1430 Q, The HR Group Architects, Buehler Engineering, Greg Folkins Photography

Designing Light-Frame Wood Structures over Podiums: Wind Considerations

Alex Dukeman & Taylor Landry

WoodWorks

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Course Description

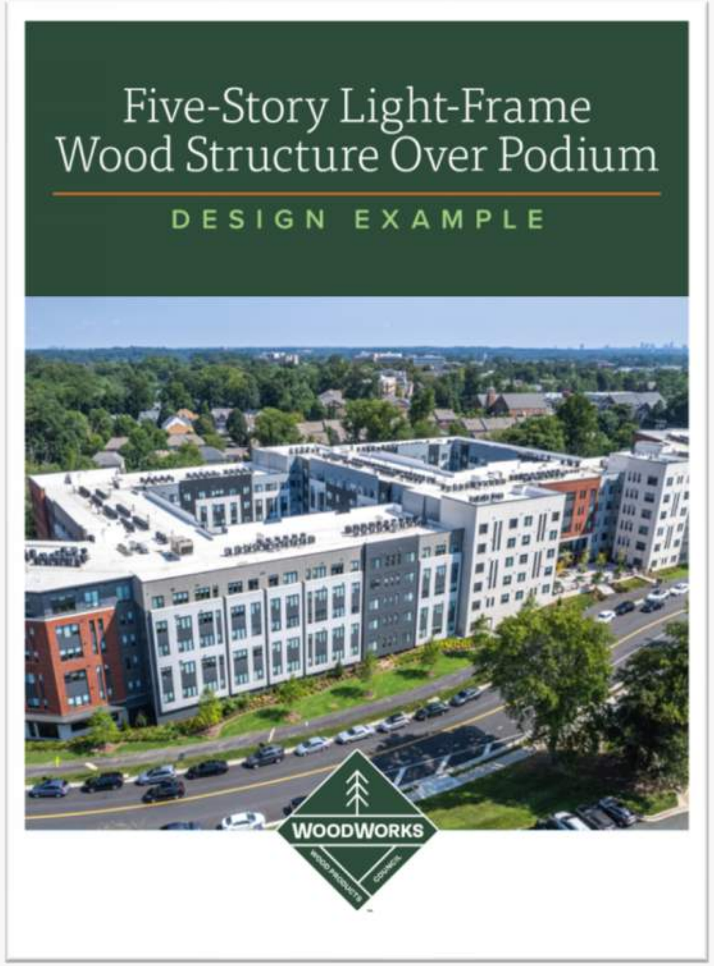
This course highlights wind design considerations for mid-rise light-frame wood buildings over concrete podiums, a common structural configuration in urban residential development. Through an abbreviated design example, participants will explore code requirements, design strategies, and material performance considerations of a sample shear wall in a high-wind, low-seismic region, based on ASCE 7-22 and the 2024 IBC. This course emphasizes critical code interpretation, common design pitfalls, and real-world considerations to ensure structural safety, durability, and code-compliant design.

This webinar will serve as both a primer and a bridge to more comprehensive guidance. A newly published design example, which complements this presentation, will also be shared with attendees.

Learning Objectives

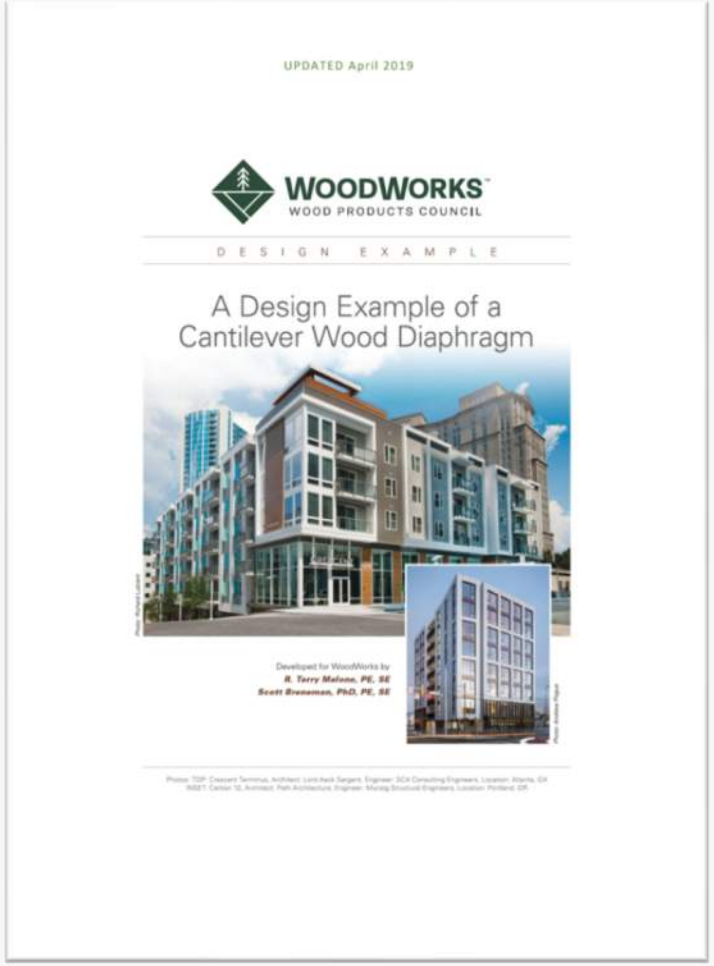
1. Explain the code provisions of the 2024 IBC, ASCE 7-22, and 2021 SDPWS that apply to the structural performance of wood-over-podium building configurations under wind loads.
2. Differentiate between wind and seismic design requirements for light-frame wood lateral force-resisting systems and identify how governing load conditions vary.
3. Evaluate the structural performance and limitations of gypsum wallboard versus wood structural panel shear walls to ensure safe, code-compliant design.
4. Analyze design strategies that enhance serviceability, minimize damage to finishes, and enhance long-term durability for occupant welfare.

Key Resources



Five-Story Light-Frame Wood Structure Over Podium

DESIGN EXAMPLE



UPDATED April 2019

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DESIGN EXAMPLE

A Design Example of a Cantilever Wood Diaphragm

Developed for WoodWorks by
R. Terry Malone, PE, SE
Scott Breneman, PhD, PE, SE

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Accommodating Shrinkage in Multi-Story Wood-Frame Structures

Richard McLain, MS, PE, SE, Senior Technical Director – Tall Wood, WoodWorks • Doug Steimle, PE, Principal, Schaefer

In wood-frame buildings of three or more stories, cumulative shrinkage can be significant and have an impact on the function and performance of finishes, openings, mechanical/electrical/plumbing (MEP) systems, and structural connections. However, as more designers look to wood-frame construction to improve the cost and sustainability of their mid-rise projects, many have learned that accommodating wood shrinkage is actually very straightforward.

Wood is hygroscopic, meaning it has the ability to absorb and release moisture. As this occurs, it also has the potential to change dimensionally. Knowing how and where wood shrinks and swells helps designers detail their buildings to minimize related effects.

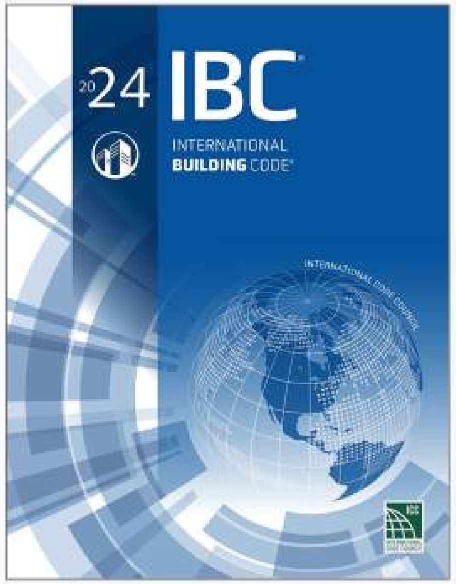
Wood shrinkage occurs perpendicular to grain, meaning that a solid sawn wood stud or floor joist will shrink in its cross-section dimensions (width and depth). Longitudinal shrinkage is negligible, meaning the length of a stud or floor joist will essentially remain unchanged. In multi-story buildings, wood shrinkage is therefore concentrated at the wall plates, floor and roof joists, and rim boards. Depending on the materials and details used at floor-to-wall and roof-to-wall intersections, shrinkage in light-frame wood construction can range from 0.05 inches to 0.5 inches per level.

This publication will describe procedures for estimating wood shrinkage and provide detailing options that minimize its effects on building performance.

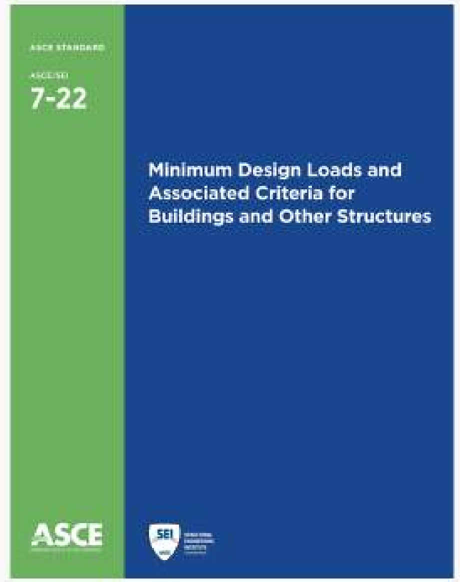
Wood Science & Shrinkage

Understanding the cellular structure of wood allows us to understand how moisture and wood interact and identify the paths that moisture typically travels. Within wood, moisture is present in two forms: (1) free water in cell cavities, and (2) bound water in cell walls. Simplistically, wood's cellular structure can be imagined as a bundle of drinking straws held together with a rubber band, with each straw representing a longitudinal cell in the wood. Water can be free water stored in the straw cavity or bound water absorbed by the straw walls. At high moisture contents, water exists in both locations. As the wood dries, the free water is released from the cell cavities before the bound water is released from the cell walls. When wood has no free water and yet the cell wall is still saturated, it is said to be at its fiber saturation point (FSP). Imagine a sponge that has just been taken out of a bucket filled with water. As the sponge is lifted from the bucket, water comes out of the pores. When the sponge is squeezed, more water comes out of the pores. The moment when no water can be squeezed out of the sponge but yet it still feels damp is analogous to the FSP. The moisture retained in the sponge is the bound water and water that has been squeezed out is the free water.

