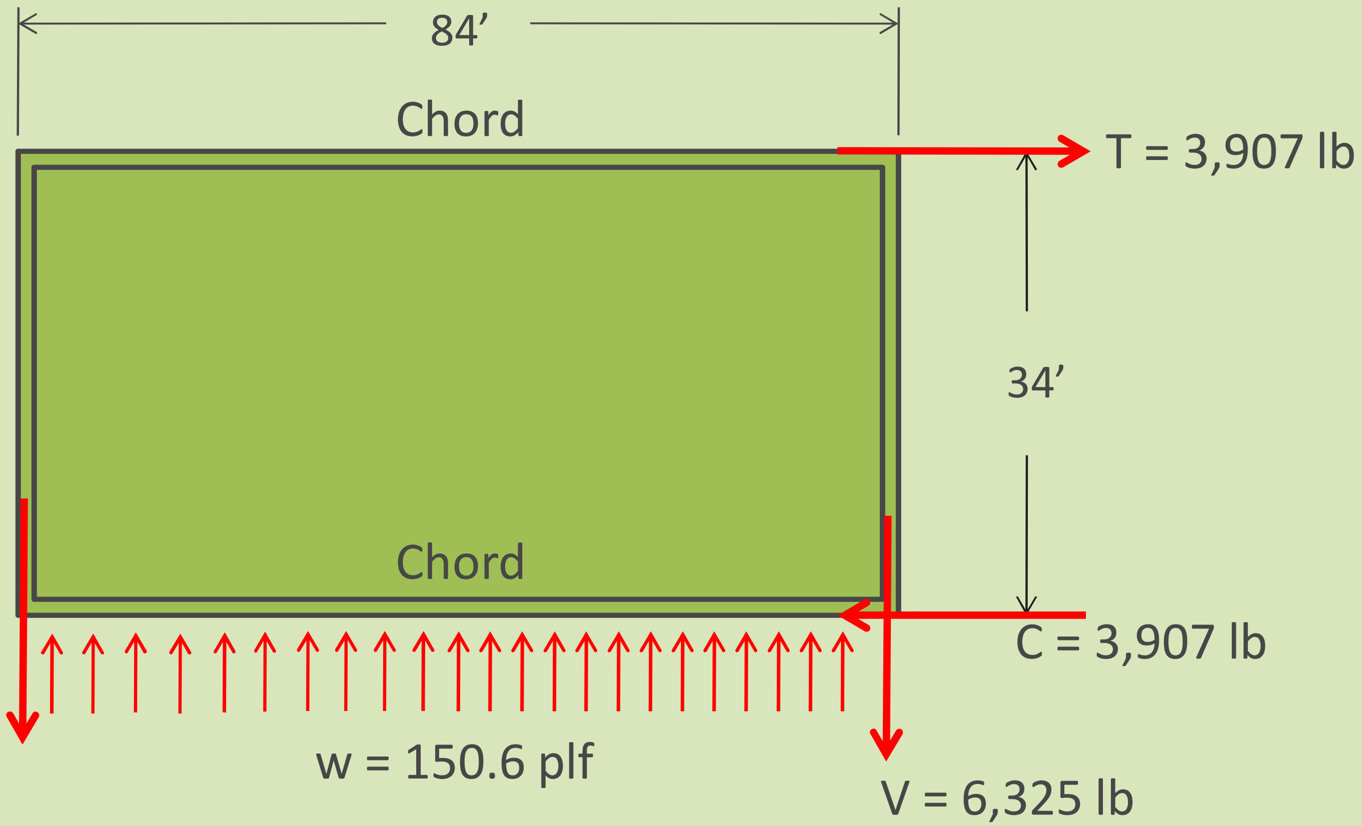
- (2) 2x6 Spruce Pine Fir #2 top plates
- F_t = 450 psi (Tension Parallel to Grain – NDS Supplement Table 4A)
- F_c = 1,150 psi (Compression Parallel to Grain – NDS DVWC Table 4A)
- ***** F_t = 450 psi will control *****



Diaphragm Design – Chords

- T = C = 3,907 lbs
- $F'_t = F_t C_d C_M C_t C_F C_i$
- F'_t = (450 psi) 1.6 = 720 psi
- f_t = T/A = 3,907/(1.5"x5.5") = 474 psi < 720 psi *chord ok*

- *Note only 1 top plate required for chord force*



Diaphragm Design – Deflection

From SDPWS commentary:

The total mid-span deflection of a blocked, uniformly nailed (e.g. same panel edge nailing) wood structural panel diaphragm can be calculated by summing the effects of four sources of deflection:

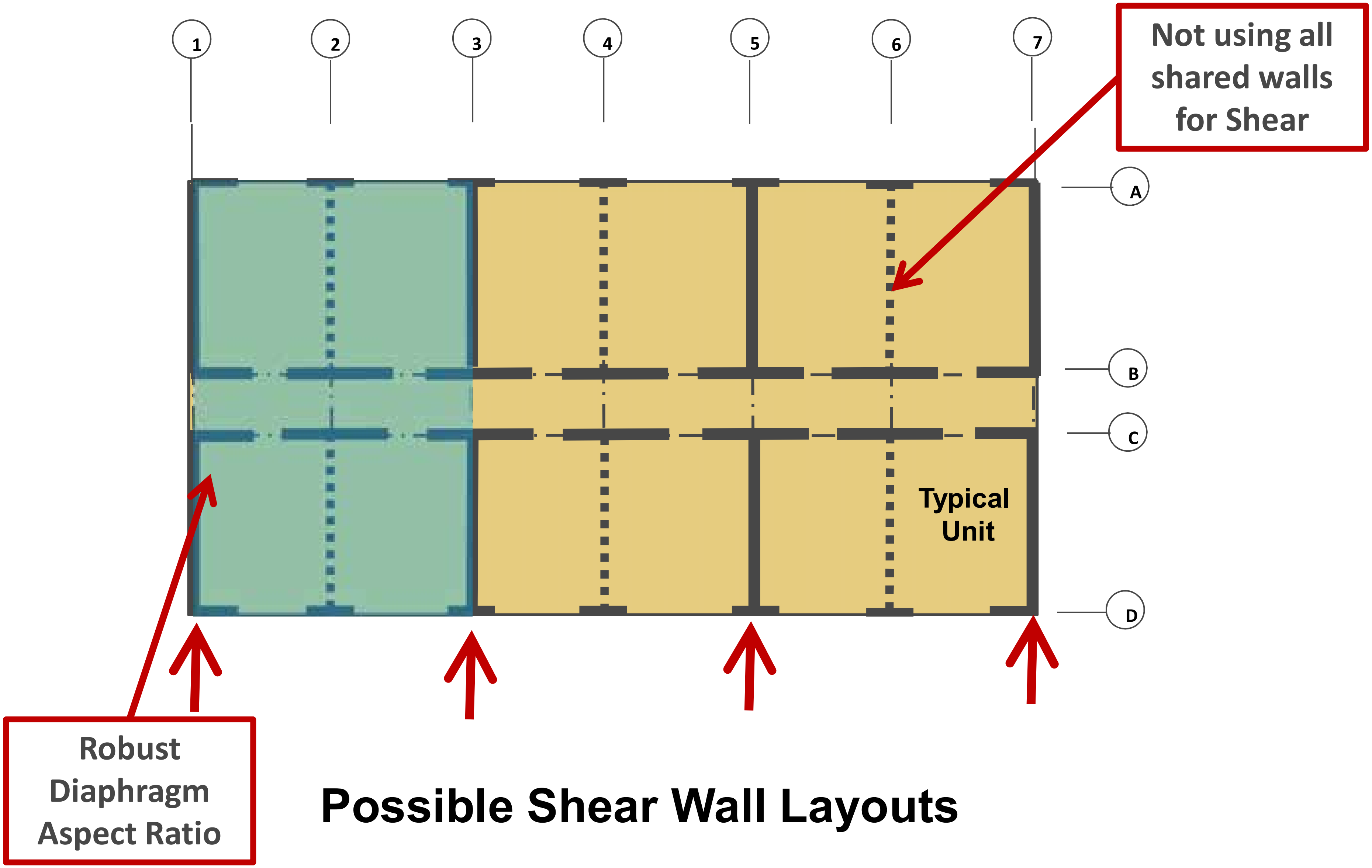
- *Framing bending deflection*
- *Panel shear deflection*
- *Deflection from nail slip*
- *Deflection due to chord splice slip*

SDPWS equation C4.2.2-1:

(bending, chord deformation excluding slip) *(shear, panel deformation)* *(shear, panel nail slip)* *(bending, chord splice slip)*

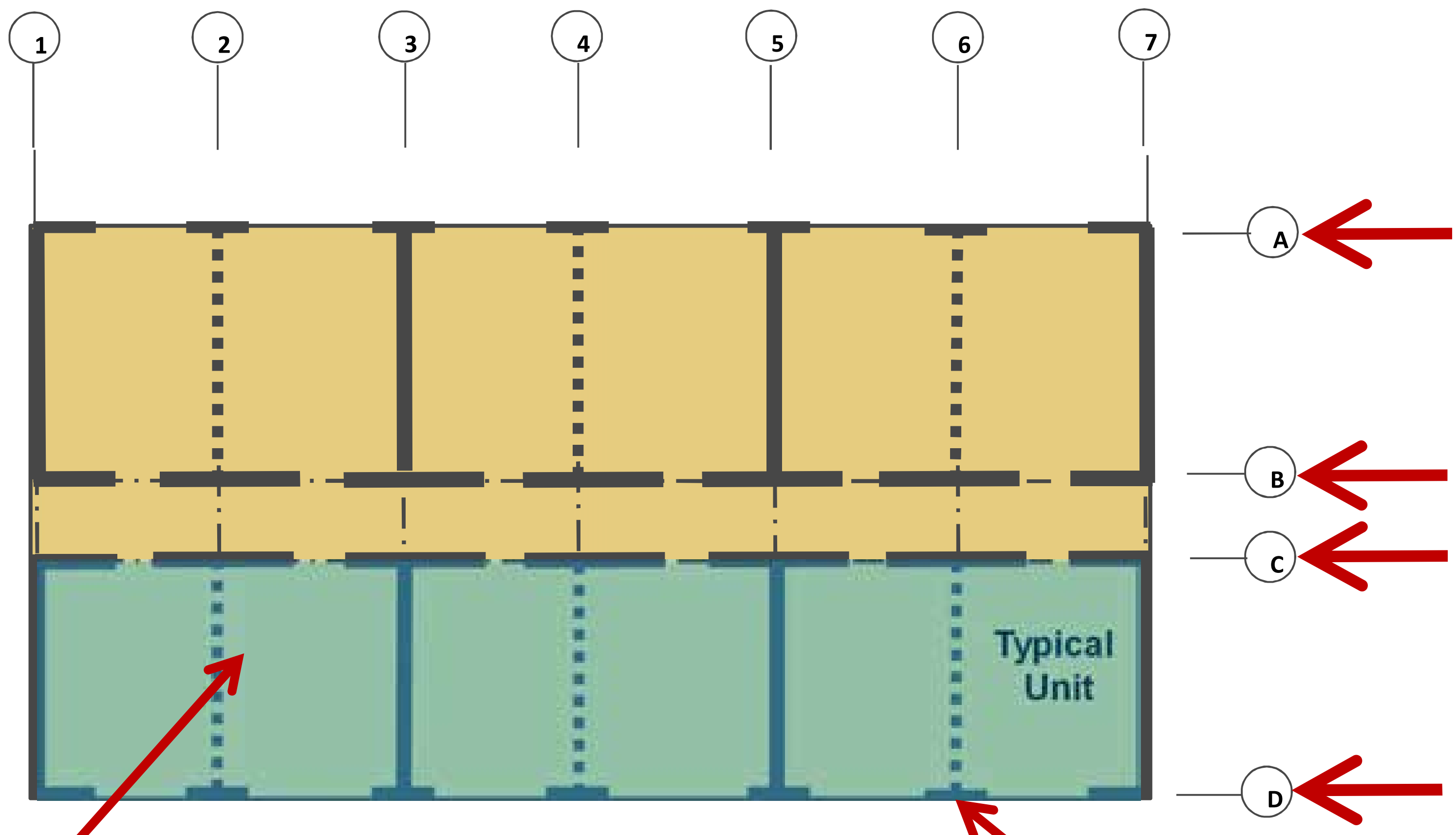
$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{vL}{4G_v t_v} + 0.188Le_n + \frac{\sum(x\Delta_c)}{2W} \quad \text{(C4.2.2-1)}$$

Diaphragm Modeling Methods



Possible Shear Wall Layouts

Diaphragm Modeling Methods



Robust Diaphragm Aspect Ratio

Possible Shear Wall Layouts

But maybe not much wall available on exterior

Rigid or Flexible Diaphragm?

Light Frame Wood Diaphragms often default to Flexible Diaphragms

Code Basis: ASCE 7-10 26.2 Definitions (Wind)

Diaphragms constructed of wood structural panels are permitted to be idealized as flexible

Code Basis: ASCE 7-10 12.3.1.1 (Seismic)

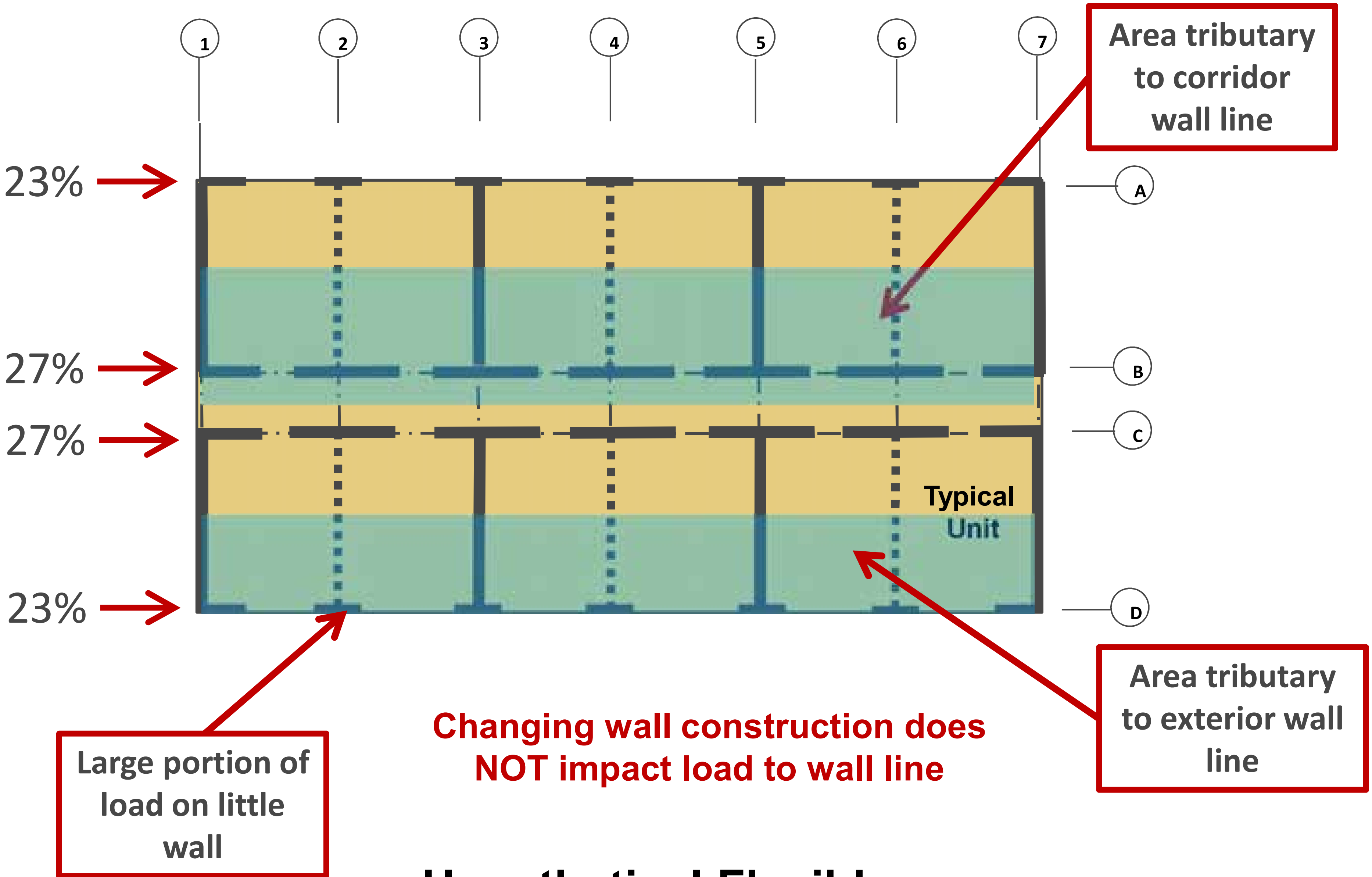
Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

[…]

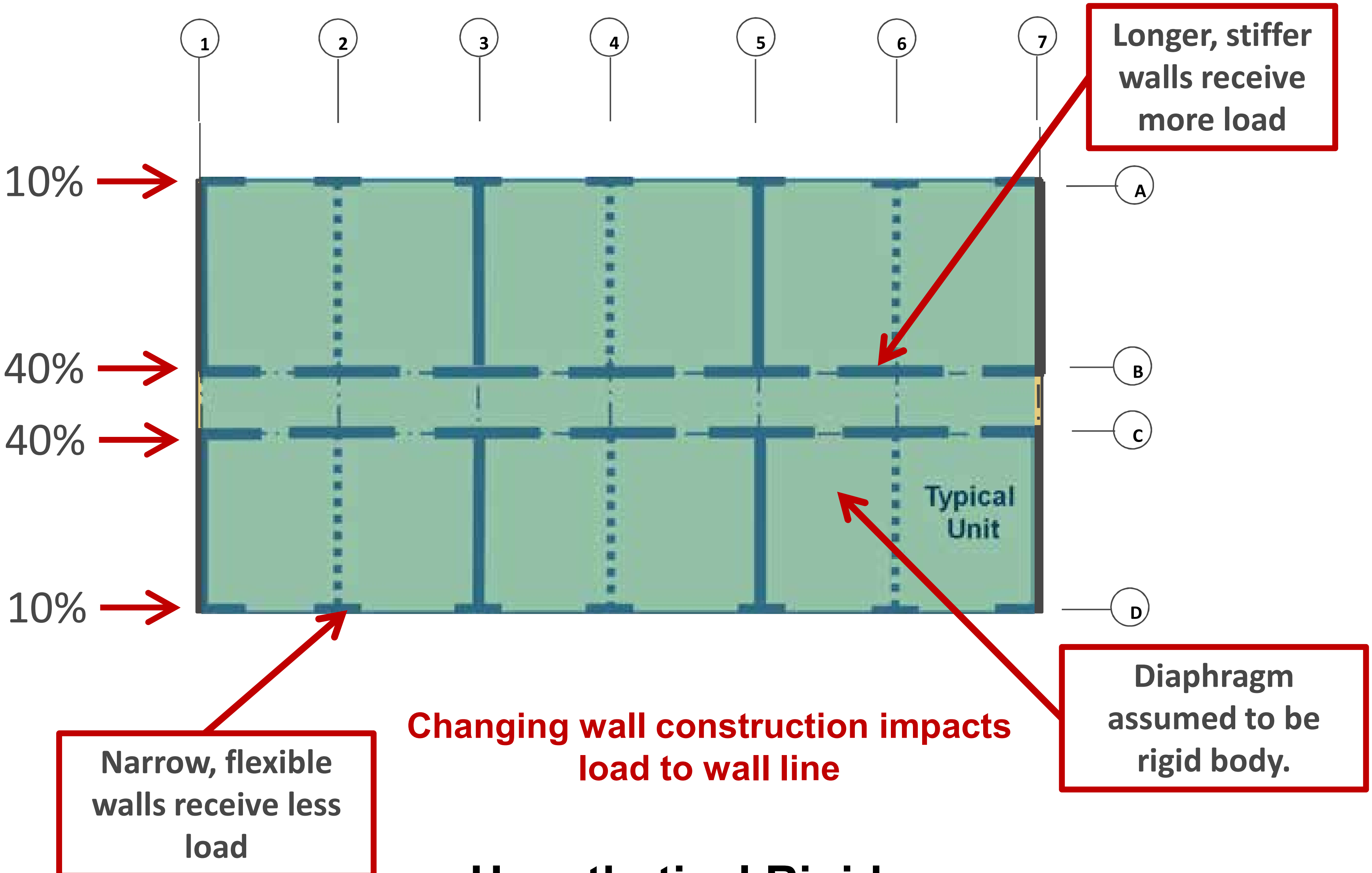
c. In structures of light-frame construction where all of the following conditions are met:

1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 1/2 in. thick.

2. Each line of vertical elements of the seismic force resisting system complies with the allowable story drift of Table 12.12-1..



Hypothetical Flexible Diaphragm Distribution



Hypothetical Rigid Diaphragm Distribution

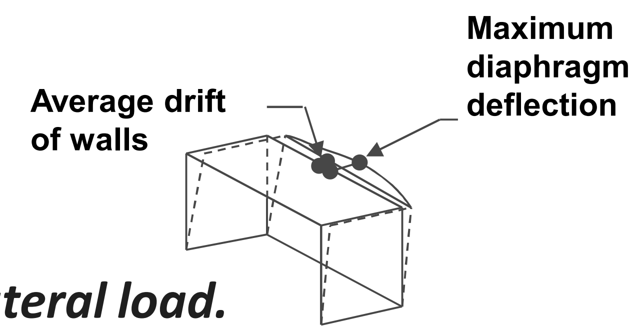
Can a Rigid Diaphragm be Justified?

ASCE 7-10 12.3.1.3 (Seismic)

*[Diaphragms] are permitted to be idealized as **flexible** where the computed maximum **in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements** of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1.*

2015 SDPWS 4.2.5 – Rigid diaphragm

*It shall be permitted to idealize a diaphragm as **rigid** when the computed maximum in-plane deflection of the diaphragm itself under lateral load **is less than or equal to two times the average deflection of adjoining vertical elements of the lateral force-resisting system of the associated story under equivalent tributary lateral load.***



Rigid Diaphragm Analysis

Some Advantages of Rigid Diaphragm

- More load (plf) to longer interior/corridor walls
- Less load (plf) to narrow walls where overturning restraint is tougher
- Can tune loads to walls and wall lines by changing stiffness of walls

Some Disadvantages of Rigid Diaphragm

- Considerations of torsional loading necessary
- More complicated calculations to distribute load to shear walls
- May underestimate "Real" loads to narrow exterior walls
- Justification of rigid assumption

Two More Diaphragm Approaches

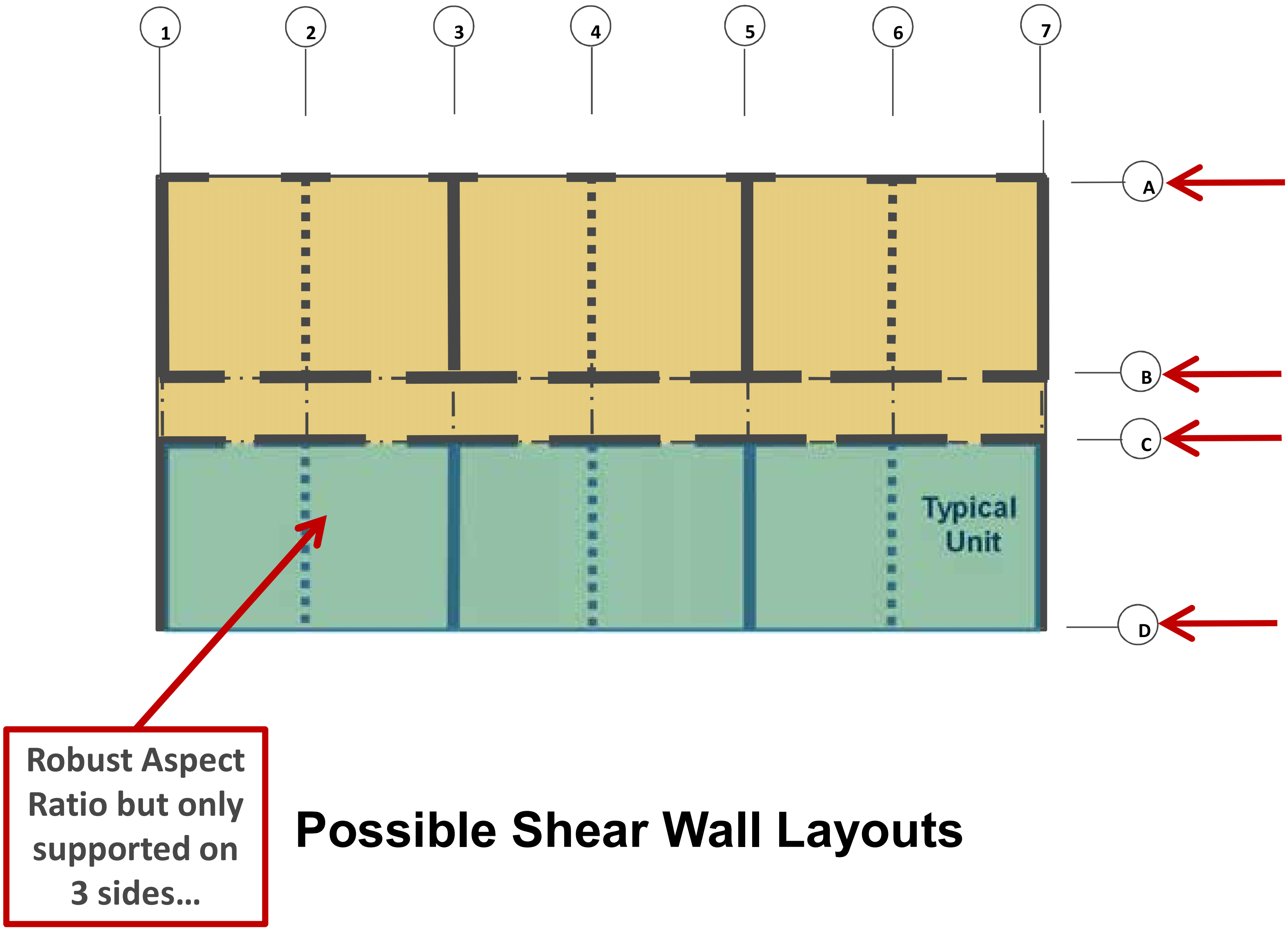
Semi-Rigid Diaphragm Analysis

- Neither idealized flexible nor idealized rigid
- Explicit modeling of diaphragm deformations with shear wall deformations to distribute lateral loads
- Not easy

Enveloping Method

- Idealized as BOTH flexible and rigid.
- Individual components designed for worst case from each approach
- Been around a while, officially recognized in the 2015 SDPWS

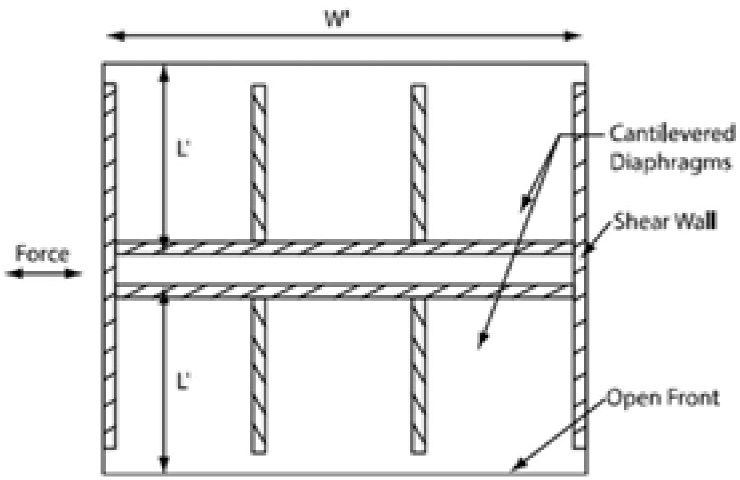
No Shear Walls at Exterior



Possible Shear Wall Layouts

Open Front Structure & Cantilevered Diaphragms in SDPWS 2015

Cantilevered Diaphragm



SDPWS 4.2.5.2

L'/W' ≤ **1.5**

When Torsionally Irregular

L'/W' ≤ 1, one story

≤ 2/3, multi-story

L' ≤ **35** ft

Provided diaphragms modelled as rigid or semi-rigid and for seismic, the story drift at each edge of the structure within allowable story drift of ASCE 7. Story drifts include torsion and accidental torsional loads and deformations of the diaphragm.

Small Openings in Diaphragms

Accounting for openings in shear panels (diaphragms and shear walls) is a code requirement (IBC 2305.1.1)

No code path for checking minimum size opening limit (other than prescriptive design – IBC 2308.4.4.1 & 2308.7.6.1)

Do you need to account for a

12” square opening in a diaphragm?

http://cwc.ca/wp-content/uploads/2013/11/Design-example-of-designing-for-openings-in-wood-diaphragm.pdf

Small Openings in Diaphragms

FPInnovations method for checking small holes in diaphragms:

Recommend running an analysis of the opening's effects on the diaphragm unless the following conditions are met.

3. It is strongly recommended that analysis for a diaphragm with an opening should be carried out except where <u>all four</u> of the following items are satisfied:

   a. Opening depth no greater than 15% of diaphragm depth;
   b. Opening length no greater than 15% of diaphragm length;
   c. Distance from diaphragm edge to the nearest opening edge is a minimum of 3 times the larger opening dimension; and
   d. The diaphragm portion between opening and diaphragm edge satisfies the maximum aspect ratio requirement.

Overview

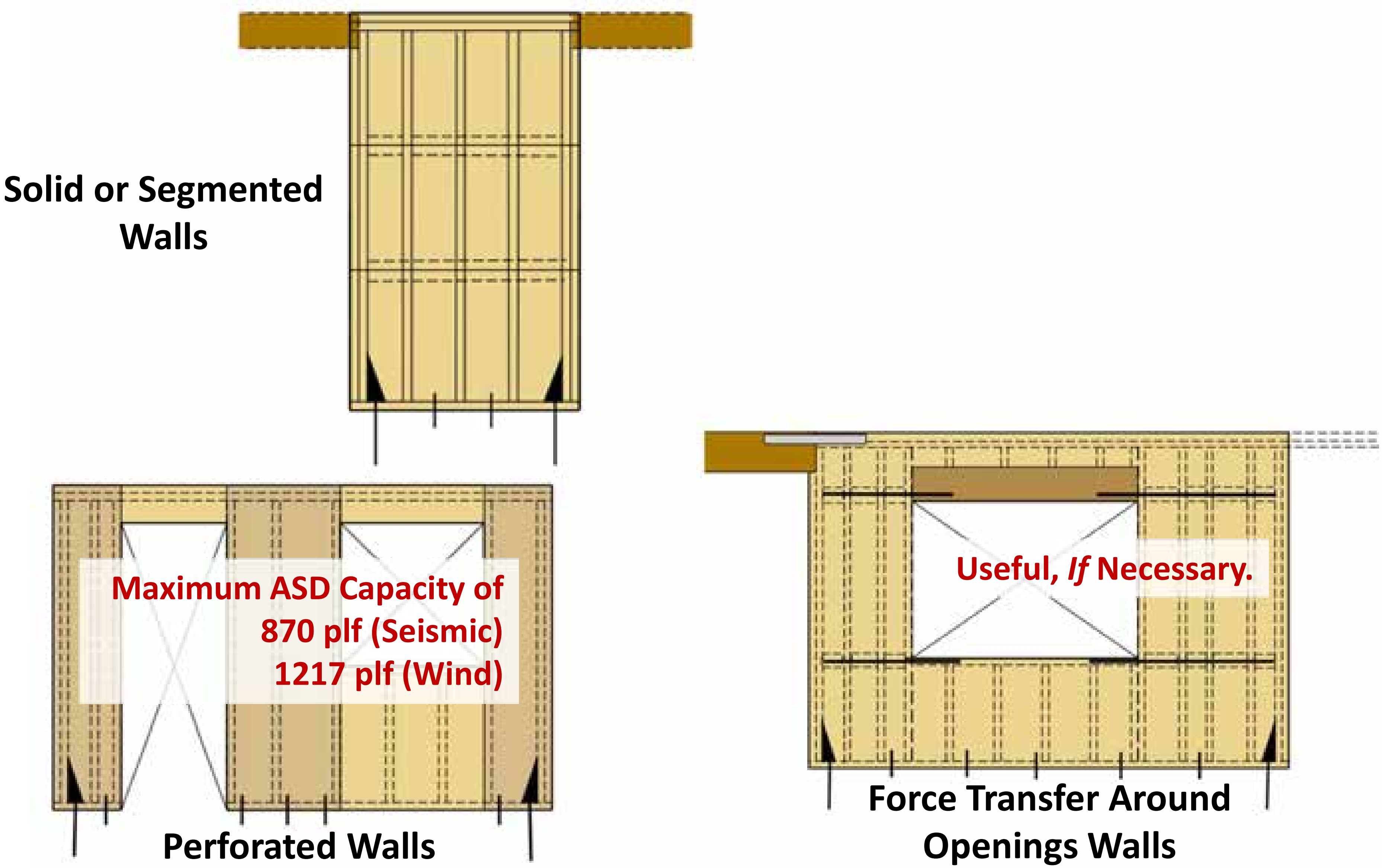
- Diaphragms
- Shear Walls

Shearwall Functions

Lateral Loads create shear (sliding) and racking forces on a structure

Sliding resisted by shearwall base anchorage
Racking resisted by shear panel & fasteners