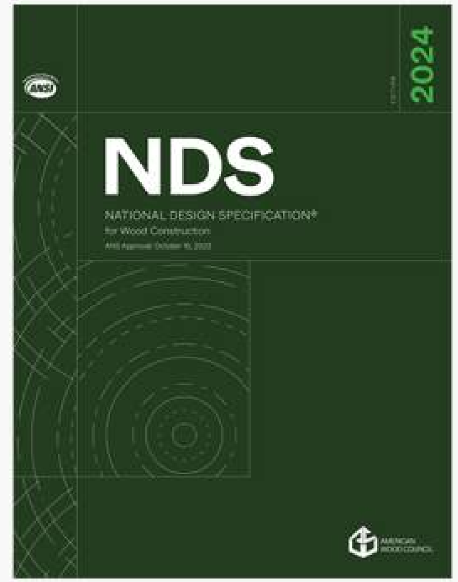
2024 IBC



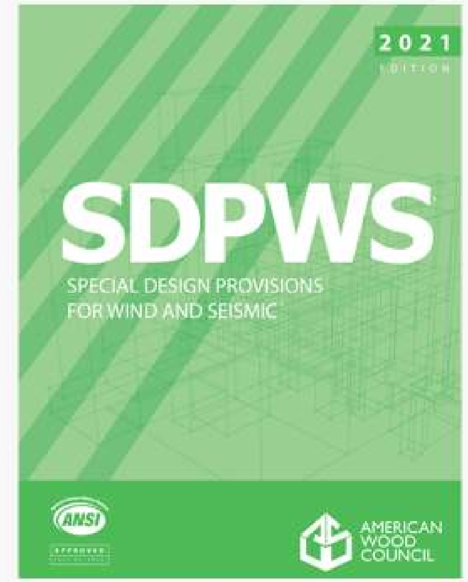
ASCE 7-22
(2022)



2024 NDS



2021 SDPWS



Design Example: Section 1.2

Outline

» Design Example Overview
» Wind vs. Seismic: What Controls?
» Wind Considerations & Loads
» Seismic Considerations & Loads
» Shear Wall Sheathing Selection: Gypsum Wallboard (GWB) vs. Wood Structural Panel (WSP)
» Design Example Comparison: Wind vs. Seismic

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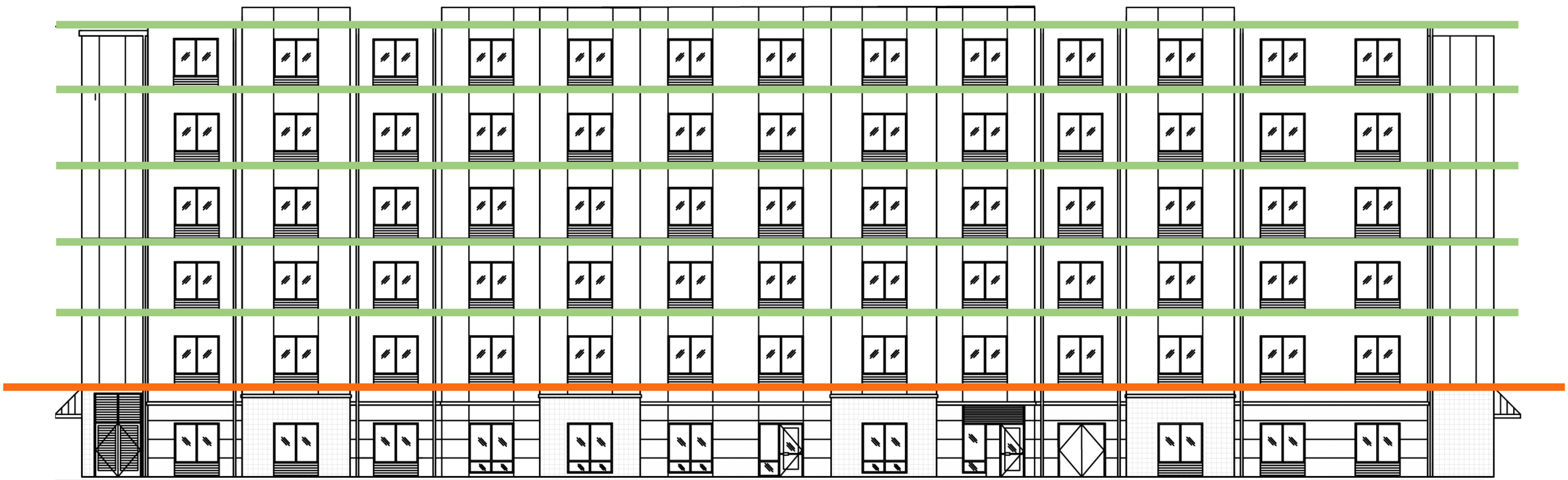
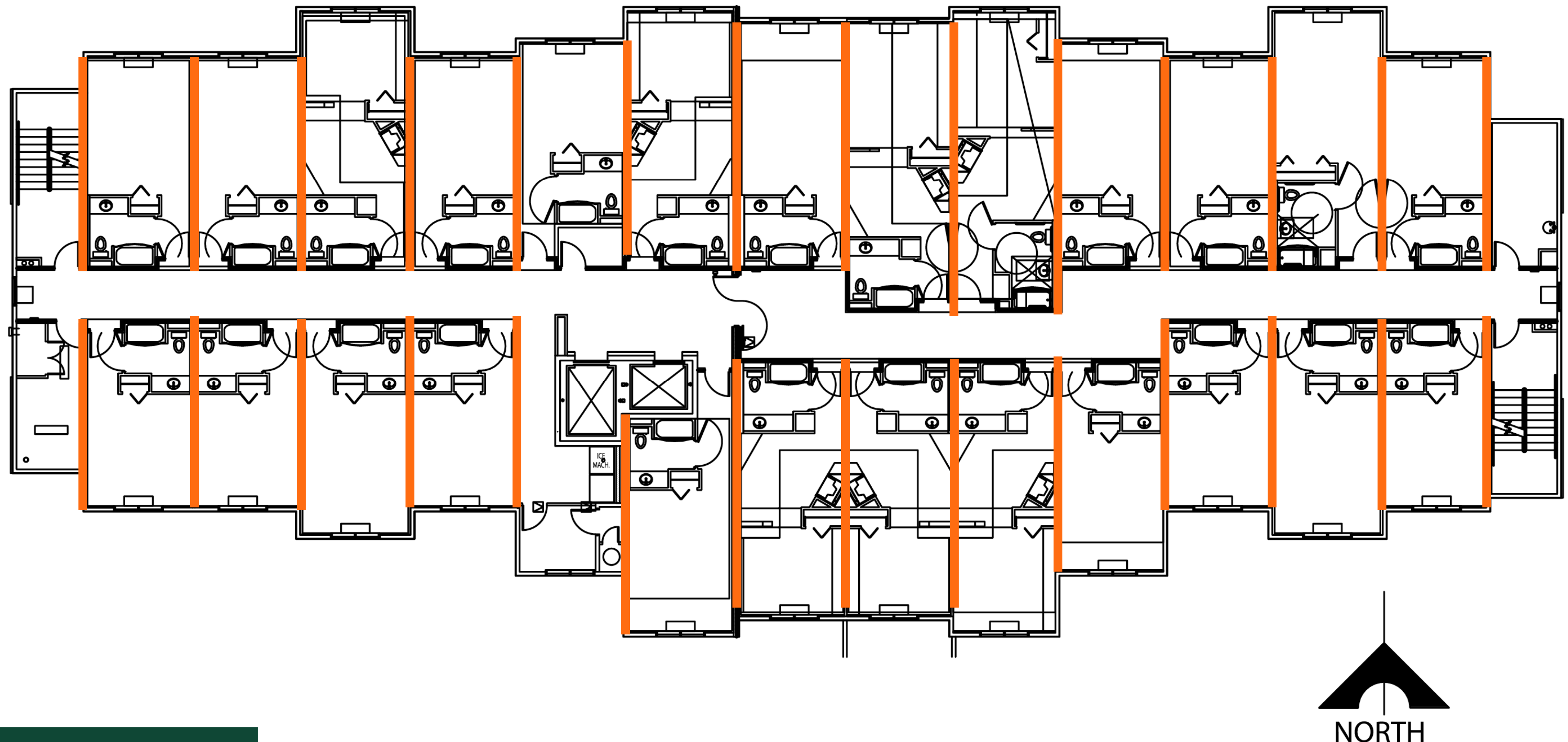
Design Example: Building Elevation