Shear Wall Configuration Options



WSP Shear Wall Capacity

- Capacities listed in AWC's Special Design Provisions for Wind and Seismic (SDPWS)
- Sheathed shear walls most common. Can also use horizontal and diagonal board sheathing, gypsum panels, fiberboard, lath and plaster, and others
- Blocked shear walls most common. SDPWS has reduction factors for unblocked shear walls
- Note that capacities are given as nominal: must be adjusted by a reduction or resistance factor to determine allowable unit shear capacity (ASD) or factored unit shear resistance (LRFD)

Shear Wall Capacity - SDPWS Chpt 4

Table 4.3A Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1,3,6,7]

Wood-based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimun Fastener Penetration in Framing Member or Blocking (in.)	Fastener Type & Size	A SEISMIC – Panel Edge Fastener Spacing (in.) 6: v_s (plf)	6: G_a (kips/in.) OSB	6: G_a (kips/in.) PLY	4: v_s (plf)	4: G_a (kips/in.) OSB	4: G_a (kips/in.) PLY	3: v_s (plf)	3: G_a (kips/in.) OSB	3: G_a (kips/in.) PLY	2: v_s (plf)	2: G_a (kips/in.) OSB	2: G_a (kips/in.) PLY	B WIND – Panel Edge Fastener Spacing (in.) 6: v_w (plf)	4: v_w (plf)	3: v_w (plf)	2: v_w (plf)
Wood Structural Panels - Structural I[4,5]	5/16	1-1/4	Nail (common or galvanized box) 6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
Wood Structural Panels – Sheathing[4,5]																			
Plywood Siding																			
Particleboard Sheathing - (M-S "Exterior Glue" and M-2 "Exterior Glue")	3/8		Nail (common or galvanized box) 6d	240	15		360	17		460	19		600	22		335	505	645	840
	3/8		8d	260	18		380	20		480	21		630	23		365	530	670	880
	1/2			280	18		420	20		540	22		700	24		390	590	755	980
	1/2		10d	370	21		550	23		720	24		920	25		520	770	1010	1290
	5/8			400	21		610	23		790	24		1040	26		560	855	1105	1455
Structural Fiberboard Sheathing	1/2		Nail (galvanized roofing) 11 ga. galv. roofing nail (0.120" x 1-1/2" long x 7/16" head)				340	4.0		460	5.0		520	5.5			475	645	730
	25/32		11 ga. galv. roofing nail (0.120" x 1-3/4" long x 3/8" head)				340	4.0		460	5.0		520	5.5			475	645	730

Divide Nominal Values by 2.0 for ASD Capacity

Multiply Nominal Values by 0.8 for LRFD Capacity

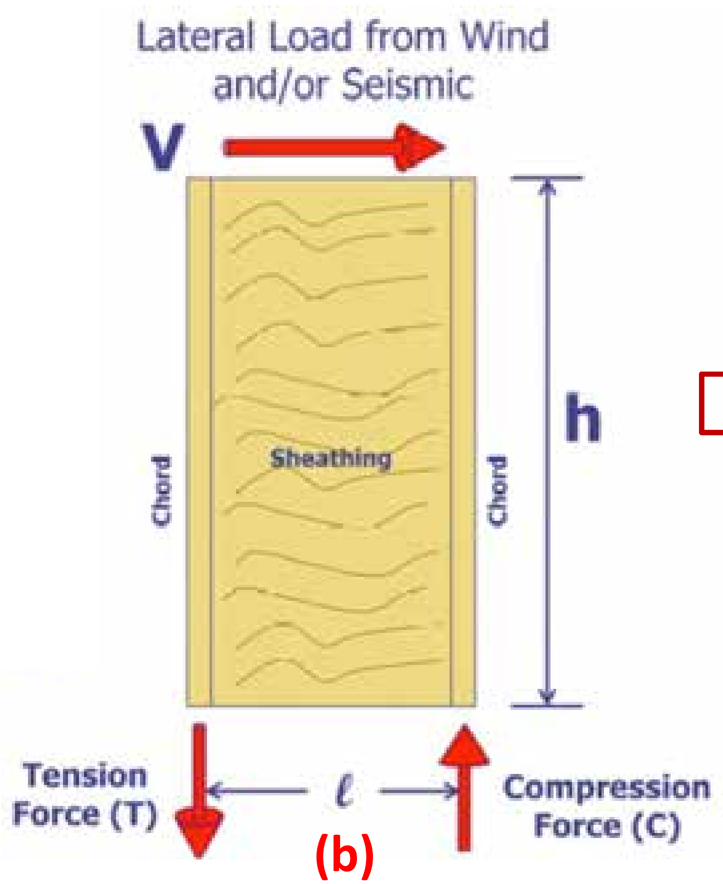
Shear Wall Capacity - SDPWS Chpt 4

Capacity based on blocked shear wall.
Reduce capacities for unblocked

Table 4.3.3.2 Unblocked Shear Wall Adjustment Factor, C_{ub}

Nail Spacing (in.)		Stud Spacing (in.)			
Supported Edges	Intermediate Framing	12	16	20	24
6	6	1.0	0.8	0.6	0.5
6	12	0.8	0.6	0.5	0.4

Shear Wall Requirements in AWC SDPWS



Wood Education Institute

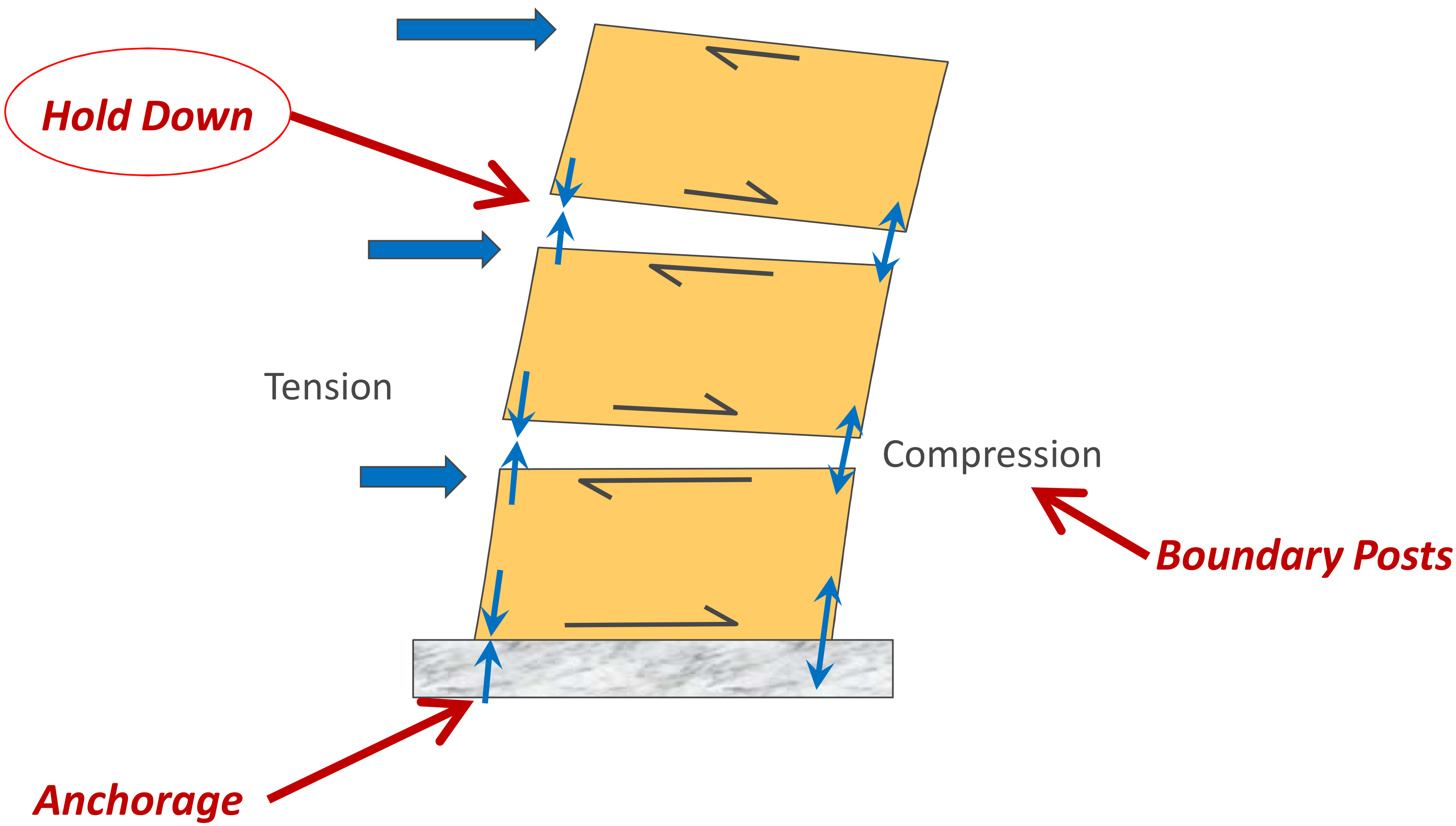
Table 4.3.4 Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type	Maximum h/b_s Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1[1]
Particleboard, blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum wallboard	2:1[2]
Portland cement plaster	2:1[2]
Structural Fiberboard	3.5:1[3]

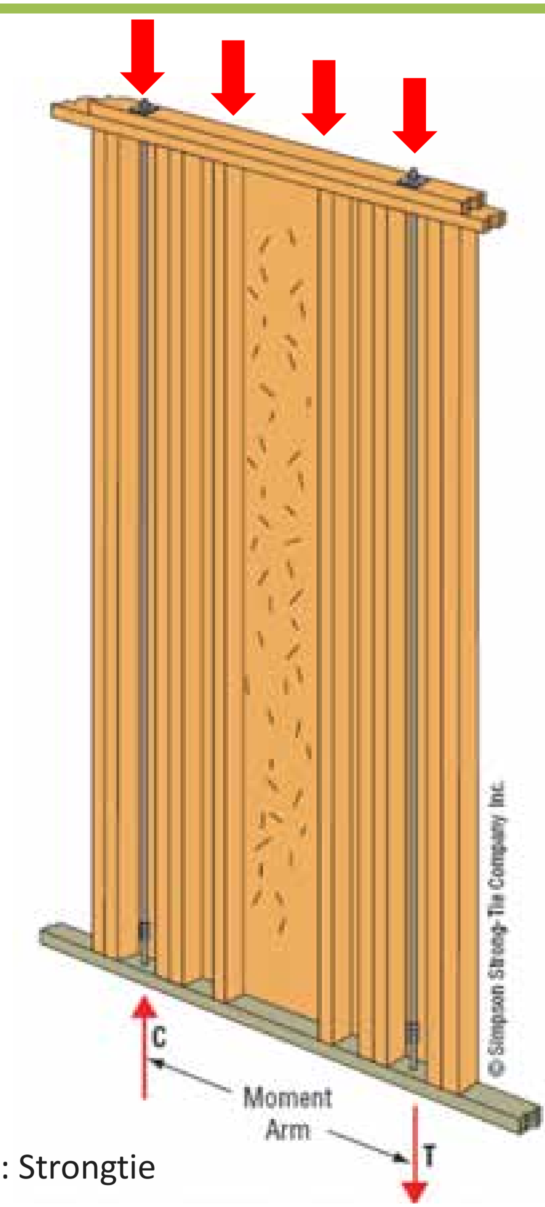
3:5:1 max aspect ratio for blocked Wood Structural Panel Shear Wall. Reduction in Capacity when greater than 2:1

Aspect Ratio = h/b

Components of Shear Wall Design



Using Dead Load to Resist Overturning



Source: Strongtie

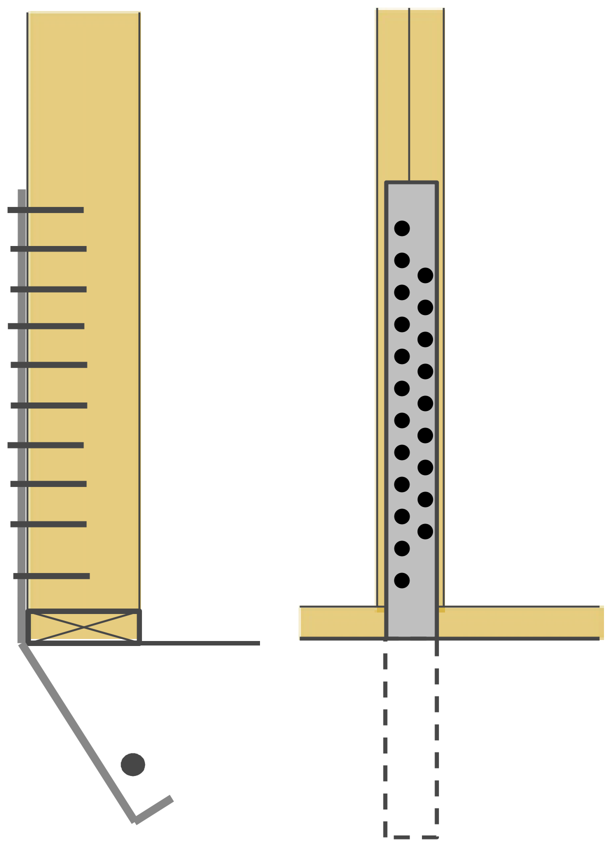
Dead load from above (Wall, Floor, Roof) can be used to resist some or all overturning forces, depending on magnitude

ASD Load Combinations of ASCE 7-10:

<u>**0.6D**</u> + 0.6W

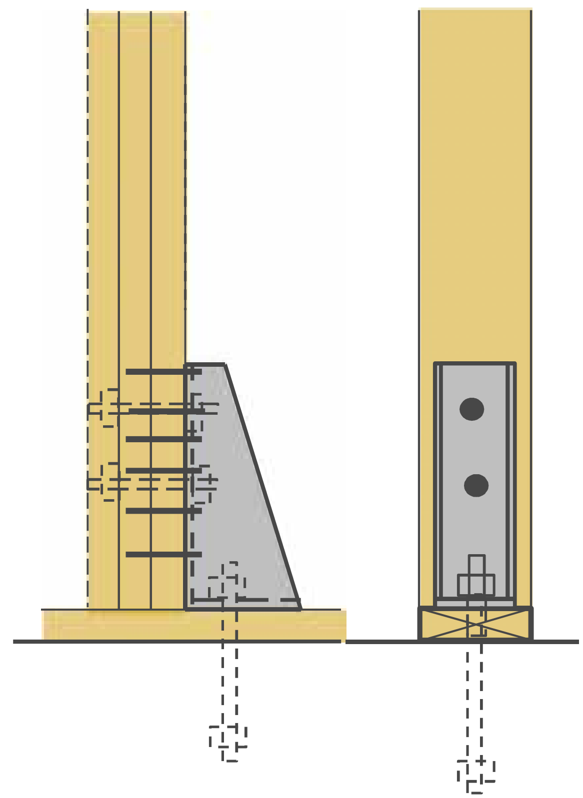
<u>**0.6D**</u> + 0.7E

Shear Wall Holdown Options



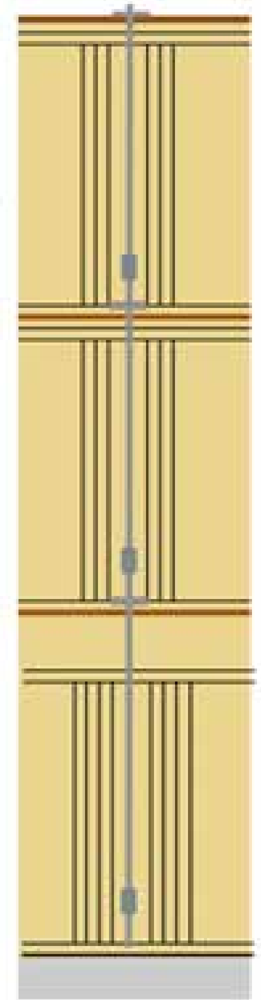
Strap Holdown

6+ kip story to story capacities



Bucket-Style Holdown

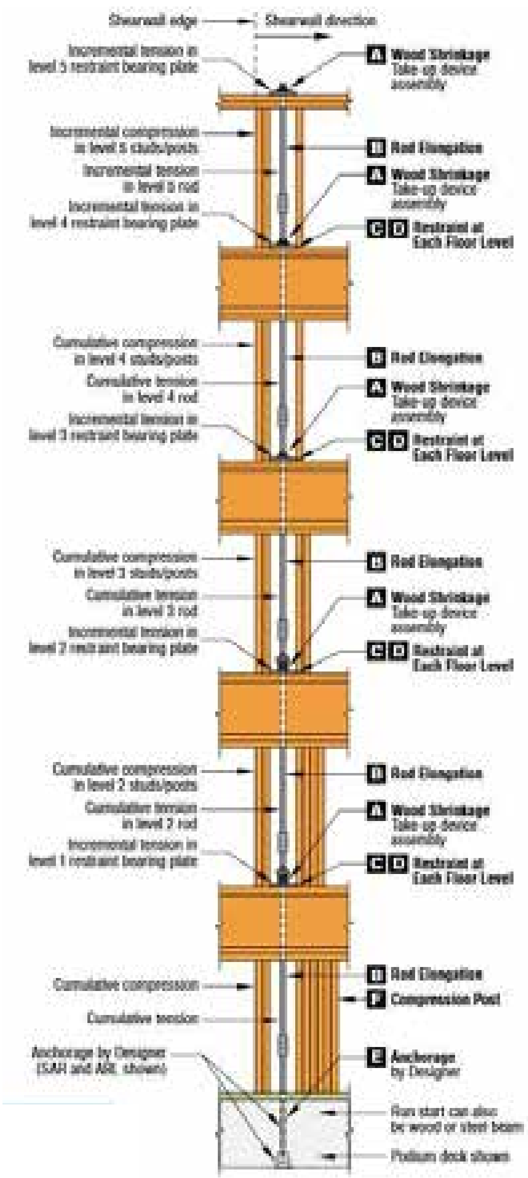
13+ kip capacities



Continuous Rod Tiedown Systems

100+ kip capacities 20+ kips/level

Shearwall Hold Downs

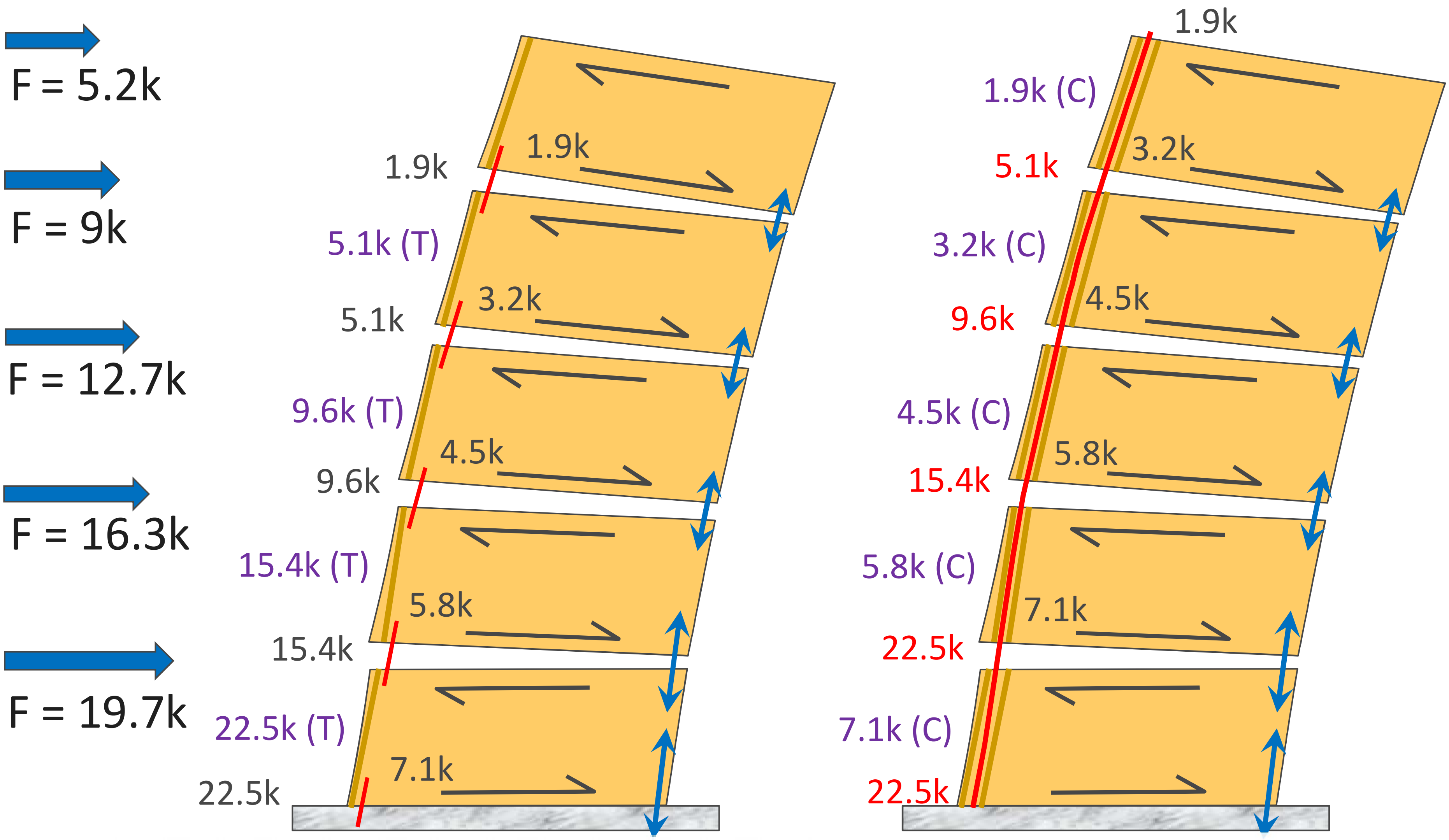


Multi-Story Continuous Rod Tiedown System

Source: MiTek

Source: Simpson Strong Tie

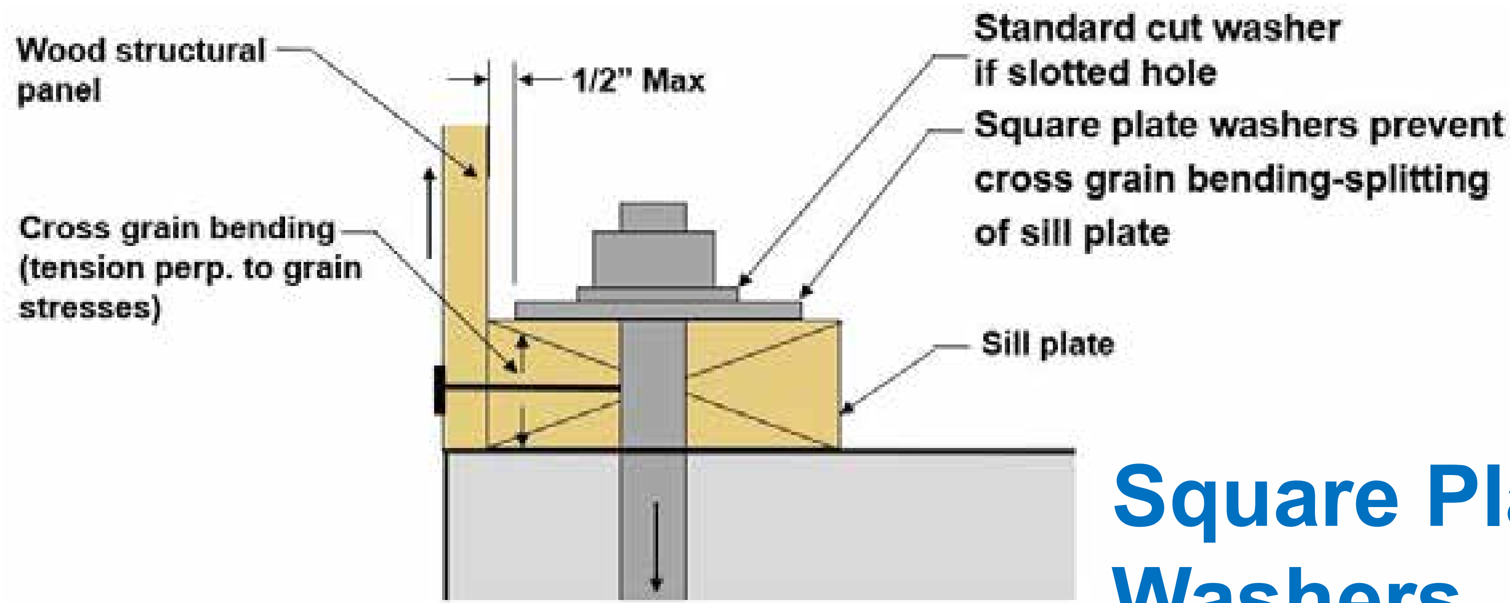
Components of Shear Wall Design



Discrete (strap or bucket) Holdown System

Continuous Rod Holdown System

Shear Wall Anchorage

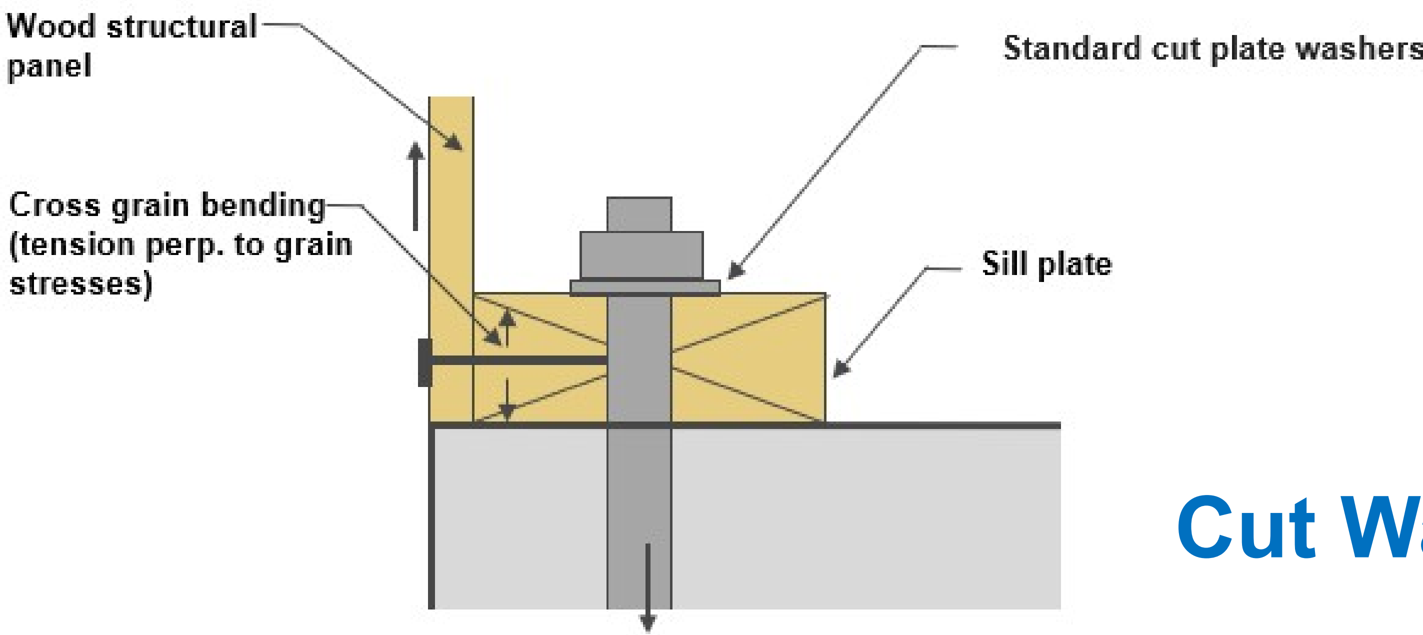


Square Plate Washers

4.3.6.4.3 Anchor Bolts:

- **Foundation anchor bolts shall have a steel plate washer under each nut.**
- **Minimum size-0.229”x3”x3” in.**
- **The hole in the plate washer - Diagonally slotted, width of up to 3/16” larger than the bolt diameter, and a slot length not to exceed 1-3/4” is permitted if standard cut washer is provided between the nut and the plate.**
- **The plate washer shall extend to within 1/2" of the edge of the bottom plate on the side(s) with sheathing.**
- **Required where sheathing nominal unit shear capacity is greater than 400 plf for wind or seismic. (i.e. 200 plf ASD, 320 plf LRFD)**

Shear Wall Anchorage



Cut Washers

Standard cut washers

- **Permitted to be used where anchor bolts are designed to resist shear only and the following requirements are met:**

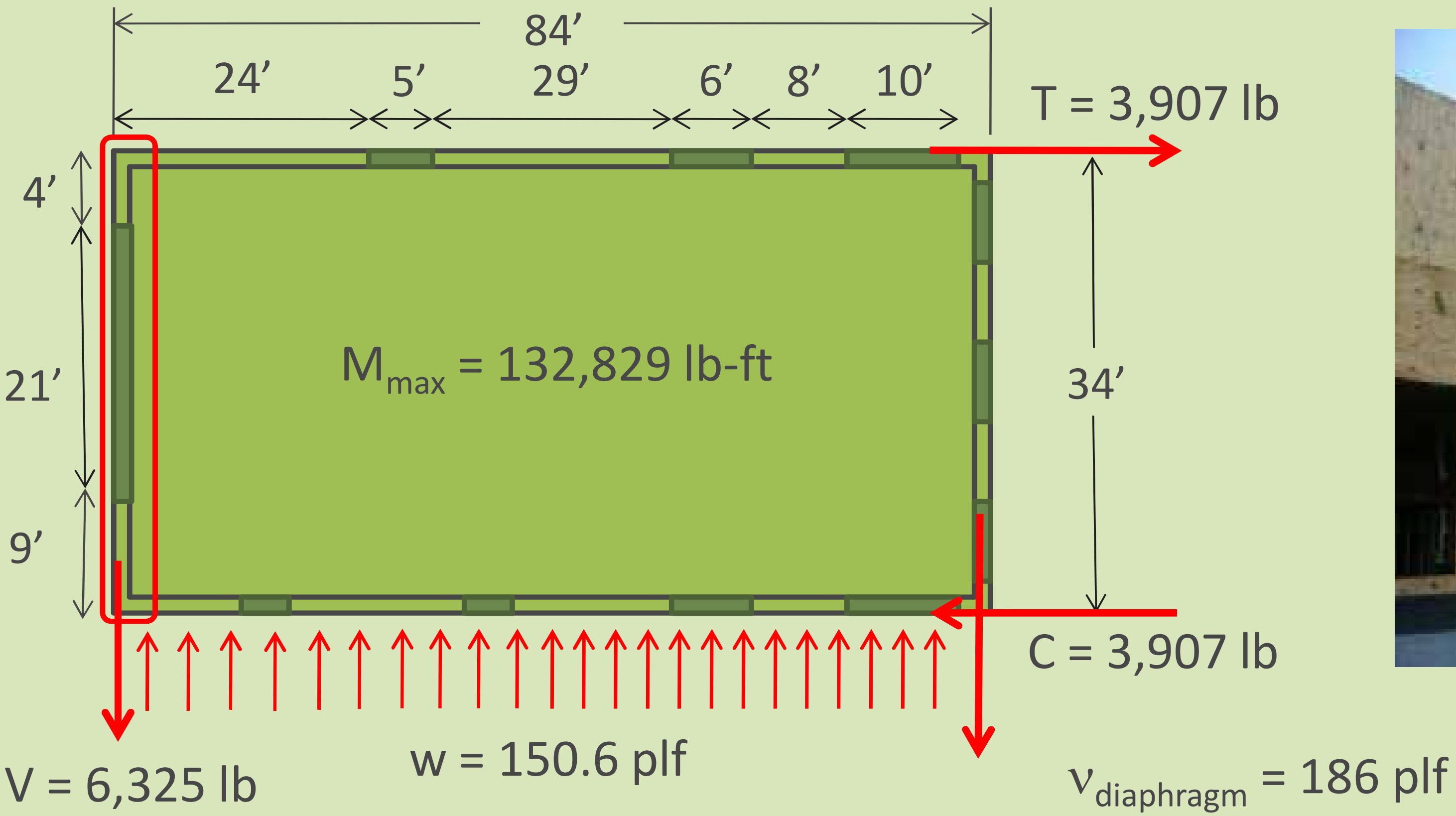
a) The shear wall is designed *segmented wall* with required uplift anchorage at shear wall ends sized to resist overturning <u>neglecting</u> DL stabilizing moment.

b) Shear wall aspect ratio, h:b, does not exceed 2:1.

c) The <u>nominal</u> unit shear capacity of the shear wall does not exceed 980 plf for seismic or 1370 plf for wind.