Project Location: Providence, RI



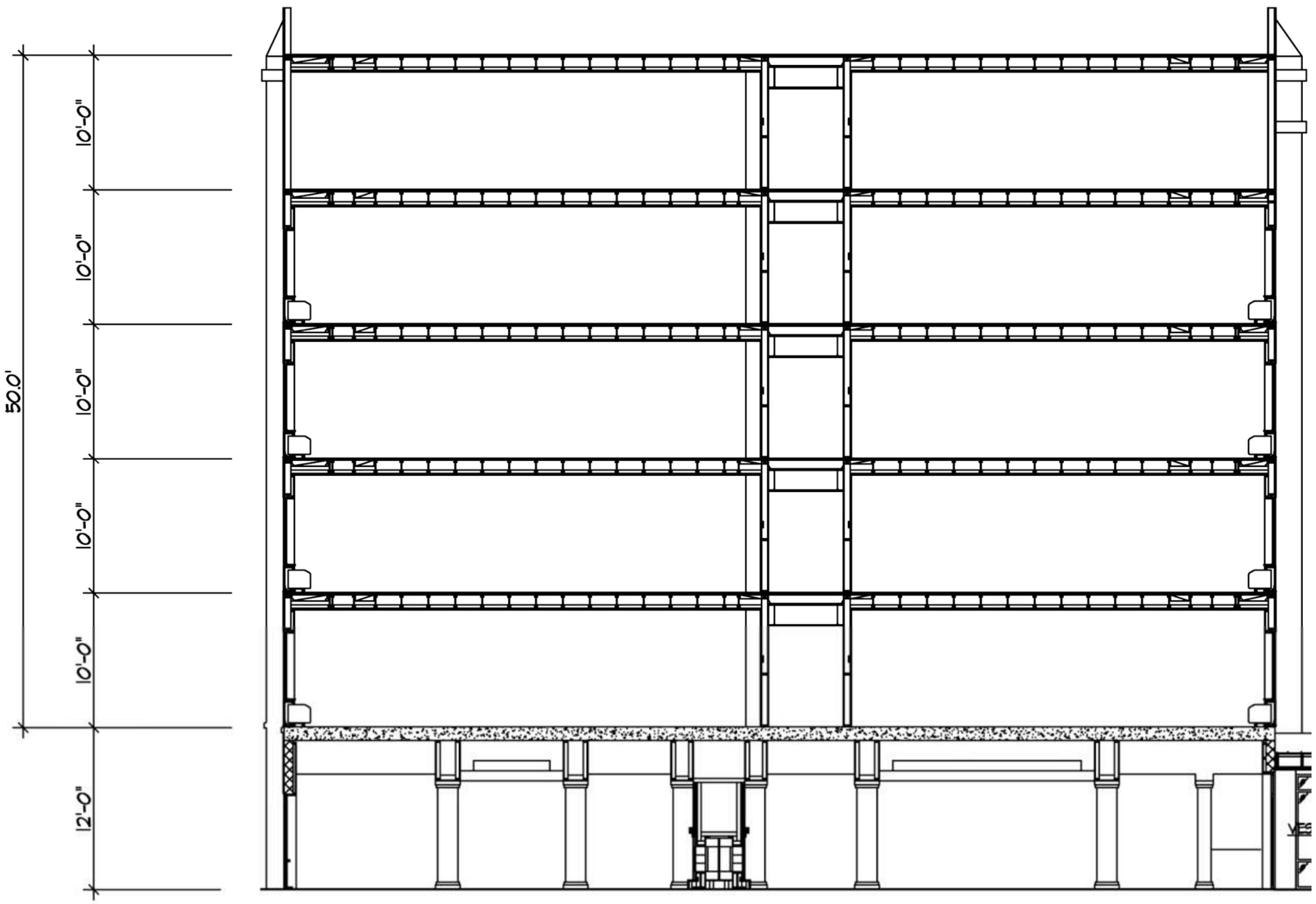
Design Example: Figure 1.1

Design Example: Plan View



Design Example: Figure 1.2

Design Example: Building Section



Roof Weights (psf)	
Dead load	28.0
Live load	20.0

Floor Weights (psf)	
Dead load	30.0
Live load	40.0

Design Example: Figure 1.3

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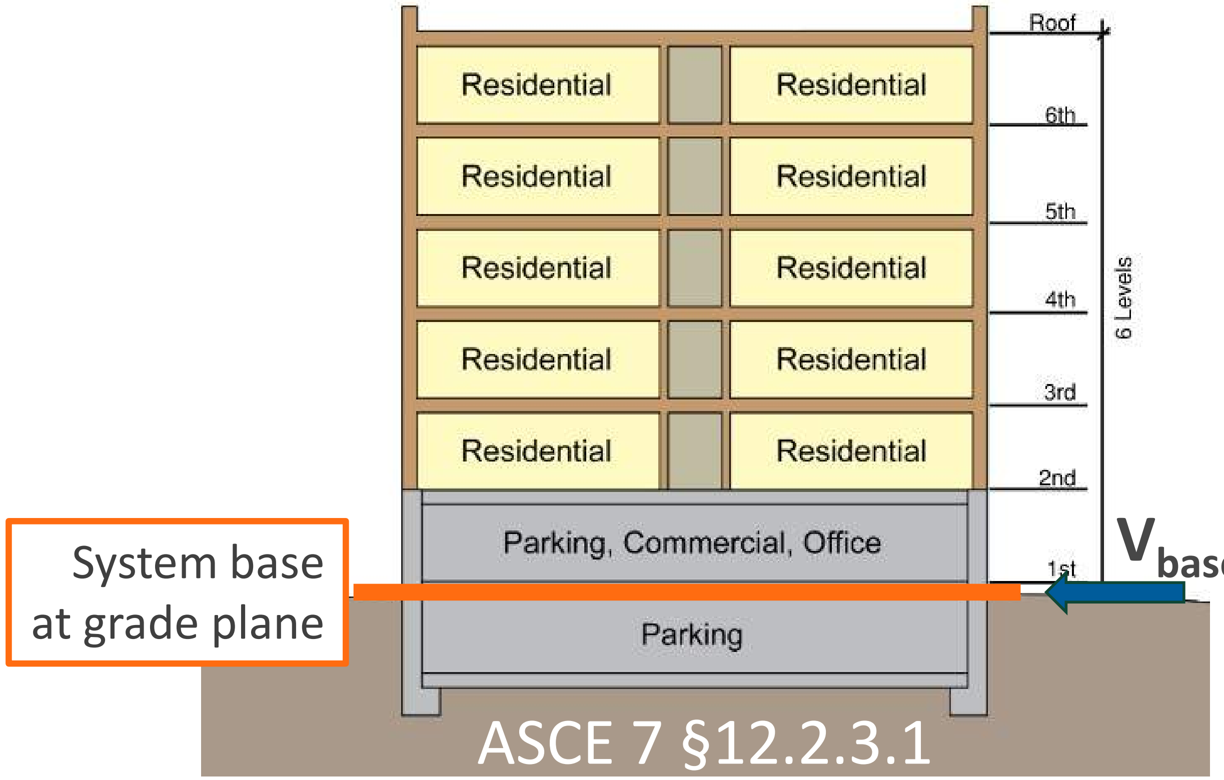
Wind vs. Seismic: What controls?

- Some things are wind controlled, and some things are seismic controlled, even in a wind region
- high wind/low seismic areas: still need to consider seismic forces in your design; even if nominal wind loads seem higher!
- With application of load and capacity factors, seismic can control certain aspects of the design.

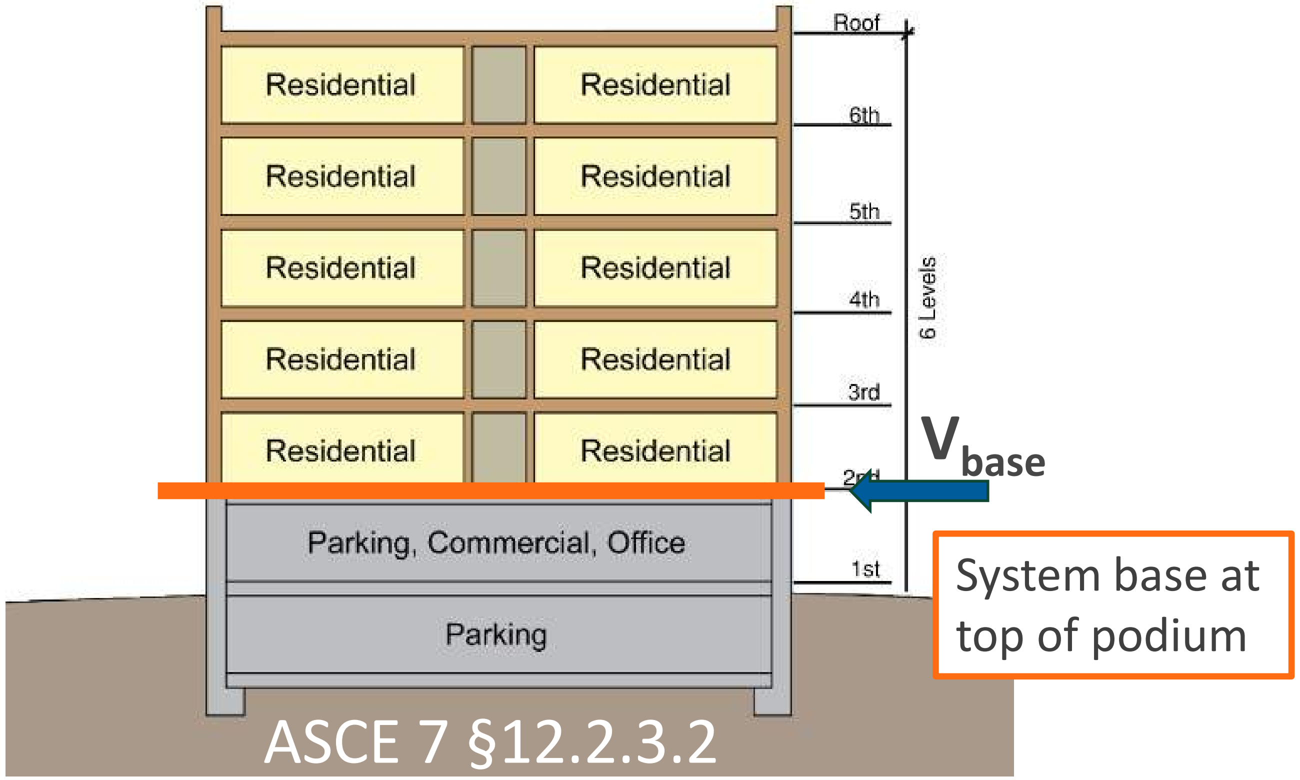
Wind vs. Seismic: What controls?

System Base

- To determine seismic demands, ASCE 7 Section 12.2.3.2 permits two-stage equivalent lateral force analysis of flexible upper portion and rigid lower portion
  - Does not apply to wind demands



WIND LOADING



SEISMIC LOADING

Design Example: Section 3.2

Wind vs. Seismic: What controls?

Load Factors

- ASD Load Factors:
  - Wind Load Combinations (ASCE 7 Section 2.4.1)

    5a. $1.0D + \underline{\mathbf{0.6W}}$

    6a. $1.0D + 0.75L + 0.75\underline{(\mathbf{0.6W})} + 0.75L_r$

    7a. $0.6D + \underline{\mathbf{0.6W}}$

  - Seismic Load Combinations (ASCE 7 Section 2.4.5):

    8. $1.0D + \underline{\mathbf{0.7E_v}} + \underline{\mathbf{0.7E_h}}$

    9. $1.0D + 0.75(\underline{\mathbf{0.7E_v}} + \underline{\mathbf{0.7E_h}}) + 0.75L = 1.0D + 0.525E_v + 0.525E_h + 0.75L$

    10. $0.6D - \underline{\mathbf{0.7E_v}} + \underline{\mathbf{0.7E_h}}$

Note: Combinations have been simplified based on the loads being considered in this example.

$D = Dead\ load$
$E = Earthquake\ load$
$E_h = Seismic\ horizontal$
$E_v = Seismic\ vertical$
$L = Live\ load$
$L_r = Roof\ live\ load$
$W = Wind\ load$

Wind vs. Seismic: What controls?

Capacity Adjustment Factors

- Seismic design (SDPWS Section 4.1.4.1):
  - Divide tabulated nominal shear capacity by 2.8 for ASD

$$v_{ASD} = v_{nom}/2.8$$

  - Multiply tabulated nominal shear capacity by 0.5 for LRFD

$$v_{LRFD} = v_{nom} * 0.50$$

- Wind design (SDPWS Section 4.1.4.2):
  - Divide tabulated nominal shear capacity by 2.0 for ASD

$$v_{ASD} = v_{nom}/2.0$$

  - Multiply tabulated nominal shear capacity by 0.8 for LRFD

$$v_{LRFD} = v_{nom} * 0.80$$

Note: Reduce capacities if not Douglas-Fir-Larch or Southern Pine.

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Directional (Ch 27) vs. Envelope (Ch 28) Procedure

- **Directional Procedure (Ch 27):** A procedure for determining wind loads on buildings and other structures for specific wind directions, in which the external pressure coefficients used are based on past wind tunnel testing of prototypical building models for the corresponding direction of wind.

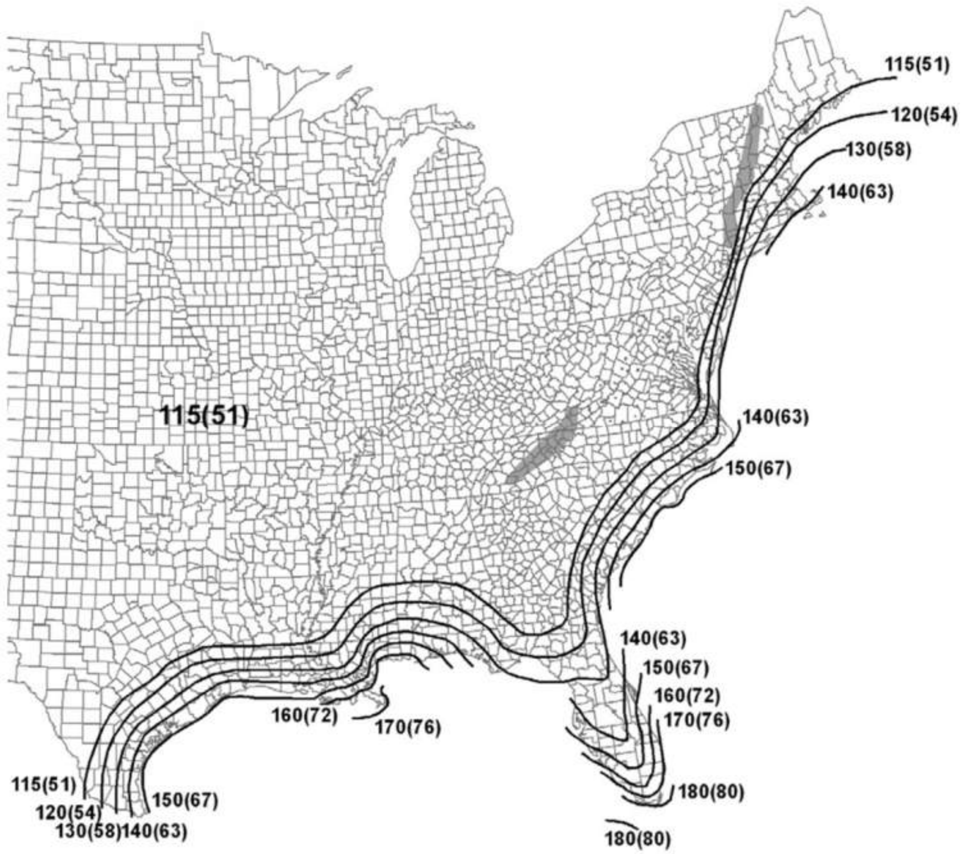
- **Envelope Procedure (Ch 28):** A procedure for determining wind load cases on buildings in which pseudoexternal pressure coefficients are derived from past wind tunnel testing of prototypical building models successively rotated through 360 degrees, such that the pseudopressure cases produce key structural actions (e.g., uplift, horizontal shear, and bending moments) that envelope their maximum values among all possible wind directions.