Retail Restaurant – Shear Wall Design

- ***Shear wall capacity: wall sections not equal in width***
- Assume 15/32", *Wood Structural Panels - Sheathing* attached with 8d nails @ 3" o.c to 2x6 Spruce Pine Fir framing spaced 16" o.c.

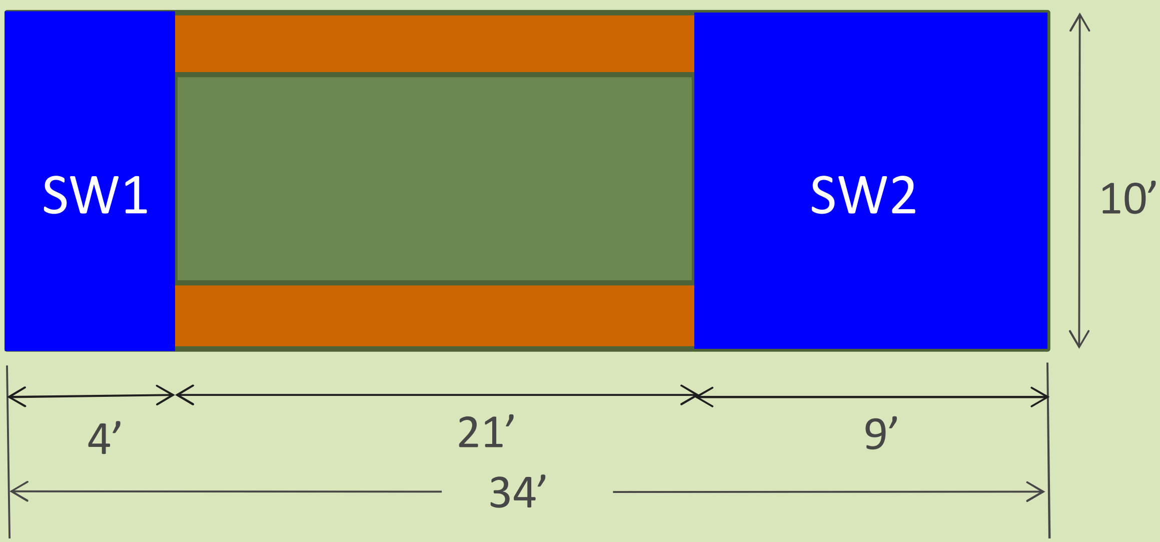


Shear Wall Aspect Ratios

Check Aspect Ratios: Assume blocked WSP shear wall

Shear Wall aspect ratios: SW1 = 10'/4' = 2.5 < 3.5 OK
SW2 = 10'/9' = 1.1 < 3.5 OK

Note that aspect ratio of SW1 is greater than 2, so it's capacity will need to be adjusted per SDPWS 4.3.4.2

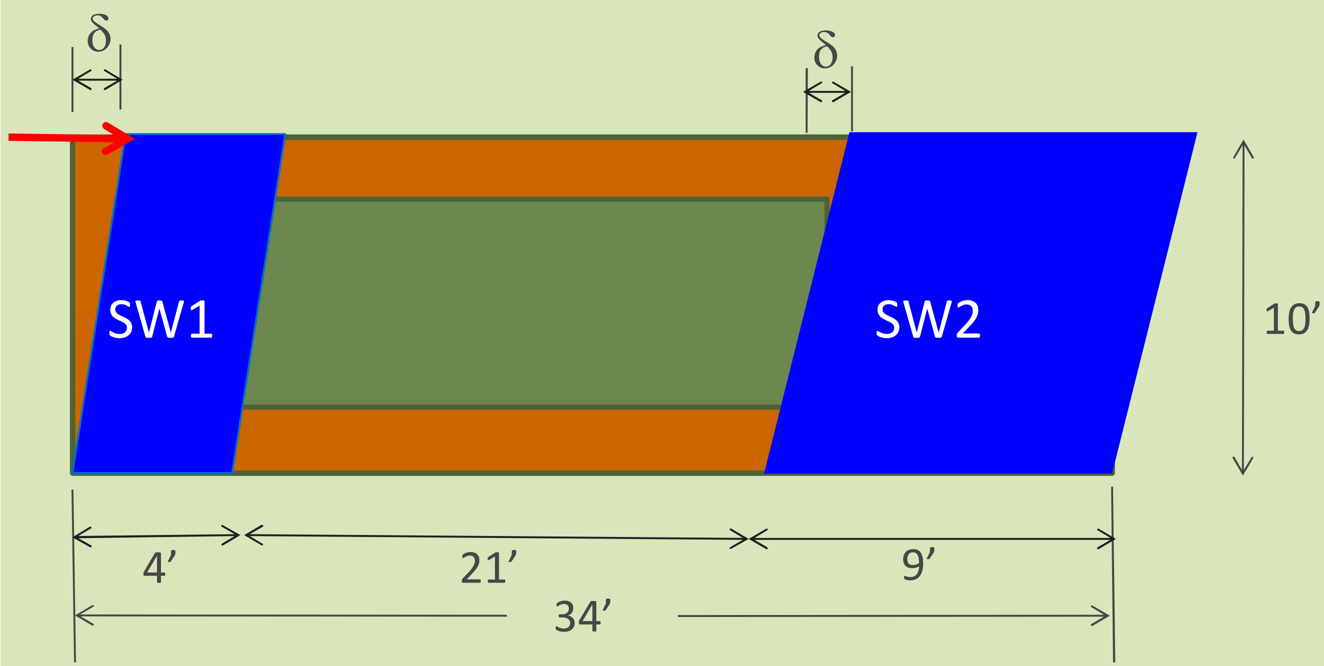


Rear Wall Elevation

Shear Walls in a Line

SDPWS 4.3.3.4.1
Shear distribution to individual shear walls in a shear wall line shall provide the same calculated deflection, δ_{sw}, in each shear wall.

$\delta_{SW1} = \delta_{SW2}$ = *Equal Deflection Method*



Given the same load, which shear wall will deflect less?

Shear Walls in a Line

Equal Deflection Method

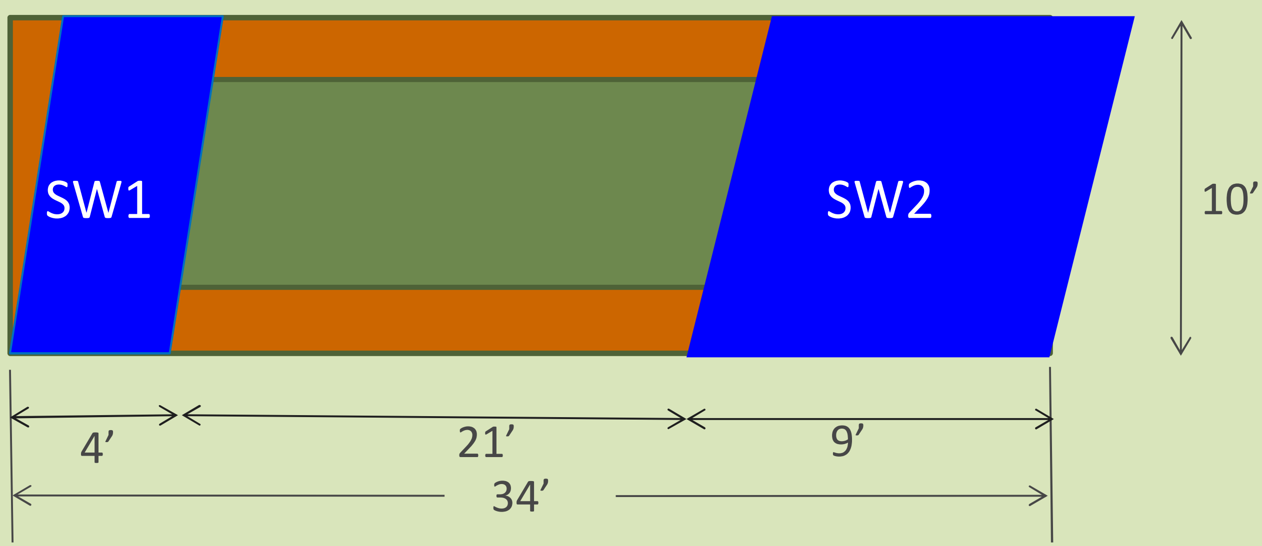
SW1

$h/b_s = 2.5 > 2$

Aspect Ratio Factor = $1.25-0.125(h/b_s) = 0.938$ *(SDPWS 4.3.4.2)*

Nominal Unit Shear Capacity = 1,370 lb/ft (SDPWS Table 4.3A)

Adjusted ASD Capacity = [(1,370 plf)(0.92)/2]*0.938 = 591 lb/ft



Shear Walls in a Line

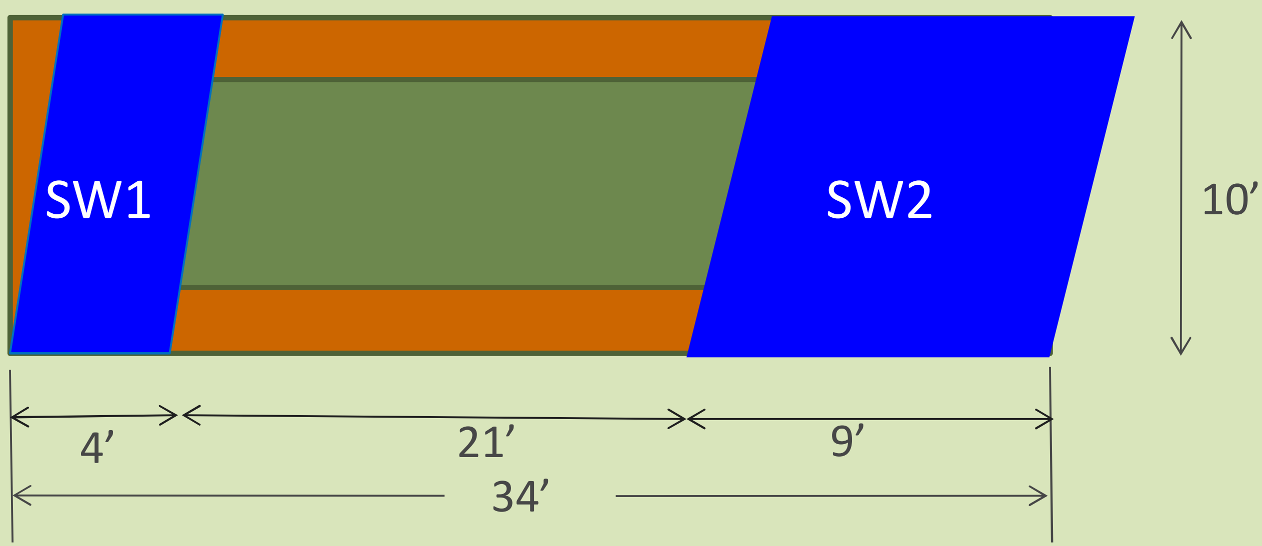
Equal Deflection Method

SW2

$h/b_s = 1.1 < 2$

Nominal Unit Shear Capacity = 1,370 lb/ft (SDPWS Table 4.3A)

Adjusted ASD Capacity = (1,370 plf)(0.92)/2 = 630 lb/ft



Shear Walls in a Line

Determine the deflection of **SW2** at its ASD unit shear capacity

$$\delta_{sw} = \underbrace{\frac{8vh^3}{EAb}}_{(bending)} + \underbrace{\frac{vh}{1000G_a}}_{(shear)} + \underbrace{\frac{h}{b}\Delta_a}_{(wall\ anchorage\ slip)} \qquad (C4.3.2\text{-}2)$$

v = 630 lb/ft

E = 1,400,000 psi (NDS Supplement Table 4A)

A = 2(1.5"x5.5") = 16.5 in^2 (2-2x6 stud end post)

b = 9'

h = 10'

G_a = 14 k/in (SDPWS Table 4.3A)

Δ_a = vertical elongation of wall anchorage

Shear Walls in a Line

Determine the deflection of **SW2** at its ASD unit shear capacity

$$\delta_{sw} = \underbrace{\frac{8vh^3}{EAb}}_{\text{(bending)}} + \underbrace{\frac{vh}{1000G_a}}_{\text{(shear)}} + \underbrace{\frac{h}{b}\Delta_a}_{\text{(wall anchorage slip)}} \qquad \text{(C4.3.2-2)}$$

**From the holdown manufacturer, the deflection of the anchor at its capacity of 6,560 lbs = 0.091”*

SW2 anchorage force = (630 lb/ft)(10’) = 6,300 lb

*Assuming vertical elongation is linear, we can calculate elongation for our load of 6,300 lbs.

Δ_a = 6,300 * 0.091” / 6,560 lb = 0.087”

δ_{SW2} = 0.571”

k = stiffness of the anchorage = F / δ (deflection / elongation)

k = 6,560 lbs / 0.091” = 72,087 lb/in

Shear Walls in a Line

Determine the unit shear in **SW1** that produces the same deflection as **SW2**

$$v_{SW1} = \frac{\delta}{\dfrac{8h^3}{EAb_{SW1}} + \dfrac{h}{1000G_a} + \dfrac{h^2}{kb_{SW1}}}$$

$$v_{SW1} = \frac{0.45''}{\dfrac{8(10')^3}{(1{,}400{,}000)(16.5)(4')} + \dfrac{(10')}{1000(14{,}000)} + \dfrac{(10')^2}{(64{,}924)(4')}}$$

$v_{SW1} =$ *497 lb/ft < 591 lb/ft*

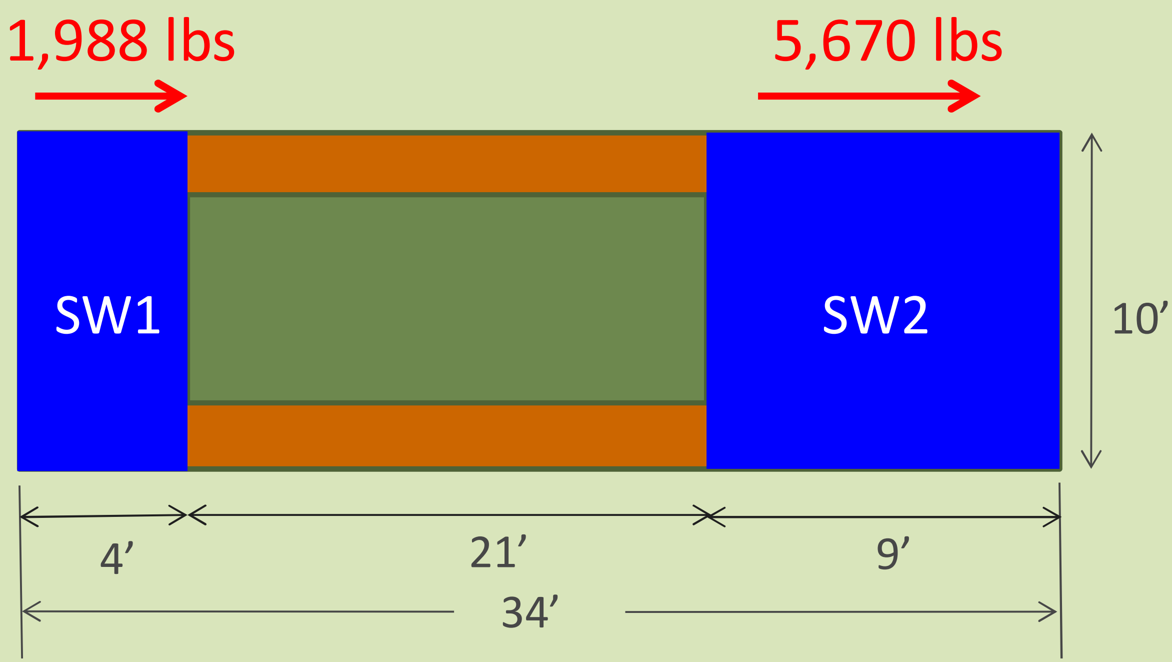
Shear Walls in a Line

Shear Wall Line Capacity

V = (497 lb/ft)*4’ + (630 lb/ft)*9’

V = 1,988 lbs + 5,670 lbs

V = 7,658 lb > 6,325 lb

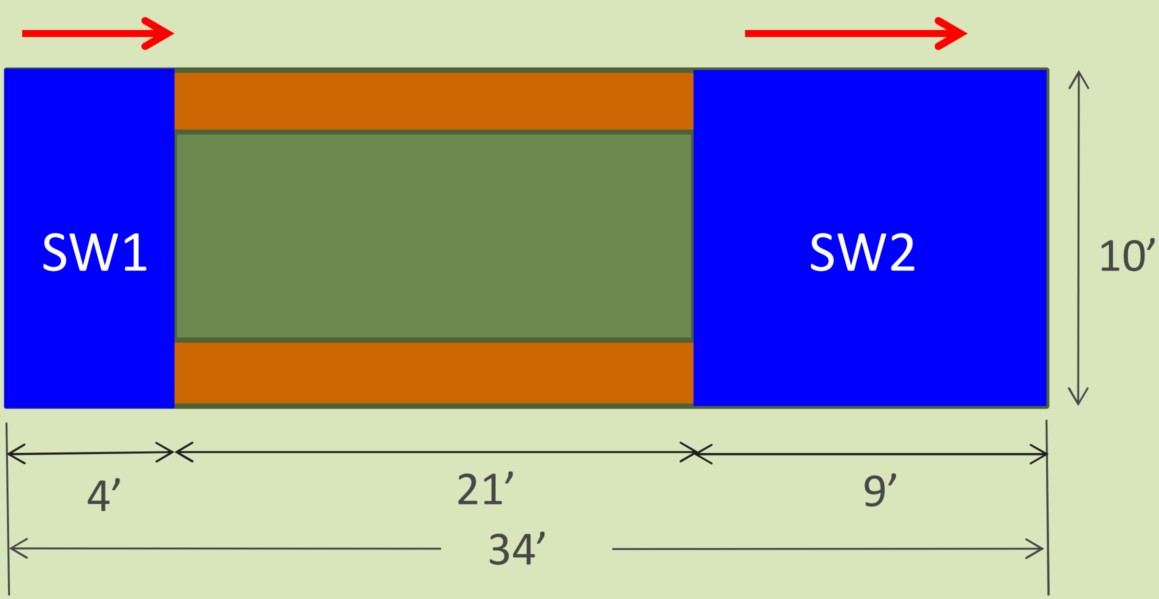


Shear Walls in a Line

Simplified Method

For Wood Structural Panels, distribution of shear in proportion to shear strength of each shear wall is permitted provided that shear walls with aspect ratio greater than 2:1 have strength adjusted by the $2b_s/h$ factor.

2015 SDPWS 4.3.3.4.1, Exception 1.



Exceptions:

1. Where nominal shear capacities of all wood structural panel shear walls with aspect ratios (h/b_s) greater than 2:1 are multiplied by $2b_s/h$ for design, shear distribution to individual full-height wall segments shall be permitted to be taken as proportional to the shear capacities of individual full height wall segments used in design. Where multiplied by $2b_s/h$, the nominal shear capacities need not be reduced by the adjustment in 4.3.4.2.

Shear Wall Aspect Ratios

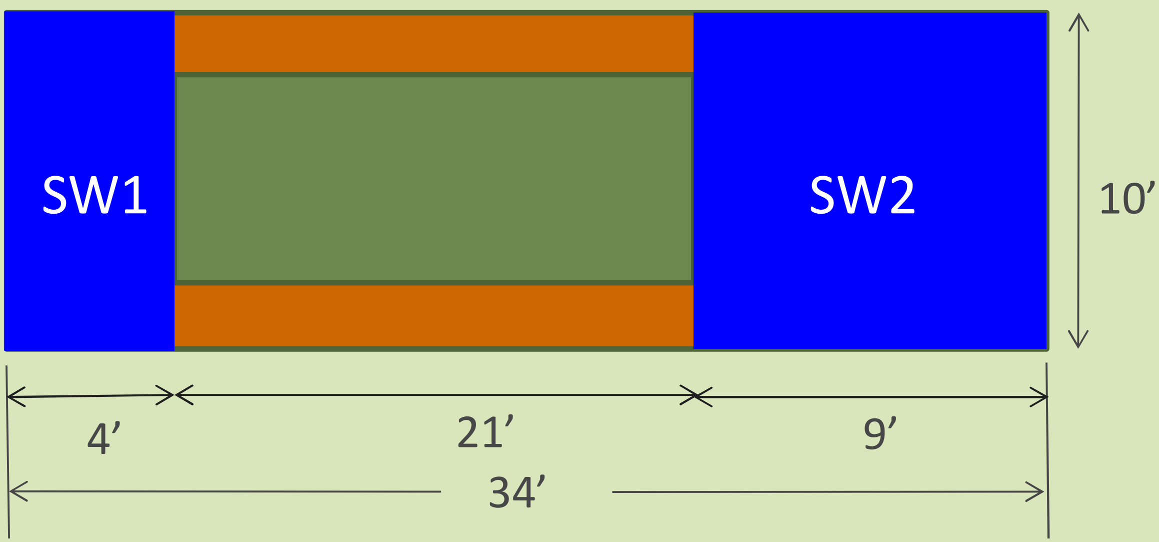
Simplified Method

Aspect Ratios: Assume blocked WSP shear wall

Shear Wall aspect ratios: *SW1 = 10'/4' = 2.5 < 3.5 OK*

(SDPWS Table 4.3.4) *SW2 = 10'/9' = 1.1 < 3.5 OK*

SW1 = 2.5 > 2 : nominal shear capacity will need to be adjusted by $2b_s/h$ *per SDPWS 4.3.3.4.1 Exception 1.*



Rear Wall Elevation

Shear Walls in a Line

Simplified Method

SW1

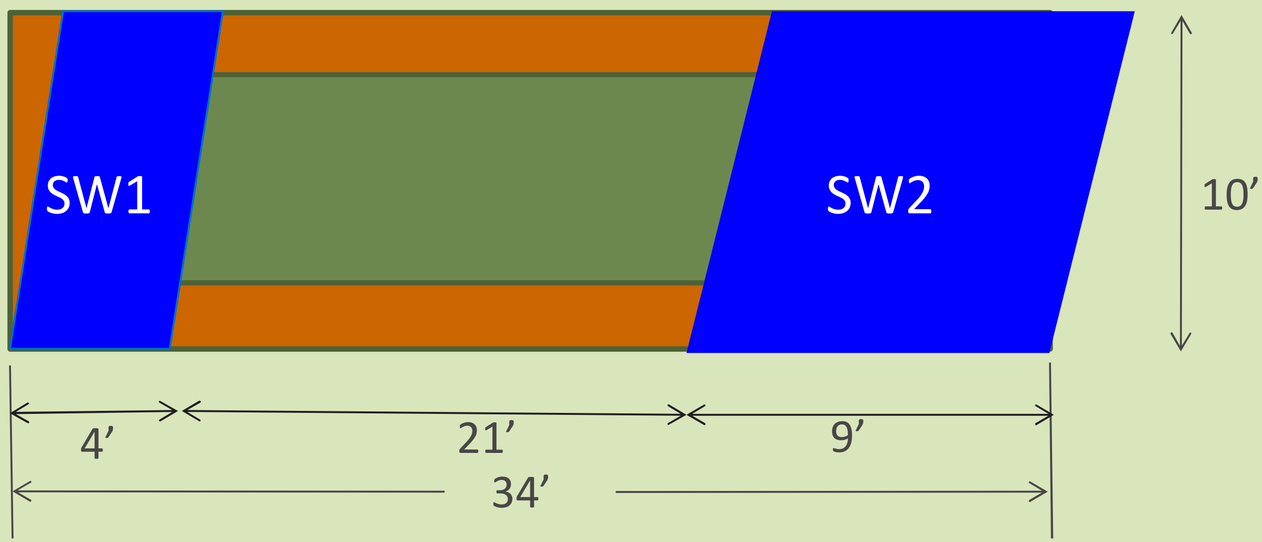
$h/b_s = 2.5 > 2$

~~Aspect Ratio Factor = 1.25-0.125(h/b_s) = 0.938 *(SDPWS 4.3.4.2)*~~

$2b_s/h$ = 2(4'/10') = 0.8

Nominal Unit Shear Capacity = 1,370 lb/ft (SDPWS Table 4.3A)

Adjusted ASD Capacity = [(1,370 plf)(0.92)/2]*0.8 = 504 lb/ft



Shear Walls in a Line

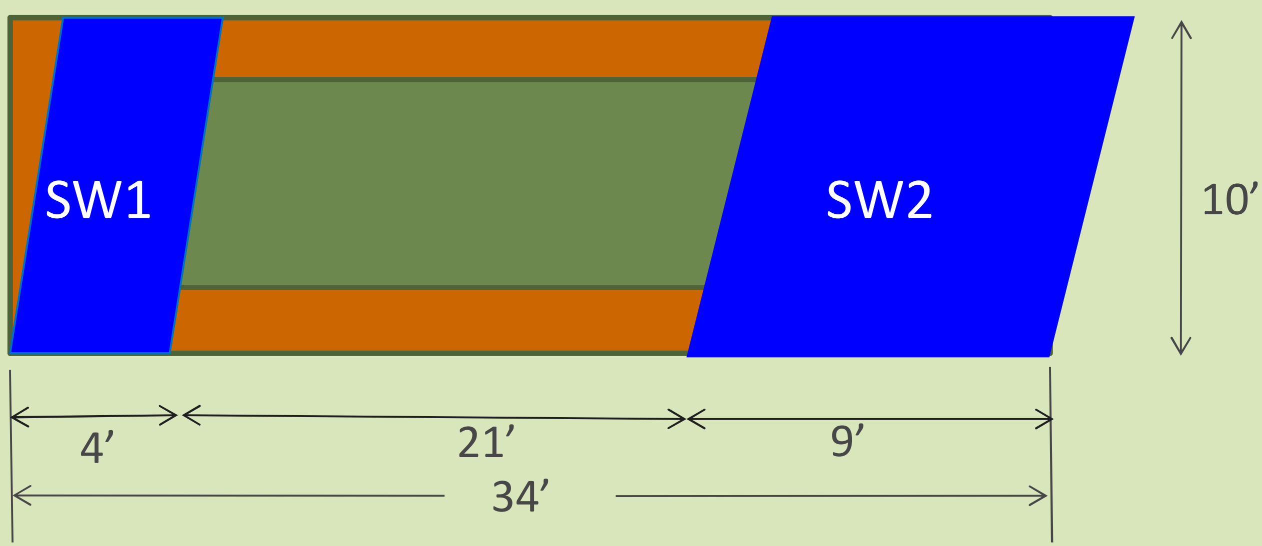
Simplified Method

SW2

$h/b_s = 1.1 < 2$

Nominal Unit Shear Capacity = 1,370 lb/ft (SDPWS Table 4.3A)

Adjusted ASD Capacity = (1,370 plf)(0.92)/2 = 630 lb/ft



Shear Walls in a Line

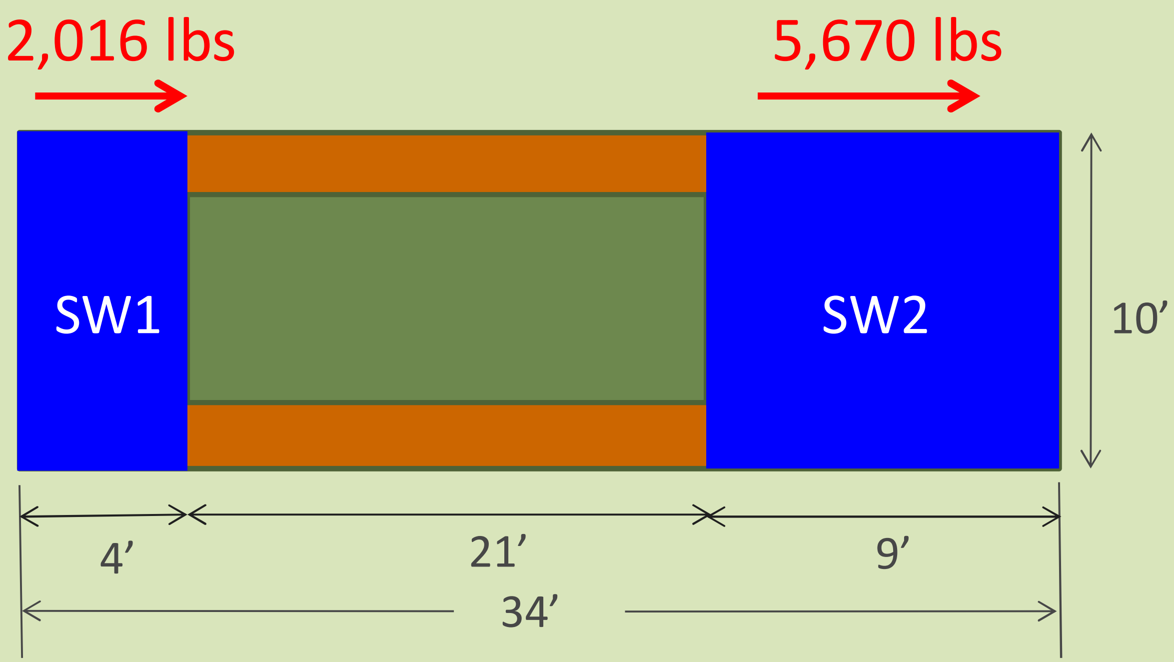
Simplified Method

Shear Wall Line Capacity

V = (504 lb/ft)*4' + (630 lb/ft)*9'