BUILDING, LOW-RISE: An enclosed, partially enclosed, or partially open building that complies with the following conditions:

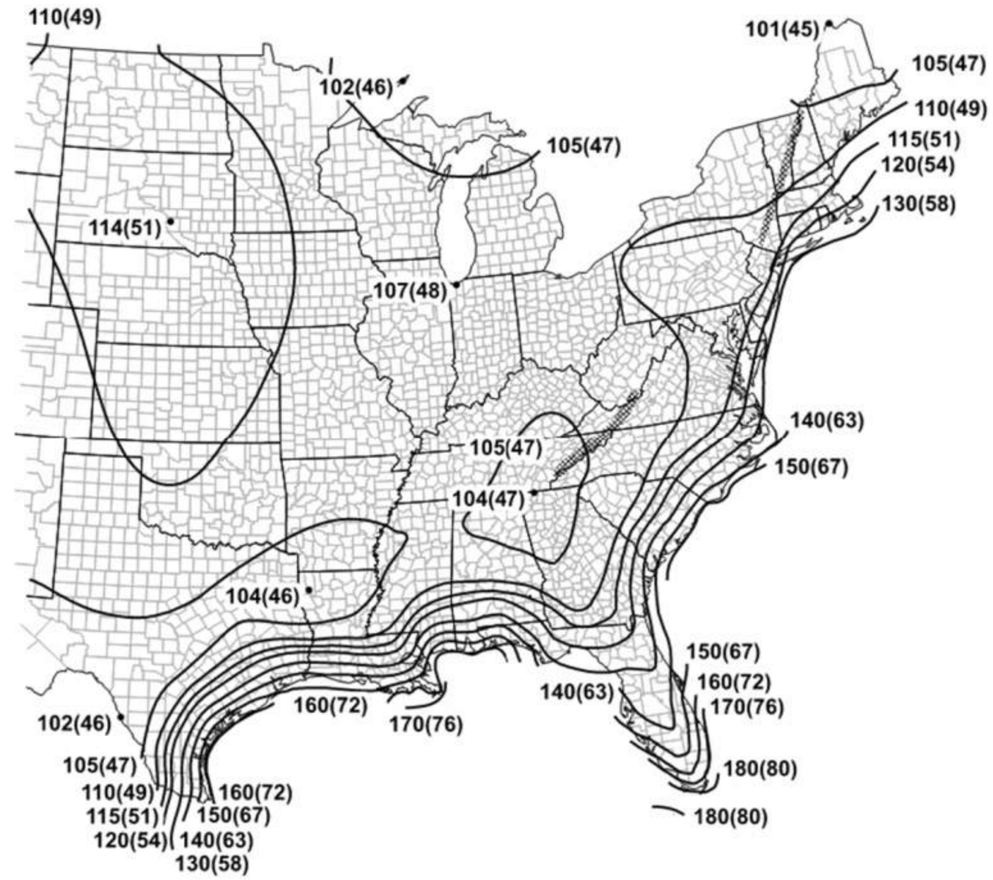
1. Mean roof height h less than or equal to 60 ft (18 m), and
2. Mean roof height h does not exceed the least horizontal dimension.

ASCE 7-22

Wind Speed Maps



ASCE 7-10, Fig. 26.5-1A



ASCE 7-22, Fig. 26.5-1B

Noteworthy Adjustment Factors

- Directionality factor, K_d (ASCE 7-22, §26.6)
  - No value change, just applied in a different equation in ASCE 7-22

$$q_z = 0.00256 K_z K_{zt} \mathbf{K_d} K_e V^2 \quad \begin{bmatrix}\text{ASCE 7} - 16 \\ \text{Eq. 26.10} - 1\end{bmatrix}$$

$$p = q \mathbf{K_d} G C_p - q_i \mathbf{K_d} (G C_{pi}) \quad \begin{bmatrix}\text{ASCE 7} - 22 \\ \text{Eq. 27.3} - 1\end{bmatrix}$$

- Ground Elevation Factor, K_e (ASCE 7-22, §26.9)
  - Introduced in ASCE 7-16 and worth taking advantage of. Projects at higher elevations can see significant ***decreases*** in design wind pressures
  - Permitted to be taken as 1.0 for all cases
  - More than 70% of the lower 48 is 500ft above sea level
    - K_e = 0.982 @ Elevation of 500 ft

$$q_z = 0.00256 K_z K_{zt} \mathbf{K_e} V^2 \quad \begin{bmatrix}\text{ASCE 7} - 22 \\ \text{Eq. 26.10} - 1\end{bmatrix}$$

Table 26.9-1. Ground Elevation Factor, K_e.

Ground Elevation above Sea Level		Ground Elevation Factor, K_e
ft	m	
<0	<0	See note 2
0	0	1.00
1,000	305	0.96
2,000	610	0.93
3,000	914	0.90
4,000	1,219	0.86
5,000	1,524	0.83
6,000	1,829	0.80
>6,000	>1,829	See note 2

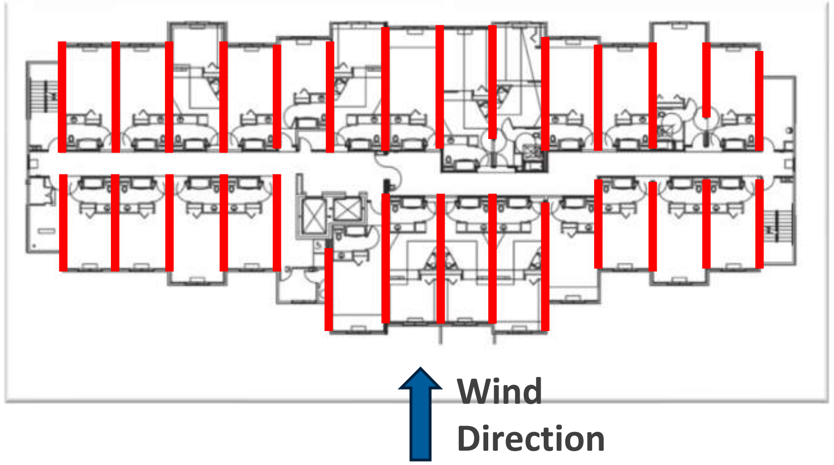
Design Example: Section 4.3

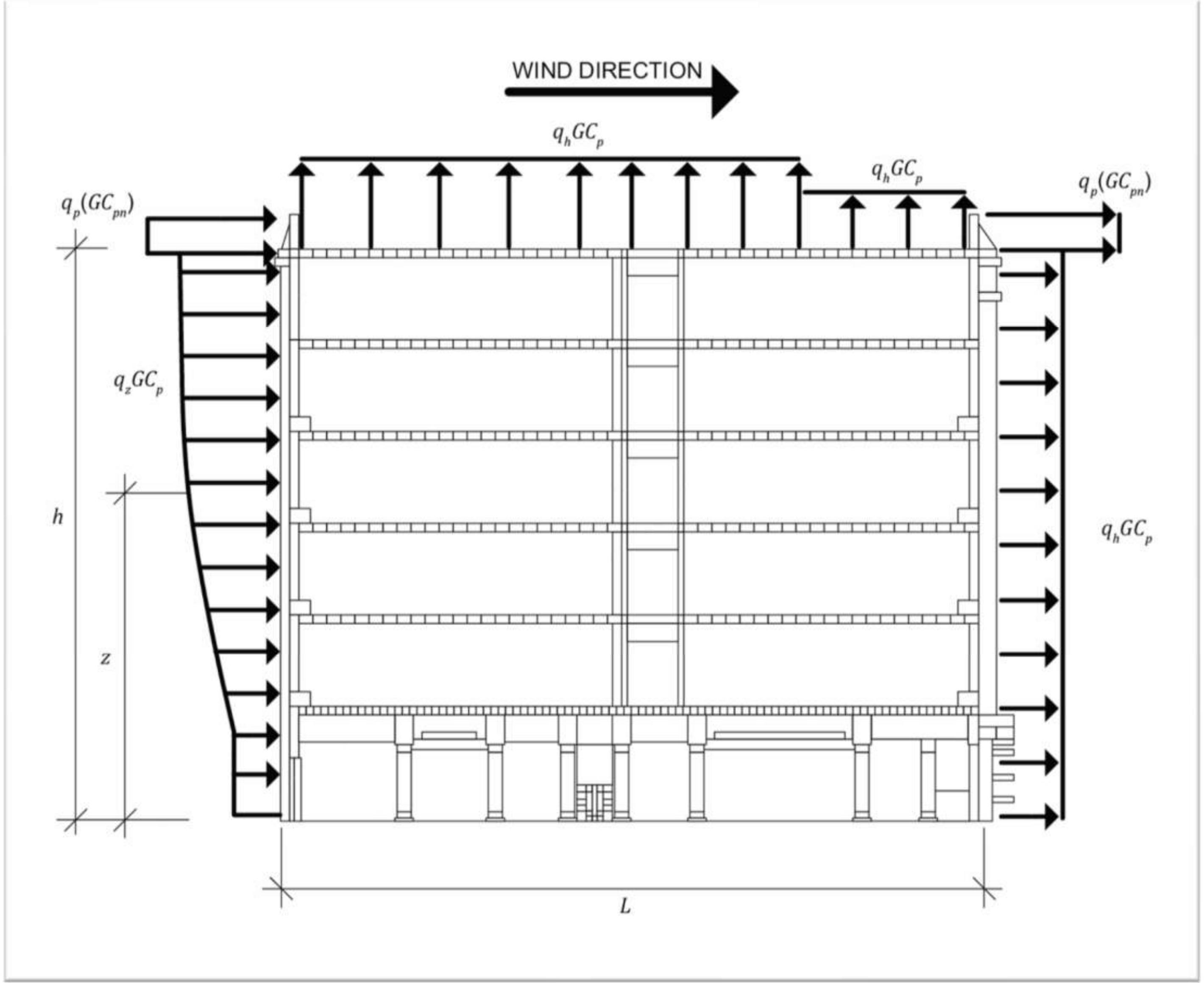
Design Example Parameters – Providence, RI

- Risk Category II *(ASCE 7-22, §1.5)*
- Basic Wind Speed, V = 120 mph *(ASCE 7-22, §26.5)*
- Directionality factor, K_d = 0.85 *(ASCE 7-22, §26.6)*
- Exposure category = C *(ASCE 7-22, §26.7)*
- Topographic factor, K_{zt} = 1.00 *(ASCE 7-22, §26.8)*
- Ground elevation factor, K_e = 1.00 *(ASCE 7-22, §26.9)*
- Velocity pressure exposure coefficient, K_z = Varies *(ASCE 7-22, §26.10.1)*
- Gust-effect factor, G = 0.85 *(ASCE 7-22, §26.11)*
- Enclosure classification = Enclosed *(ASCE 7-22, §26.12)*
- Internal pressure coefficient, (GC_{pi}) = ± 0.18 *(ASCE 7-22, §26.13)*