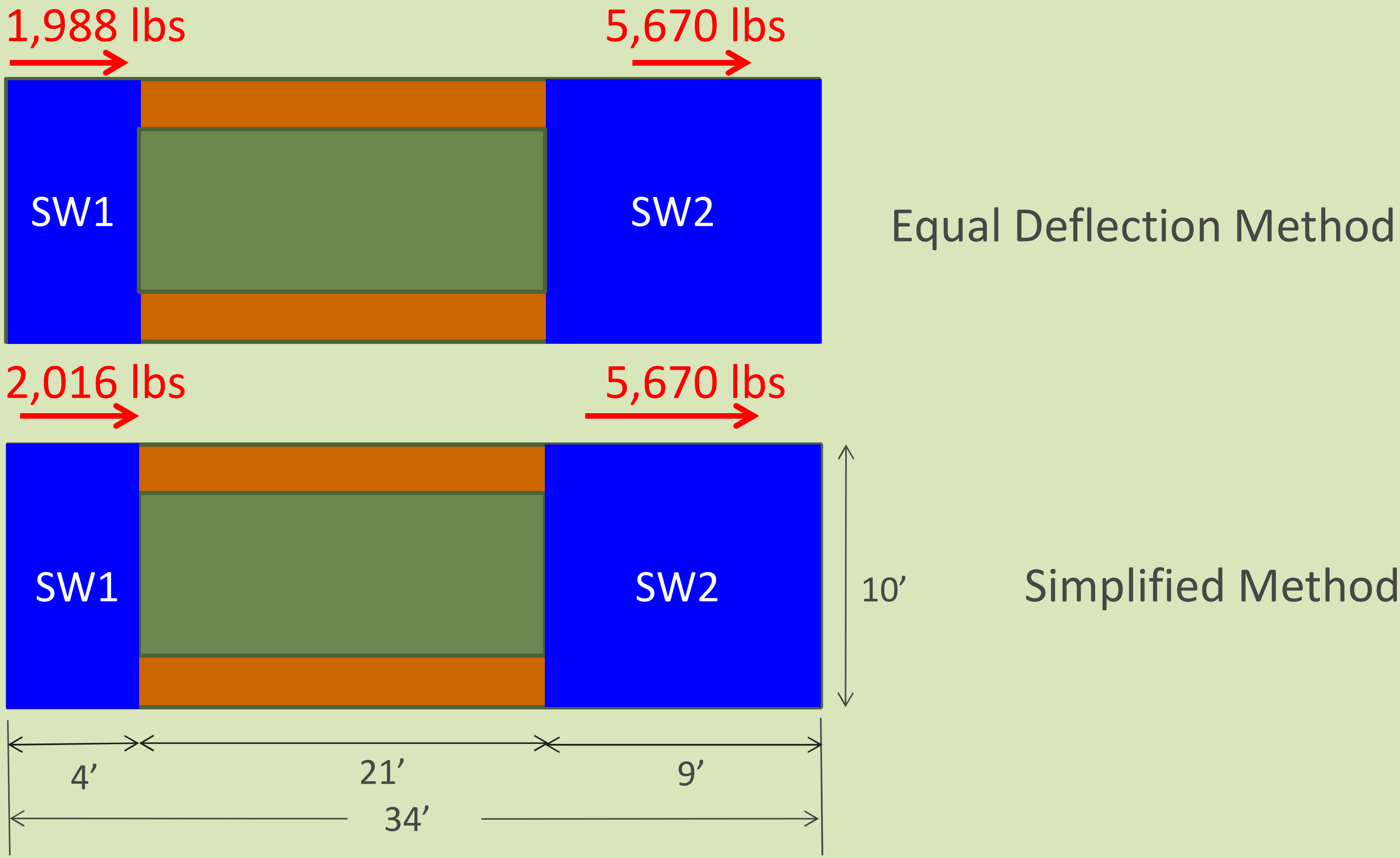
V = 2,016 lbs + 5,670 lbs

V = 7,686 lb > 6,325 lb

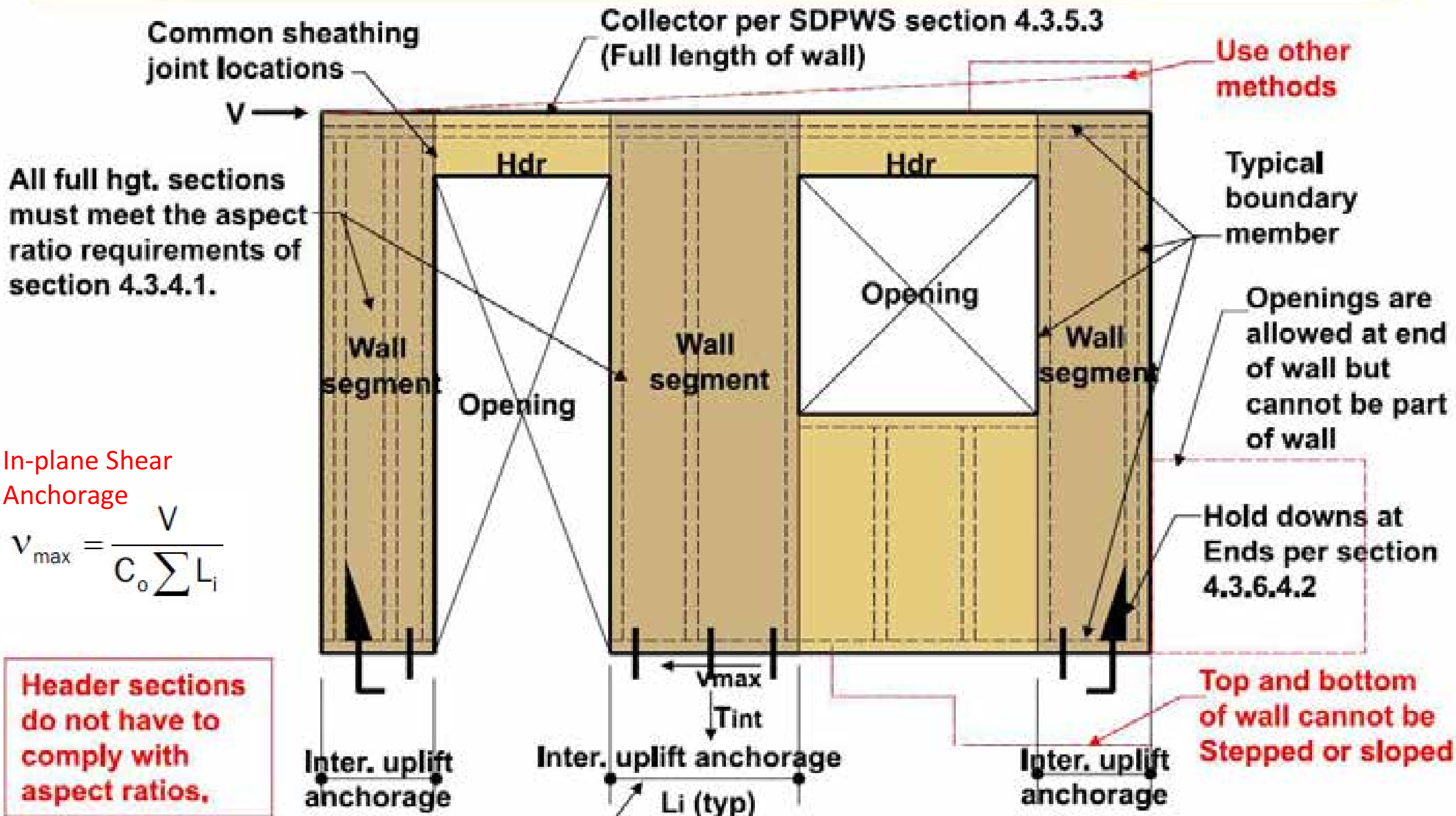


*** Note that the capacity of the wall is quite a bit higher than needed. Designer could look at increasing sheathing nail spacing.*

Shear Walls in a Line



Perforated Shear Walls- Empirical Design

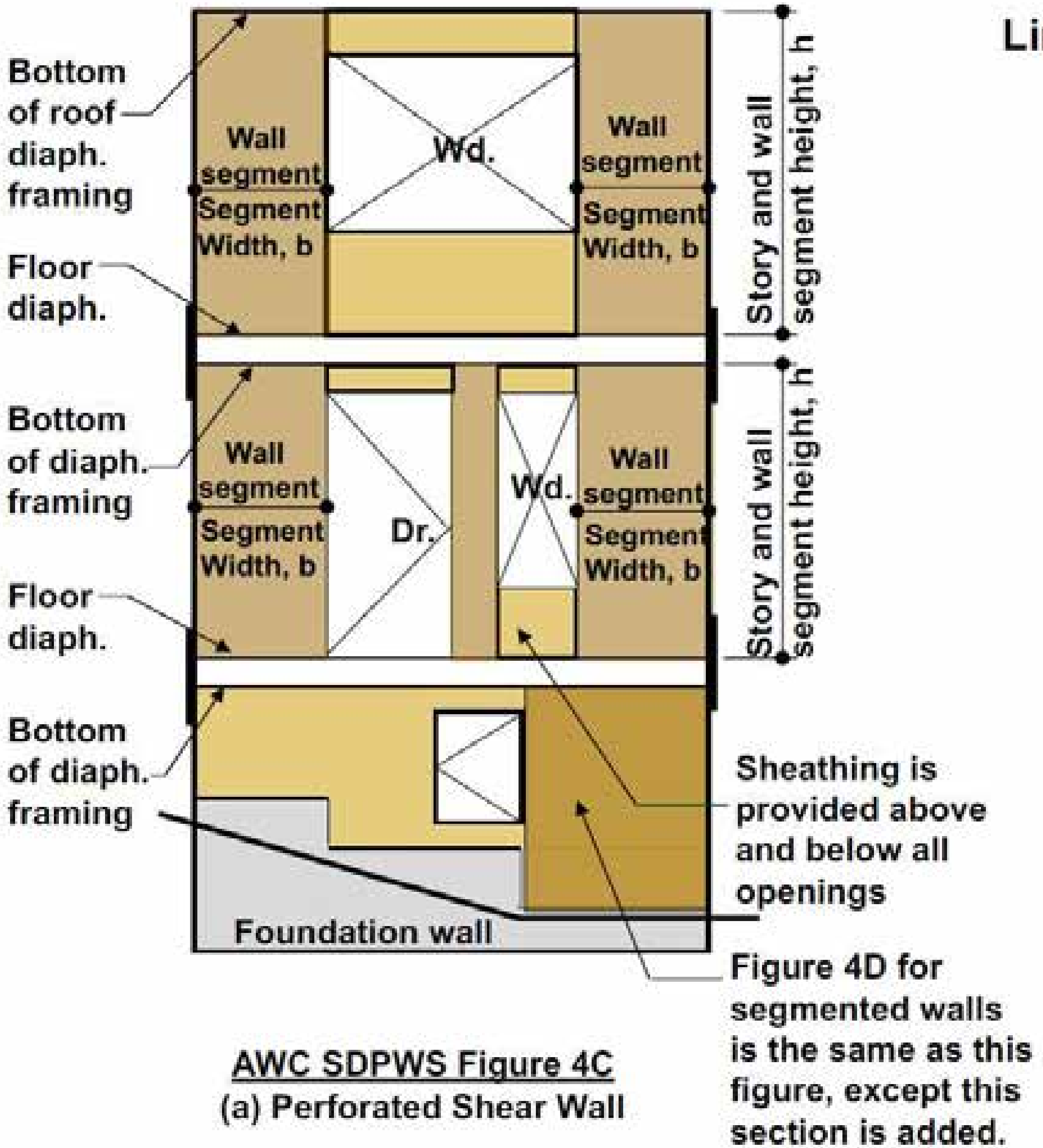


$$v_{max} = \frac{V}{C_o \sum L_i}$$

Intermediate uplift anchorage is required at each full height panel locations… "<u>in addition to</u>…" per section 4.3.6.4.2.1.

Reference examples

- APA Diaph-and-SW Construction Guide
- AWC-Perforated Shear Wall Design



AWC SDPWS Figure 4C
(a) Perforated Shear Wall

Limitations:

- The aspect ratio limitations of Table 4.3.4 shall apply.
- Sections exceeding 3.5:1 aspect ratio shall not be considered a part of the wall.
- Vn ≤ 1740 plf seismic-WSP 1 side
- Vn ≤ 2435 plf wind-WSP 1 side
- Vn ≤ 2435 plf wind-WSP 2 sides
- A full height pier section shall be located at each end of the wall.
- Where a horizontal offset occurs, portions on each side of the offset shall be considered as separate perforated walls.
- Collectors for shear transfer shall be provided through the full length of the wall.
- Uniform top-of-wall and bottom-of wall plate lines. Other conditions require other methods.
- Maximum wall height ≤ 20'.

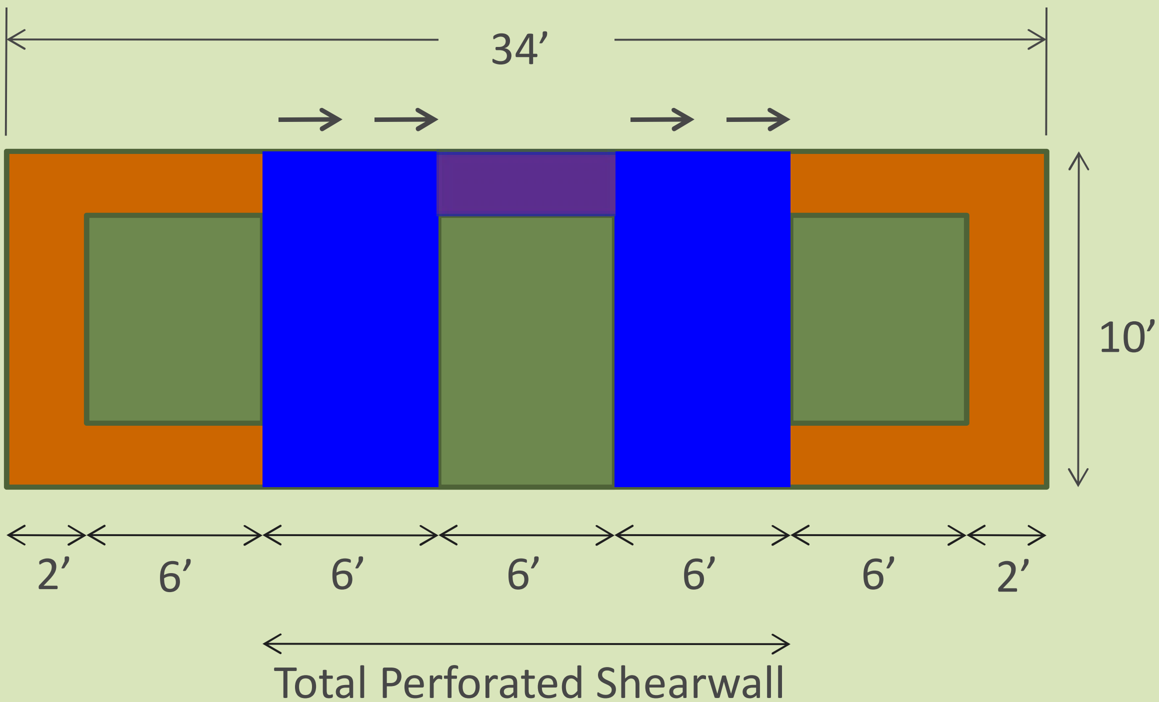
Allowable Perforated Shear Wall Aspect Ratios

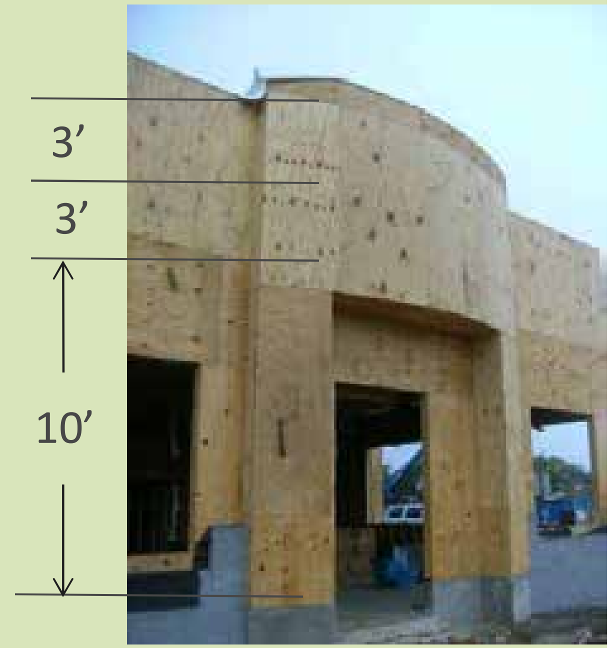
Perforated Shearwall Design

- Check Aspect Ratios: Assume blocked WSP Shearwall
- 10'/2' = 5 > 3.5; Inadequate
- 10'/6' = 1.67 < 3.5; OK

Use only full height sheathed sections to resist shear

$\nu_{shearwall}$ = 6,325 lb/ 12' = 527 plf





Perforated Shearwall Capacity

Wall has 12’/18’ = 67% full height sheathing, max. opening H = 6’-8”

Multiply capacity by 0.75 for opening 2H/3

Reduced capacity is 630 plf*0.75 = 473 plf < 527 plf, Inadequate

SDPWS Table 4.3.3.5

Table 4.3.3.5 Shear Capacity Adjustment Factor, C_o

Wall Height, h	Maximum Opening Height [1]				
	h/3	h/2	2h/3	5h/6	h
8' Wall	2'-8"	4'-0"	5'-4"	6'-8"	8'-0"
10' Wall	3'-4"	5'-0"	6'-8"	8'-4"	10'-0"
Percent Full-Height Sheathing [2]	**Effective Shear Capacity Ratio**				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

1 The maximum opening height shall be taken as the maximum opening clear height in a perforated shear wall. Where areas above and/or below an opening remain unsheathed, the height of each opening shall be defined as the clear height of the opening plus the unsheathed areas.