Developing the Wind Pressures

- Need to develop pressures for:
  - Windward wall
  - Leeward wall
  - Roof
  - Parapets





Design Example: Figure 4.1

Developing the Wind Pressures

Walls & Roof

$$p = qK_dGC_p - q_iK_d(GC_{pi}) \quad \begin{bmatrix}\text{ASCE 7} - 22 \\ \text{Eq. } 27.3 - 1\end{bmatrix}$$

- » Windward walls $q = q_z$
- » Leeward walls & Roof $q = q_h$

- » Windward wall C_p = 0.80
- » Leeward wall C_p = -0.28
- » Roof C_p = varies

$\begin{bmatrix}\text{ASCE 7} - 22 \\ \text{Fig. } 27.3 - 1\end{bmatrix}$

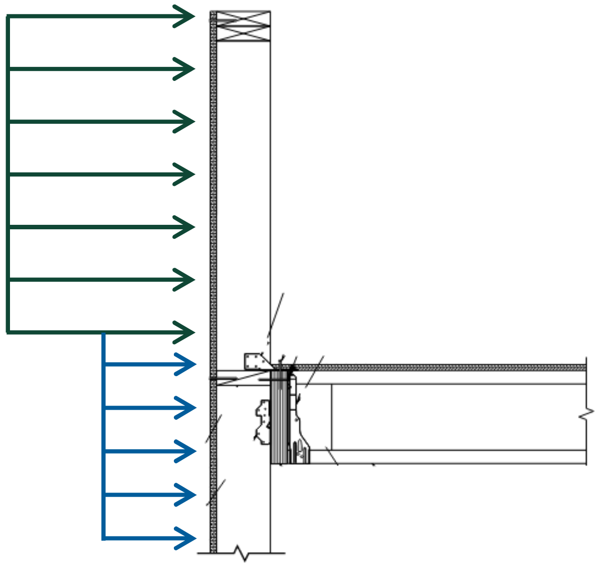
Hz. Dist. from Windward Edge	Roof C_p Transverse Direction		
	0 to $h/2$	$h/2$ to h	h to $2h$
Case a	-1.15	-0.77	-0.63
Case b	-0.18	-0.18	-0.18

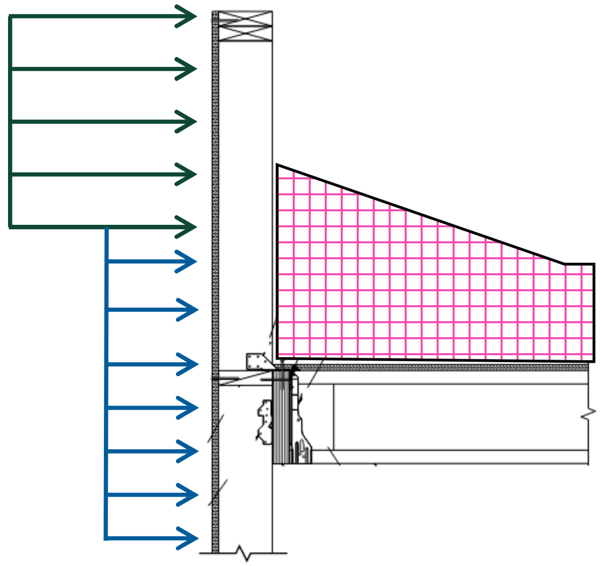
- » Case (a) A positive value of (GC_{pi}) applied to all internal surfaces, or
- » Case (b) A negative value of (GC_{pi}) applied to all internal surfaces

Parapets

$$p_p = q_pK_d(GC_{pn}) \quad \begin{bmatrix}\text{ASCE 7} - 22 \\ \text{Eq. } 27.3 - 3\end{bmatrix}$$

- » (GC_{pn}) = +1.5 for windward parapet
  -1.0 for leeward parapet

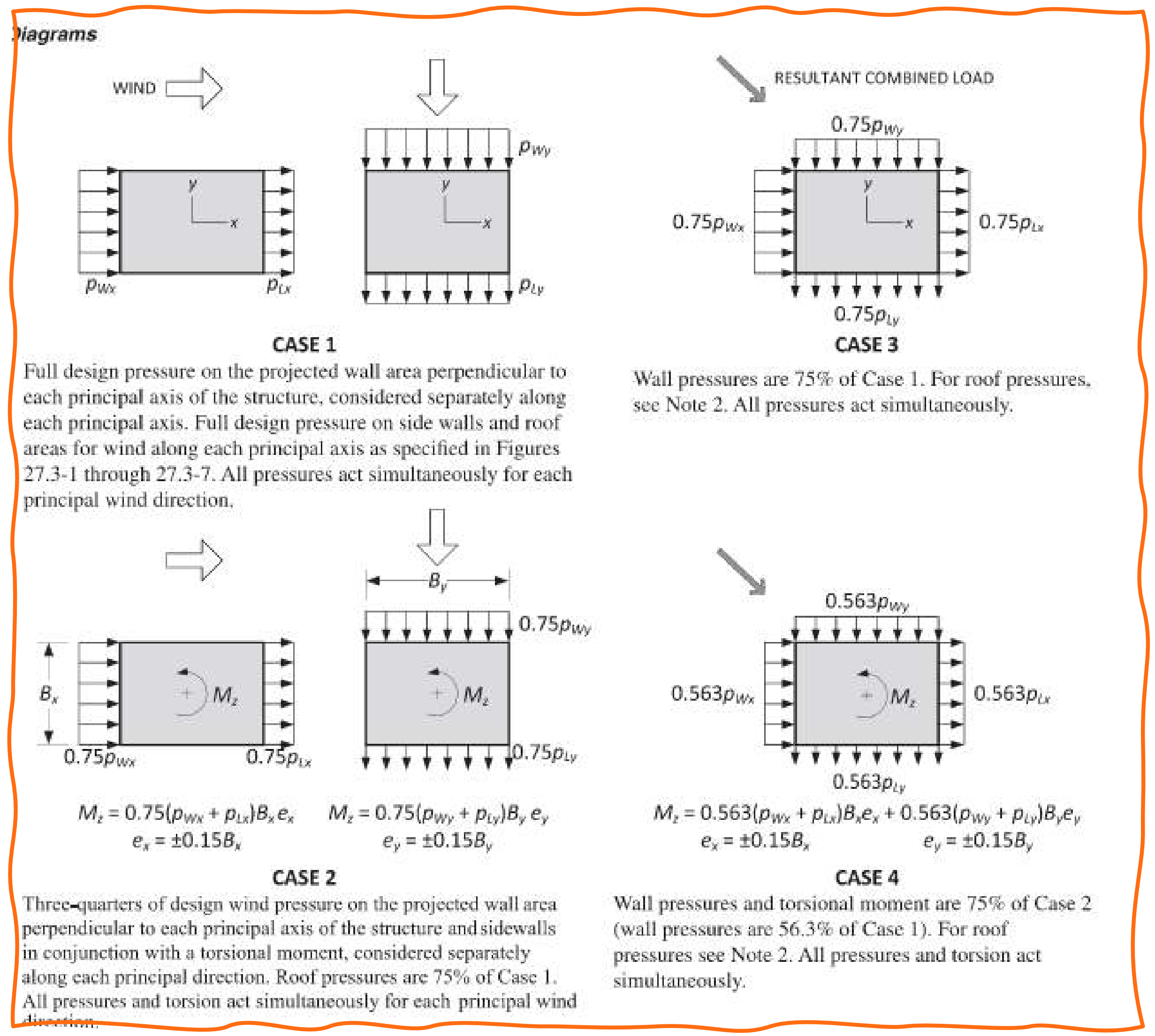




Design Example: Section 4.3.1

Applied Load Cases

- Design example runs through Case 1 (orthogonal loading)
- Designers should consider applicable cases on each project
  - Dependent on diaphragm classification (rigid vs. flexible)
  - May be able to apply exclusions in Appendix D



ASCE 7-22, Fig. 27.3-8

Design Example: Section 4.5

Design Example Wind Pressures

Level	Windward Wall Pressure					Leeward Wall Pressure					Total
	z (ft)	K_z	q_z (psf)	p Case a (psf)	p Case b (psf)	h_x (ft)	K_h	q_h (psf)	p Case a (psf)	p Case b (psf)	p (psf)
Parapet	65	1.15	42.3	54.0		65	1.15	42.3	-36.0		**89.9**
Roof	62	1.14	41.9	30.6	17.8	62	1.14	41.9	-8.7	-21.6	**39.4**
6th	52	1.10	40.4	29.8	17.0	62	1.14	41.9	-8.7	-21.6	**38.5**
5th	42	1.05	38.7	28.8	16.0	62	1.14	41.9	-8.7	-21.6	**37.5**
4th	32	0.99	36.6	27.6	14.8	62	1.14	41.9	-8.7	-21.6	**36.3**
3rd	22	0.92	33.9	26.0	13.2	62	1.14	41.9	-8.7	-21.6	**34.8**
2nd	12	0.85	31.4	24.5	11.7	62	1.14	41.9	-8.7	-21.6	**33.3**
Base	0	0.85	31.4	24.5	11.7	62	1.14	41.9	-8.7	-21.6	**33.3**

	Flat Roof Wind Pressure (psf)			
Horizontal Dist. from Windward Edge	**0 to h/2**	**h/2 to h**	**h to 2h**	**> 2h**
Case a	-41.3	-29.8	-25.4	-23.1
Case b	1.0	1.0	1.0	1.0

Design Example: Table 4.2A

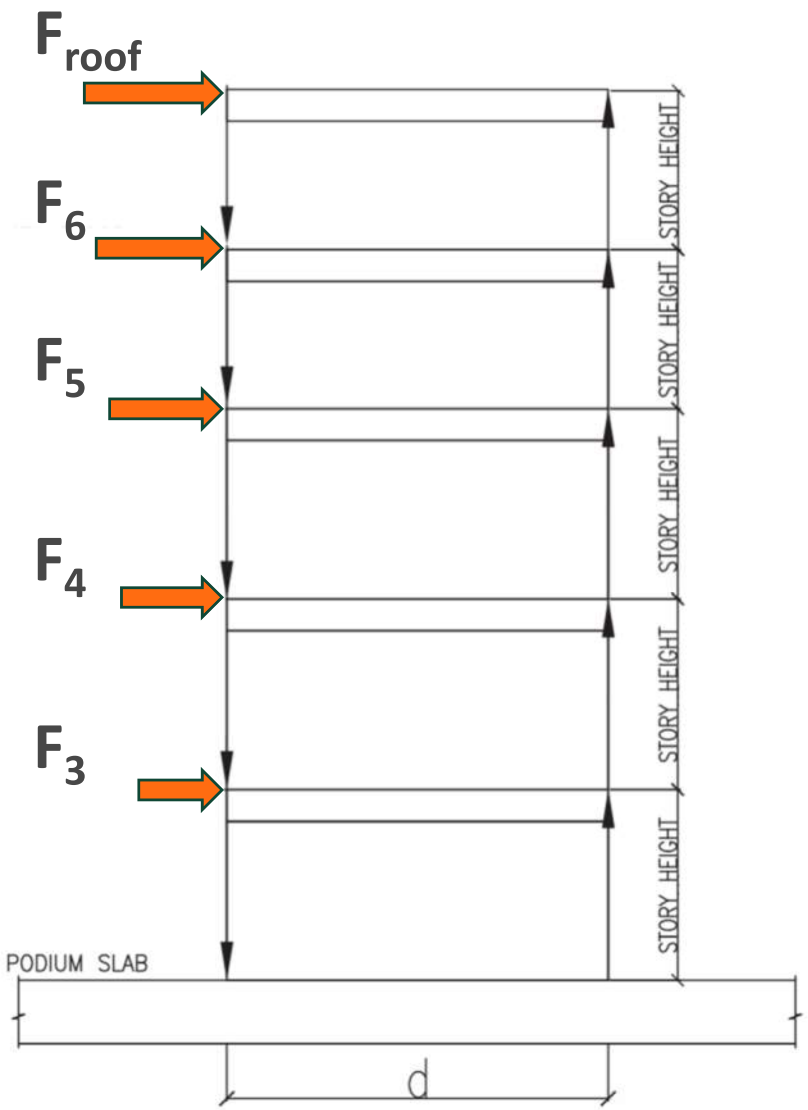
Design Example: Wind Forces to Shear Wall

Providence, RI (02901): high-wind, low-seismic site

» Story-Level Shear Wall Force, $F = p * A_{trib}$

Table 4.4: Wind forces to shear wall

Level	p (psf)	Tributary Area, A_{trib} (ft^2)	Story-Level Force, F (lb)	Cumulative Force, F_{total} (lb)	ASD Force, $0.6F_{total}$ (lb)
Parapet	89.9	39	3,508	--	--
Roof	39.4	65	2,559	6,067	3,640
6th	38.5	130	5,007	11,074	6,644
5th	37.5	130	4,878	15,951	9,571
4th	36.3	130	4,720	20,672	12,403
3rd	34.8	130	4,518	25,189	15,114



Design Example: Section 4.5

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- ➤ Seismic Considerations & Loads
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GWB vs. WSP Shear Wall Sheathing

Table 4.1: Response modification coefficients for light-frame seismic force-resisting systems

Seismic Force-Resisting System	Response Modification Coefficient, R	Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d
A.16 – Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	6.5	3*	4
A.18 – Light-frame walls with shear panels of all other materials	2	2.5	2

Partial ASCE 7 Table 12.2-1, *Reduced to 2.5 for flexible diaphragms

GWB vs. WSP Shear Wall Sheathing

Table 4.1: Response modification coefficients for light-frame seismic force-resisting systems

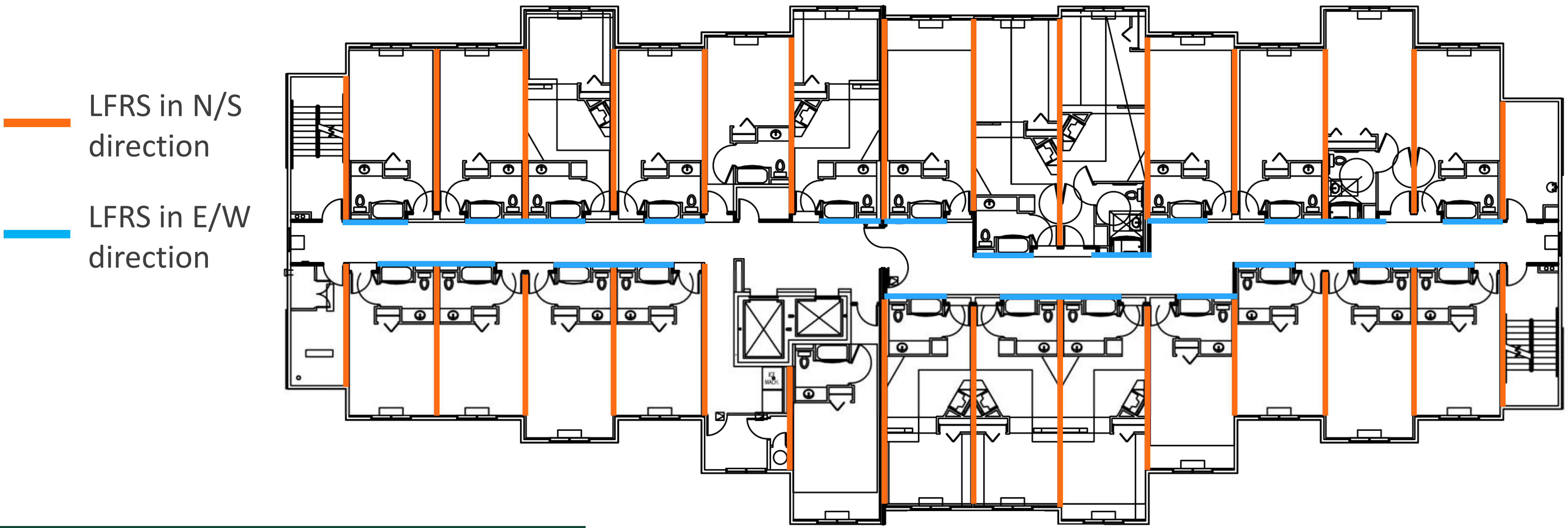
Seismic Force-Resisting System	System Limitations		
	Seismic Design Category		
	A, B, or C	D (ft)	E or F (ft)
A.16 – Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	No Limit	65	65
A.18 – Light-frame walls with shear panels of all other materials	No Limit	35	Not permitted

Partial ASCE 7 Table 12.2-1

Design Example: Section 4.2

GWB vs. WSP Shear Wall Sheathing

» ASCE 7 Section 12.2.2 permits the use of different framing systems in different directions, which is common on projects where combined orthogonal effects are not required as part of the analysis, per ASCE 7 Section 12.5.1.2.



Design Example: Section 4.1 & Figure 1.2

Design Example: Seismic Design Data

Providence, RI (02901): high-wind, low-seismic site

- Site Class: D
- $S_S = 0.210$ g
- $S_1 = 0.048$ g
- $S_{DS} = 0.1673$ g
- $S_{D1} = 0.0667$ g

- Risk Category II
- Seismic importance factor, $I_e = 1.0$
- **Seismic Design Category = B**

- Response Modification Coefficient,
  $R = 2.0$
- Seismic Response Coefficient,
  $C_S = 0.075$
- Seismic weight, $W = 2{,}460\ kips$
- Base Shear, $V = C_S * W$
  $V = 0.075 * 2{,}460\ kips = \mathbf{185\ kips}$

Design Example: Seismic Story Forces

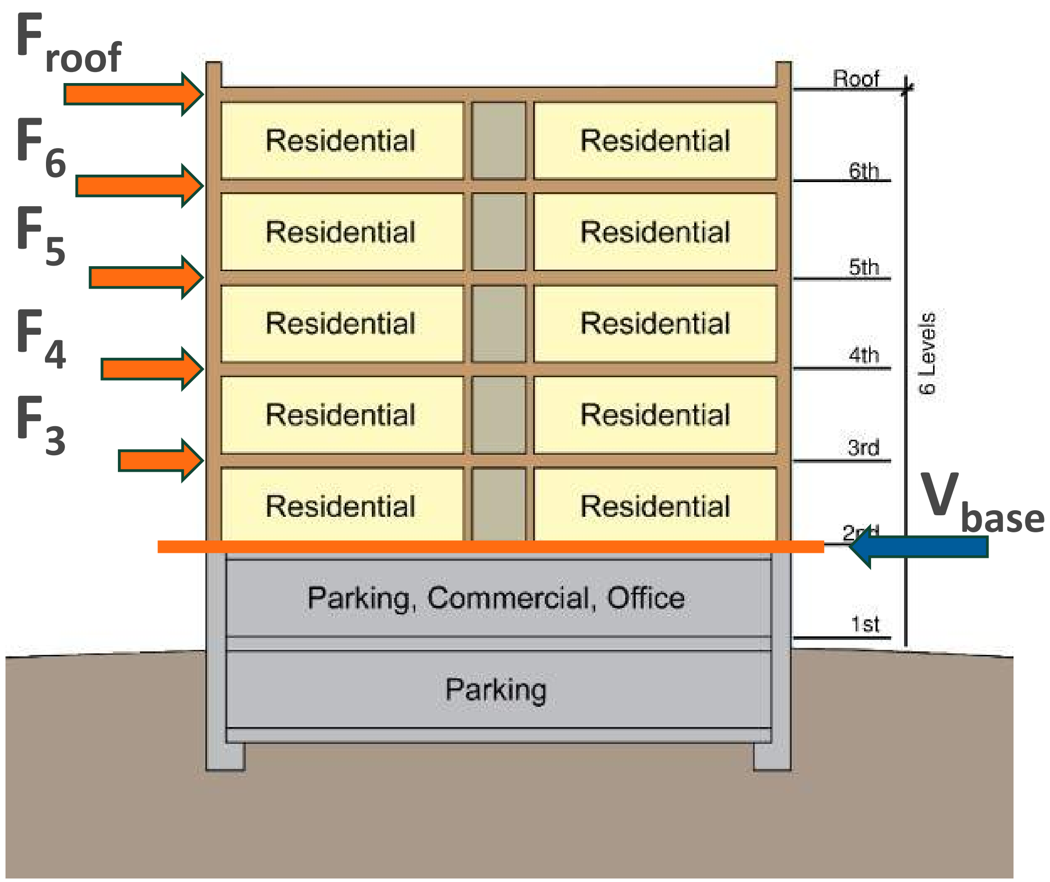
Providence, RI (02901): high-wind, low-seismic site

» Story Force, $F_x = C_{vx} * V$

Table 4.3: Vertical distribution of seismic forces

Level	W_x (kips)	C_{vx}	F_x (kips)	F_x/A (psf)
Roof	420	29%	54.0	4.5
6th	510	28%	52.4	4.4
5th	510	21%	39.4	3.3
4th	510	14%	26.3	2.2
3rd	510	7%	13.1	1.1
Sum	2,460	100%	185	