2 The sum of the perforated shear wall segment lengths, ΣL_i, divided by the total length of the perforated shear wall.

Perforated Shearwall Capacity

$\nu_{shearwall}$ = 527 plf

Try reducing nail spacing to 2" with 8d nails – **will require 3x framing**

Nominal Tabulated Capacity = 1790 plf
Adjusted ASD Capacity = (1790 plf)(0.92)(0.75)/2 = 618 plf
618 plf > 527 plf, OK
8d nails at 2" o.c. acceptable for perforated wall

PANEL GRADE	FASTENER TYPE & SIZE	MINIMUM PANEL THIICKNESS	MINIMUM FASTENER PENETRATION IN FRAMING	NAIL SPACING AT ALL PANEL EDGES	PANEL EDGE FASTENER SPACING
Wood Structural Panels – Sheathing	8d (2½ " x 0.131")	15/32"	1 3/8"	2 IN.	1280 (Seismic) 1790 (Wind)

SDPWS Table 4.3A

Perforated Shearwall Overturning

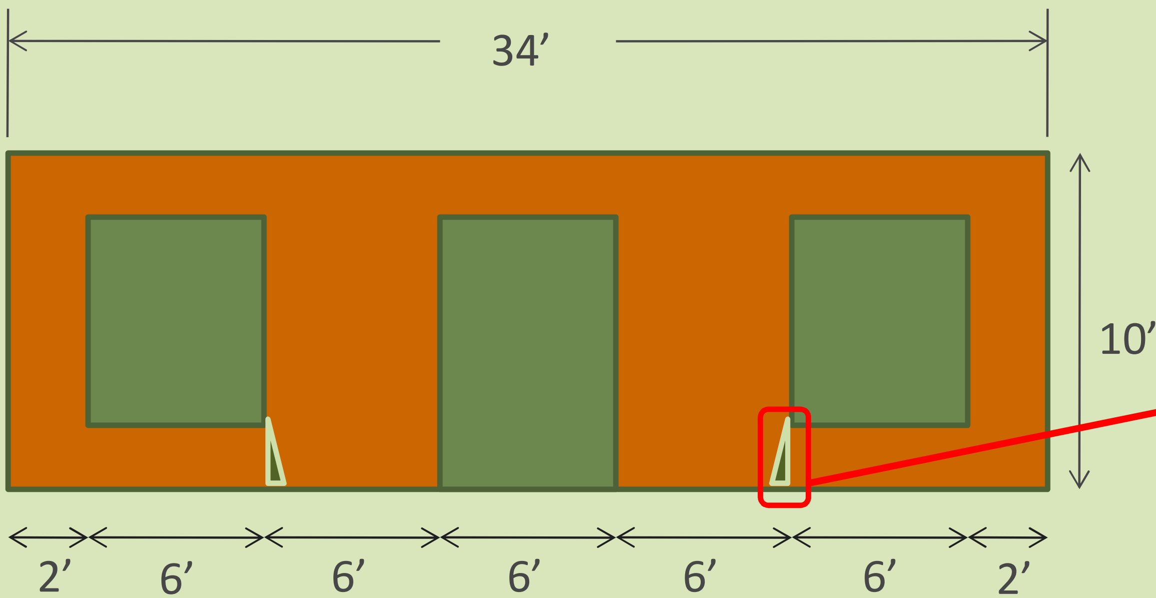
$\nu_{shearwall}$ = 527 plf

Hold downs required at ends of perforated wall

T = $\nu h/C_o$ *(similar to SDPWS equation 4.3-8)*

T = 527 plf*10'/0.75 = 7,027 lb

Selected Hold down capacity = 7,045 lb > 7,027 lb



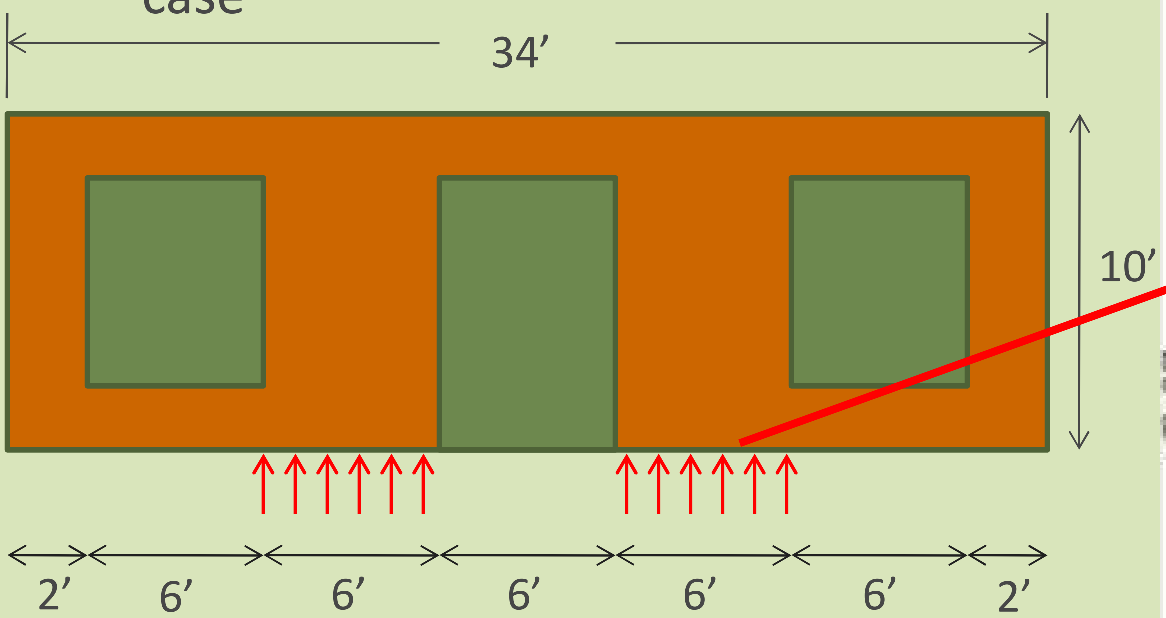
Perforated Shearwall Uplift

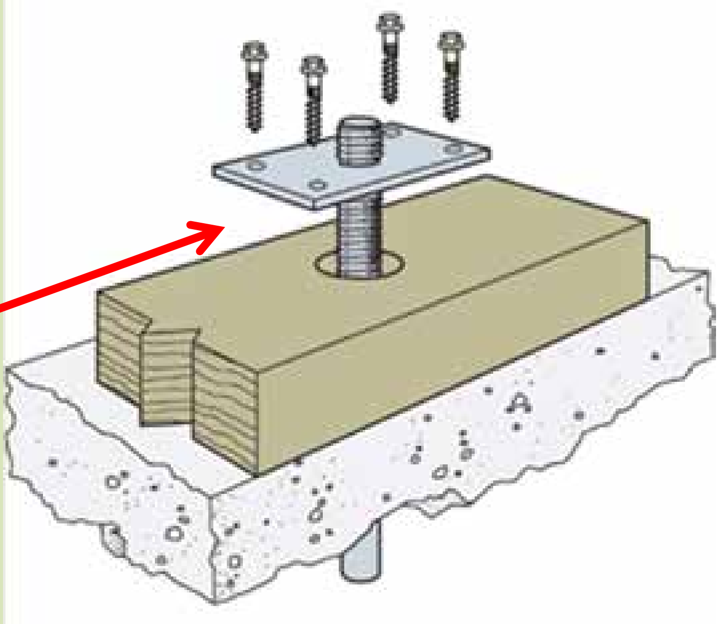
$\nu_{shearwall}$ = 527 plf/0.75 = 703 plf, use same magnitude for uniform uplift at full height segments (*SDPWS 4.3.6.4.2.1*)

One option is to use anchor bolts with large washers to resist uplift in bearing

If net washer area = 8 in^2, can resist (425 psi)(8 in^2) = 3,400 lb in uplift

- Max. anchor bolt spacing = 3,400 lb/703 plf = 4'-10" o.c.
- Will also need to check shear loads on anchor bolts for controlling case





Force Transfer Around Opening (FTAO)

SB27

Why Use Force Transfer Around Openings?

SB27 Add aspect ratio notes

Scott Breneman, 1/26/2016

Why Use Force Transfer Around Openings?

References for FTAO Design

APA Authored SEAOC Paper

https://www.apawood.org/Data/Sites/1/documents/technicalresearch/seaoc-2015-ftao.pdf

SEAOC Structural/Seismic Design Manual, Volume 2

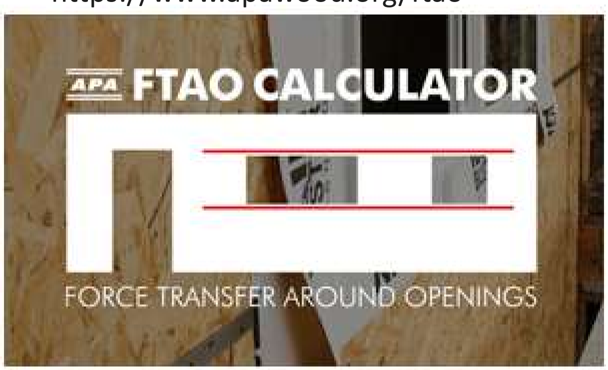
Provides narrative and worked out example

Design of Wood Structures

Textbook by Breyer et al.

Force Transfer Around Openings Calculator

https://www.apawood.org/ftao



2015 SEAOC CONVENTION PROCEEDINGS

Advancements in Force Transfer Around Openings for Wood Framed Shear Walls

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Abstract

A joint research project of *APA – The Engineered Wood Association*, University of British Columbia (UBC), and USDA Forest Products Laboratory was initiated in 2009 to examine the variations of walls with code-allowable openings. This study examines the internal forces generated during these tests and evaluates the effects of size of openings, size of full-height piers, and different analysis techniques, including the segmented method, the perforated shear wall method, and the force transfer around openings method. Full-scale wall tests as well as analytical modeling were performed. The research results obtained from this study have been used to support design methodologies in estimating the forces around the openings. Test results from the (8 foot x 12 foot) full-scale wall configurations, in conjunction with the analytical results from a computer

Introduction

Force transfer around openings (FTAO) is a popular method of shear wall analysis for wood-framed shear walls. However, the analysis method varies from engineer to engineer, published design examples typically assume the wall is symmetric around a single opening, and until recently, this design method has not been tested.

This paper discusses the shear wall design challenges structural engineers currently face, the shear wall testing that APA – The Engineered Wood Association has recently conducted, and its results. Outcomes from our testing include a rational analysis for applying FTAO to walls with asymmetric piers and walls with multiple openings, and calculating the deflection for walls detailed for FTAO. For

Double-Sided Shear Walls

High-strength wood shear walls can be double-sided with WSP sheathing on each side:

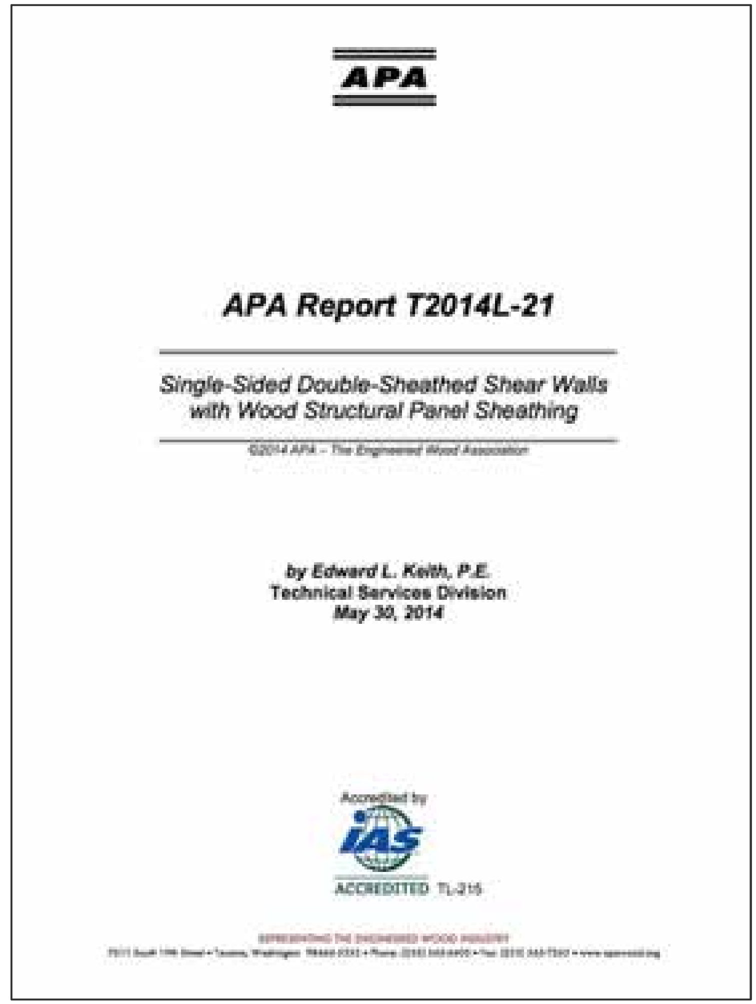
SDPWS 4.3.3.3 Summing Shear Capacities: For shear walls sheathed with the <u>same construction and materials</u> on opposite sides of the same wall, the combined nominal unit shear capacity shall be permitted to be taken as twice the nominal unit shear capacity for an equivalent shear wall sheathed on one side (4.3.5.3 has max capacities for double-sided perforated walls)

Double-Sheathed Shear Walls

There is also an option to have a single sided, double sheathed shear wall.

Testing and report by APA conclude that it is permissible to use the capacity of the wall the same as if there was one layer of WSP on each side of the wall provided that a number of criteria are met including:

- Framing members at panel joints are 3x or 2-2x
- Minimum nail spacing is 4”
- Others



APA

APA Report T2014L-21

Single-Sided Double-Sheathed Shear Walls with Wood Structural Panel Sheathing

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by Edward L. Keith, P.E.
Technical Services Division
May 30, 2014

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Gypsum Wallboard and Shear Walls

SDPWS 4.3.3.3 Summing Shear Capacities also applies to walls sheathed with gypsum wallboard on each side.

SDPWS 4.3.3.3.2 states that for shear walls sheathed with dissimilar materials on opposite sides, the combined unit shear capacity shall be either 2x the smaller nominal unit shear capacity or the larger nominal unit shear capacity, whichever is greater. The **Exception to 2015 SDPWS 4.3.3.3.2** allows the nominal sheathing capacity of gypsum wallboard to be added to the nominal sheathing capacity for the material on the opposite side for **wind** design.

Table 4.3.4 Maximum Shear Wall Aspect Ratios

Shear Wall Sheathing Type	Maximum h/b, Ratio
Wood structural panels, unblocked	2:1
Wood structural panels, blocked	3.5:1
Particleboard, blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum wallboard	2:1[1]
Portland cement plaster	2:1[1]
Structural Fiberboard	3.5:1

1 Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.

Maximum Aspect Ratio of 2:1

Gypsum Wallboard and Shear Walls

2015 SDPWS 4.3.7.2 Shear Walls using Wood Structural Panels over Gypsum Wallboard or Gypsum Sheathing Board:

Shear walls sheathed with wood structural panel sheathing over gypsum wallboard or gypsum sheathing board shall be permitted to be used to resist seismic and wind forces. The size and spacing of fasteners at shear wall boundaries and panel edges shall be as provided in Table 4.3B. The shear wall shall be constructed in accordance with Section 4.3.7.1

Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls[1]

Gypsum and Portland Cement Plaster

Sheathing Material	Material Thickness	Fastener Type & Size[2]	Max. Fastener Edge Spacing (in)[3]	Max. Stud Spacing (in.)		A SEISMIC v_s (plf)	A SEISMIC G_a (kips/in)	B WIND v_w (plf)
Gypsum wallboard, gypsum base for veneer plaster, or water-resistant gypsum backing board	1/2"	5d cooler (0.086" x 1-5/8" long, 15/64" head) or wallboard nail (0.086" x 1-5/8" long, 9/32" head) or 0.120" nail x 1-1/2" long, min 3/8" head	7	24	unblocked	150	4.0	150
			4	24	unblocked	220	6.0	220
			7	16	unblocked	200	5.5	200
			4	16	unblocked	250	6.5	250
			7	16	blocked	250	6.5	250
			4	16	blocked	300	7.5	300
		No. 6 Type S or W drywall screws 1-1/4" long	8/12	16	unblocked	120	3.5	120
			4/16	16	blocked	320	8.0	320
			4/12	24	blocked	310	8.0	310
			8/12	16	blocked	140	4.0	140
			6/12	16	blocked	180	5.0	180
	5/8"	6d cooler (0.092" x 1-7/8" long, 1/4" head) or wallboard nail (0.0915" x 1-7/8" long, 19/64" head) or 0.120" nail x 1-3/4" long, min 3/8" head	7	24	unblocked	230	6.0	230
			4	24	unblocked	290	7.5	290
			7	16	blocked	290	7.5	290
			4	16	blocked	350	8.5	350
		No. 6 Type S or W drywall screws 1-1/4" long	8/12	16	unblocked	140	4.0	140
			8/12	16	blocked	180	5.0	180

Further requirements in 2015 SDPWS 4.3.7.5

Questions?

This concludes The American Institute of Architects Continuing Education Systems Course

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