A = area of the floor plate, which is 12,000 ft^2



Design Example: Section 4.4.1

Design Example: Seismic Forces to Shear Wall

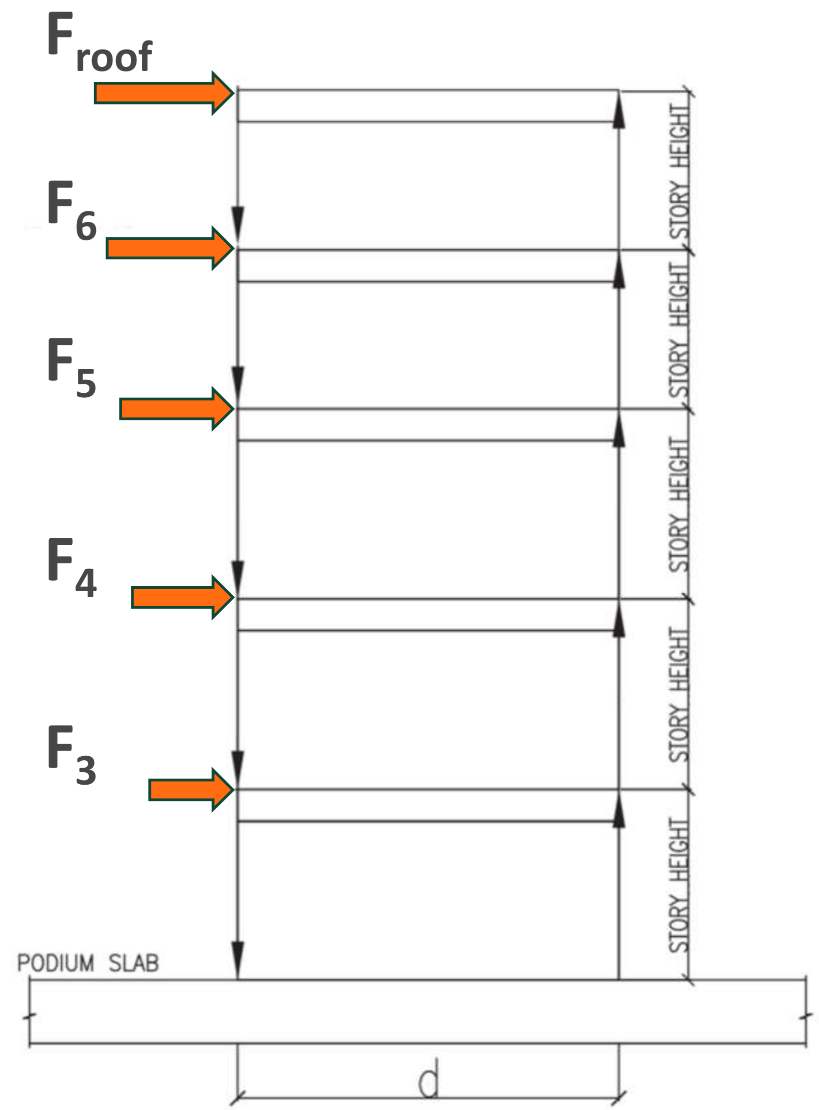
Providence, RI (02901): high-wind, low-seismic site

» Story-Level Shear Wall Force, $F = \frac{F_x}{A} * A_{trib}$

Table 4.4: Seismic forces to shear wall

Level	F_x/A (psf)	Tributary Area, A_{trib} (ft²)	Story-Level Force, F (lb)	Cumulative Force, F_{total} (lb)	ASD Force, $0.7F_{total}$ (lb)
Roof	4.5	845	3,804	3,804	2,663
6th	4.4	845	3,687	7,491	5,243
5th	3.3	845	2,775	10,265	7,186
4th	2.2	845	1,850	12,115	8,481
3rd	1.1	845	925	13,040	9,128

Tributary Area, A_{trib} (ft²) is as described in Section 3.3.3



Outline

- » Design Example Overview
- » Wind vs. Seismic: What Controls?
- » Wind Considerations & Loads
- » Seismic Considerations & Loads
- Shear Wall Sheathing Selection: GWB vs. WSP
- » Design Example Comparison: Wind vs. Seismic

WSP Sheathing: Blocked vs. Unblocked

- Unblocked WSP Shear Wall Design (SDPWS Section 4.3.5.3):
  - Unblocked design permitted with required reduction factor per SDPWS Table 4.3.5.3

$$v_{nom(ub)} = v_{nom(b)} * C_{(ub)}$$ (SDPWS Equation 4.3-2)

Table 4.3.5.3 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

Source: SDPWS (AWC, 2021)

$v_{nom(ub)}$ = nominal unit shear capacity for unblocked shear wall

$v_{nom(b)}$ = nominal unit shear capacity from Table 4.3A for WSP blocked shear wall with 24" stud spacing and nails spaced at 6" O.C. at panel edges

$C_{(ub)}$ = unblocked shear wall adjustment factor per Table 4.3.5.3

Design Example: Section 4.5.1

GWB Sheathing: Blocked vs. Unblocked

- Blocked and unblocked GWB shear wall nominal unit shear capacities provided in SDPWS Table 4.3C:

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

Gypsum Board, Gypsum Lath and Plaster, and Portland Cement Plaster

Sheathing Material	Material Thickness	Fastener Type & Size [2]	Max. Fastener Spacing [3] (in.)	Max. Stud Spacing (in.)		v_n (plf)	G_a (kips/in.)
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
			4	24	unblocked	220	6.0
			7	16	unblocked	200	5.5
			4	16	unblocked	250	6.5
			7	16	blocked	250	6.5
			4	16	blocked	300	7.5
		No. 6 Type S or W drywall screws 1-1/4" long	8/12	16	unblocked	120	3.5
			4/16	16	blocked	320	8.0
			4/12	24	blocked	310	8.0
			8/12	16	blocked	140	4.0
			6/12	16	blocked	180	5.0
	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
			4	24	unblocked	290	7.5
			7	16	blocked	290	7.5
			4	16	blocked	350	8.5
		No. 6 Type S or W drywall screws 1-1/4" long	8/12	16	unblocked	140	4.0
			8/12	16	blocked	180	5.0

Source: SDPWS (AWC, 2021)

Design Example: Table 4.5

GWB Sheathing: Cooler Nails vs. Drywall Screws

	Cooler Nails	Drywall Screws
Prevalence today	Uncommon	Common
Installation method	Typically installed by hand using a hammer	Typically installed using a high-speed screw gun set to specific depth
Installation speed	Slower and inconsistent depth	Faster and consistent depth
Lateral force-resistance capacity	Higher—fasteners bend with wall movement but do not “cut” as large of holes in drywall material	Lower—fasteners bend with wall movement and threads tend to “cut” larger holes in drywall material
		For design, recommended that the design engineer use values for screw-fastened GWB

Design Example: Section 4.2.1

Outline

- Design Example Overview
- Wind vs. Seismic: What Controls?
- Wind Considerations & Loads
- Seismic Considerations & Loads
- Shear Wall Sheathing Selection: GWB vs. WSP
- **Design Example Comparison: Wind vs. Seismic**

Providence, RI (02901): high-wind, low-seismic site

Design Example: Wind vs. Seismic

Table 4.5: Forces and demands to shear wall

Level	Load Type	Cumulative ASD Force, F_{total} (lb)	Wall Length (ft)	ASD Design Shear, V (plf)	Wall Sheathed 1 or 2 Sides	Demand per side (plf)
Roof	Seismic	2,663	58	46	2	23
	Wind	3,640		63	2	32
6th	Seismic	5,243	58	90	2	45
	Wind	6,644		115	2	58
5th	Seismic	7,186	58	124	1	124
	Wind	9,571		166	1	166
4th	Seismic	8,481	58	147	1	147
	Wind	12,403		214	1	214
3rd	Seismic	9,128	58	158	1	158
	Wind	15,114		261	1	261

a. The wall length has two 29.0-foot-long walls in the same line for a total of 58 feet.

Design Example: Section 4.5.1

Providence, RI (02901): high-wind, low-seismic site

Design Example: Wind vs. Seismic

Table 4.5: Demands to shear wall and sheathing

Level	Load Type	Wall Sheathed 1 or 2 Sides	Demand per side (plf)	Sheathing Used	Fastener Spacing	Blocking	Adjusted Allowable Shear Capacity (plf)
Roof	Seismic	2	23	5/8” GWB	8/12	Unblocked	50
	Wind	2	32				70
6th	Seismic	2	45	5/8” GWB	8/12	Unblocked	50
	Wind	2	58				70
5th	Seismic	1	124	7/16” WSP - Sheathing	6/12	Unblocked	156
	Wind	1	166				219
4th	Seismic	1	147	7/16” WSP - Sheathing	6/6	Unblocked	209
	Wind	1	214				292
3rd	Seismic	1	158	7/16” WSP - Sheathing	6/12	Blocked	261
	Wind	1	261				365

Providence, RI (02901): high-wind, low-seismic site

Design Example: Wind vs. Seismic

Table 4.5: Demands to shear wall and sheathing

Level	Load Type	Demand per side (plf)	Sheathing Used	Fastener Spacing	Blocking	Adjusted Allowable Shear Capacity (plf)	D:C Ratio
Roof	**Seismic**	23	5/8” GWB	8/12	Unblocked	50	**0.46**
	Wind	32				70	**0.46**
6th	**Seismic**	45	5/8” GWB	8/12	Unblocked	50	**0.90**
	Wind	58				70	0.83

a. Fastener spacing is listed as two numbers (“X/X”). The first number is the fastener spacing at the panel edges and the second number is the fastener spacing along intermediate (field) members.

Providence, RI (02901): high-wind, low-seismic site

Design Example: Wind vs. Seismic

Table 4.5: Demands to shear wall and sheathing

Level	Load Type	Demand per side (plf)	Sheathing Used	Fastener Spacing	Blocking	Adjusted Allowable Shear Capacity (plf)	D:C Ratio
5th	**Seismic**	124	7/16" WSP - Sheathing	6/12	Unblocked	156	**0.80**
	Wind	166				219	0.76
4th	Seismic	147	7/16" WSP - Sheathing	6/6	Unblocked	209	0.70
	Wind	214				292	**0.73**
3rd	Seismic	158	7/16" WSP - Sheathing	6/12	Blocked	261	0.61
	Wind	261				365	**0.72**

a. Fastener spacing is listed as two numbers ("X/X"). The first number is the fastener spacing at the panel edges and the second number is the fastener spacing along intermediate (field) members.
b. Per SDPWS Table 4.3A, footnote 2, nominal unit shear capacities are permitted to be increased to values shown for 15/32-inch nominal sheathing if studs are spaced at 16 inches O.C. or the panels are applied with the long dimension across the stud.
c. Unblocked WSP shear walls have been adjusted based on SDPWS Table 4.3.5.3.

Design Example: Seismic Design Data

Providence, RI (02901): high-wind, low-seismic site

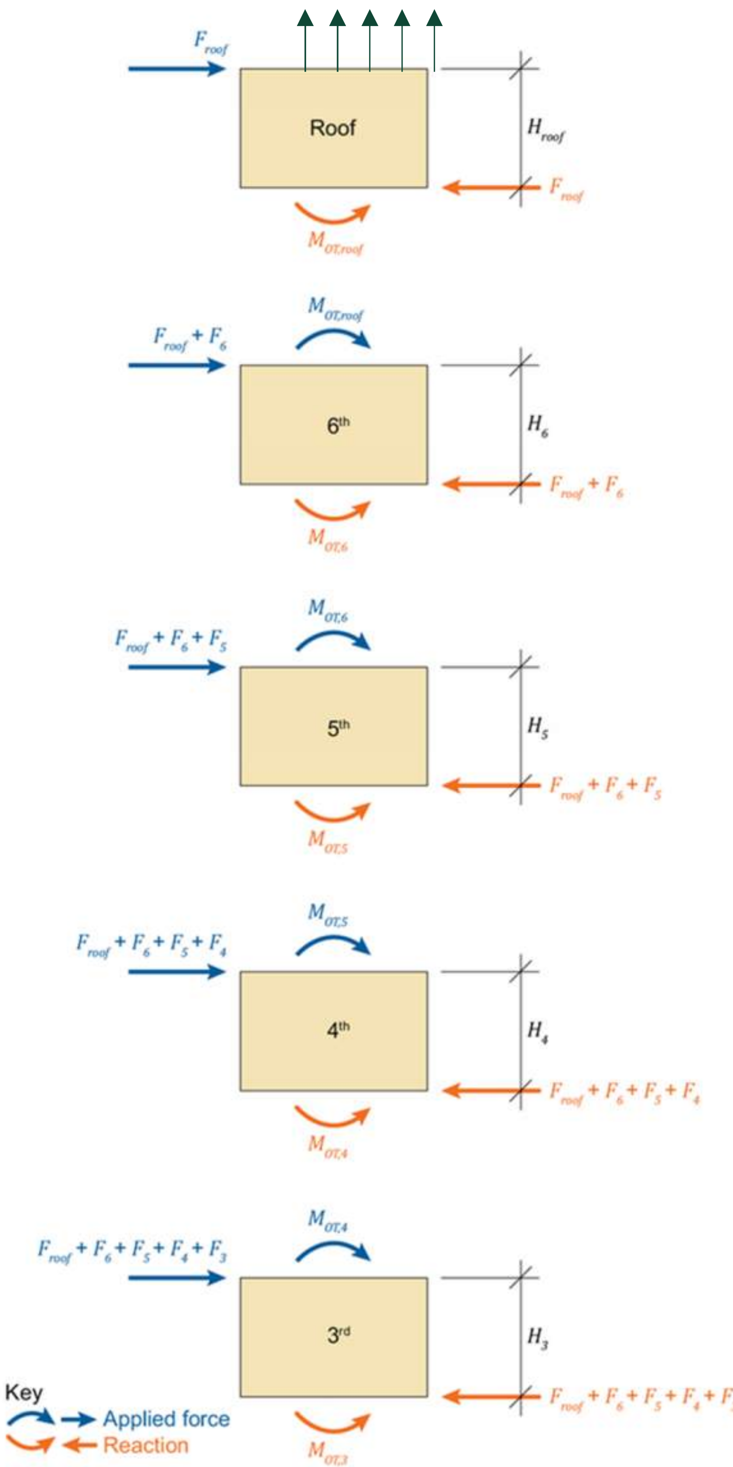
- Gypsum Wallboard Shearwall Sheathing:
  - Response Modification Coefficient, $R = 2.0$
  - Seismic Response Coefficient, $C_S = 0.075$
  - Seismic weight, $W = 2{,}460\ kips$
  - Base Shear, $V = C_S * W = \mathbf{185\ kips}$
- Wood Structural Panel Shearwall Sheathing:
  - Response Modification Coefficient, $R = 6.5$
  - Seismic Response Coefficient, $C_S = 0.023$
  - Seismic weight, $W = 2{,}460\ kips$
  - Base Shear, $V = C_S * W = \mathbf{57\ kips}$

Seismic force for GWB system is 225% greater than seismic force for WSP system

Design Example: Section 4.2

Chord Forces

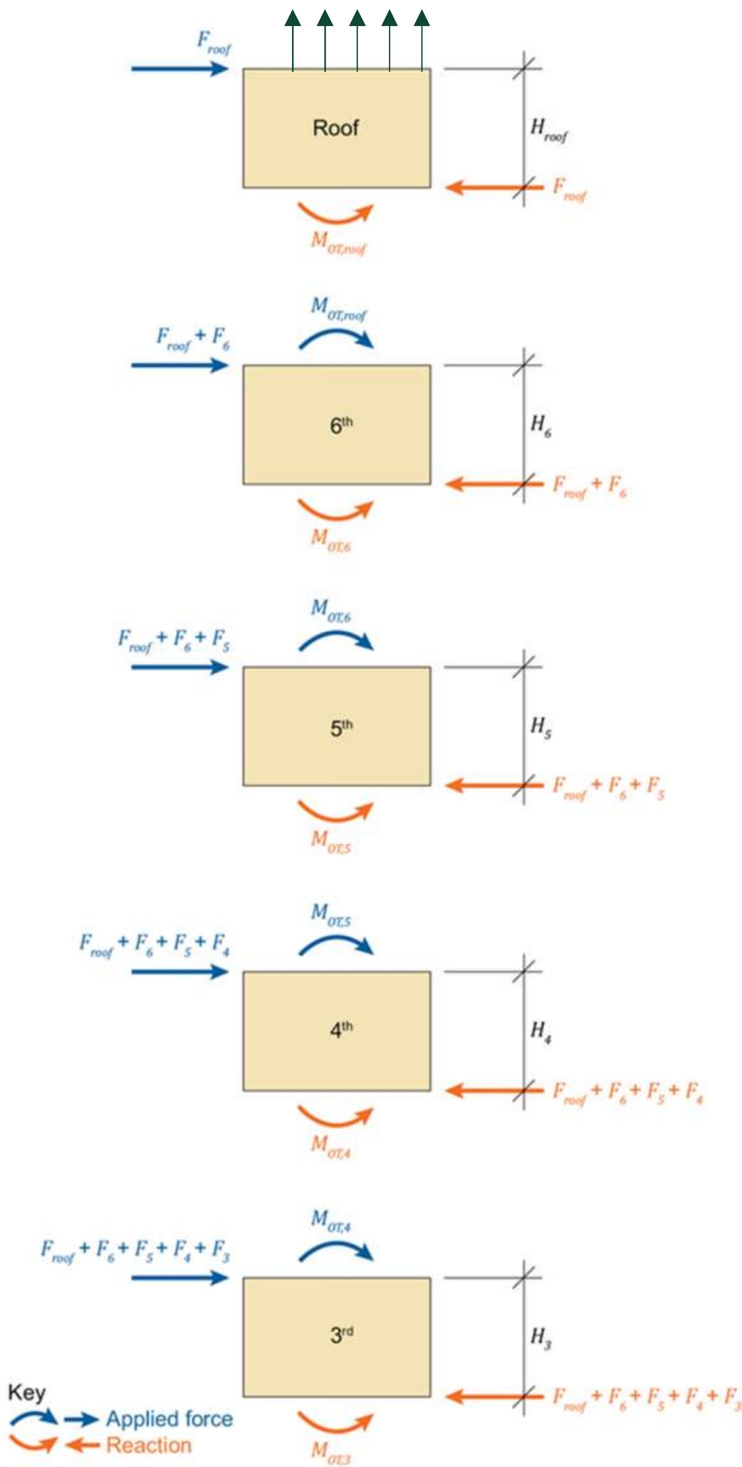
Table 4.6 Overturning moments and T/C couples					
Level	Shear Wall Story Force (lb)	Story Height (ft)	Cumulative M_{OT} (ft-k)	d (ft)	Force Couple +/- (k)
Roof	3,033	10	65	28.75	2.3
6th	2,504	10	120	28.75	4.2
5th	2,439	10	200	28.63	7.0
4th	2,360	10	304	28.50	10.7
3rd	2,259	10	430	28.38	15.1



Design Example: Sections 4.5.2 & 4.5.3 and Figure 3.6

Chord Forces

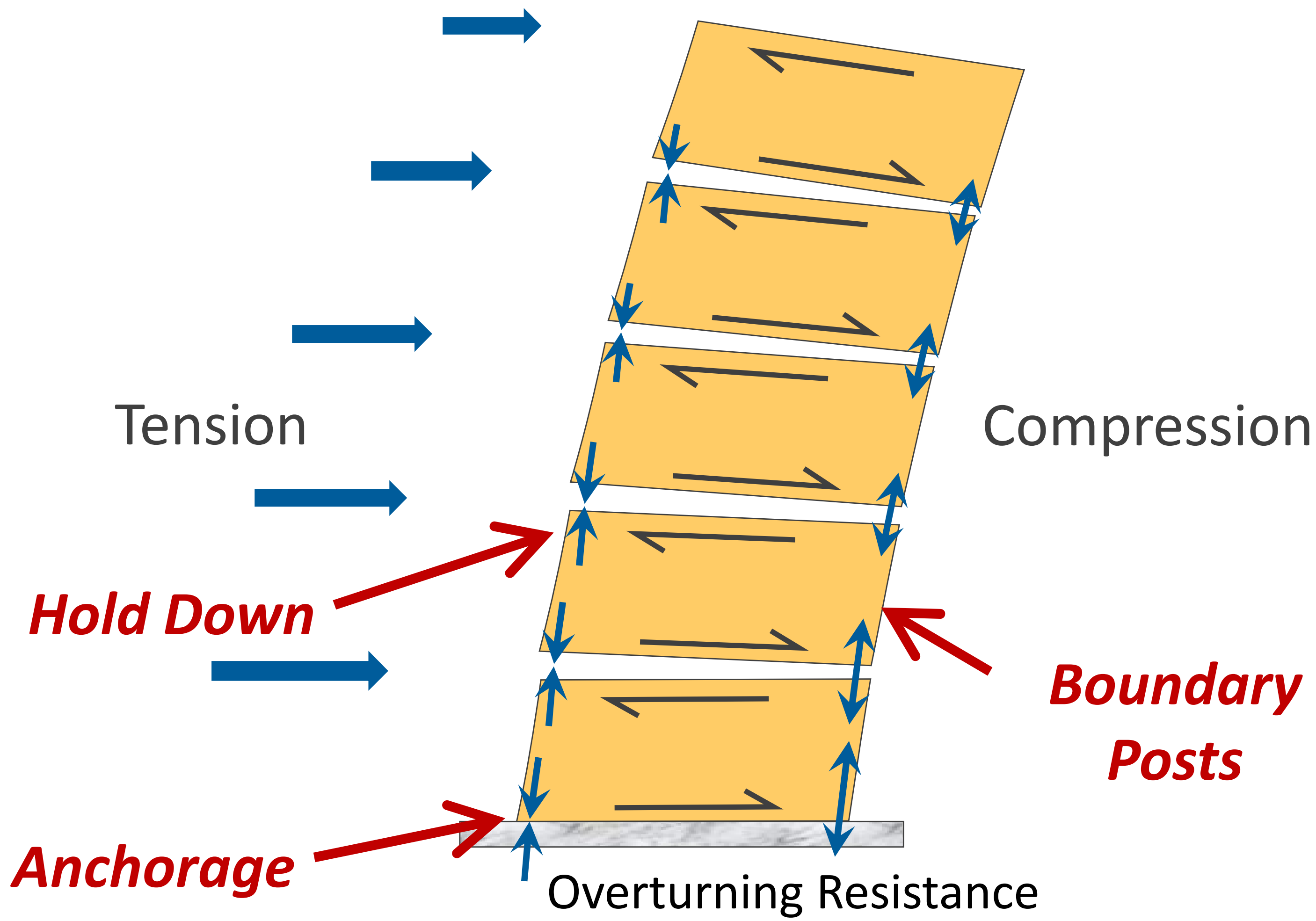
	Table 4.7 Compression Design						Table 4.9 Tension Design			
Level	Total Chord Length (ft)	P_D (k)	P_L (k)	P_{Lr} (k)	Total Comp. Demand (Load Combo 5a) 1.0D + 0.6W (k)	Chord Posts	Cumulative Resisting Moment (Dead Load) M_R (ft-k)	Cumulative M_{OT} (Wind) from Table 4.6 (ft-k)	d (ft)	Net Resultant Tension Force 0.6D + 0.6W
Roof	0.67	0.10	0.00	0.03	1.46	(2) 2x4	65	-65	28.75	0
6th	0.67	0.43	0.35	0.03	2.94	(2) 2x4	271	-120	28.75	0
5th	0.67	0.76	0.69	0.03	4.95	(3) 2x4	477	-200	28.63	0
4th	0.67	1.08	1.04	0.03	7.47	(4) 2x4	683	-304	28.50	0
3rd	0.67	1.41	1.39	0.03	10.49	(5) 2x4	889	-430	28.38	0



Design Example: Sections 4.5.2 & 4.5.3 and Figure 3.6

Shear Wall Deformations and Story Drift

- Even where wind loads govern the design, seismic drift requirements in Chapter 12 of ASCE 7 still need to be met
- No explicit criteria for wind loads within IBC or ASCE 7
- Limitations for wind loads are typically governed by serviceability
  - For example, occupant comfort and finish requirements



Shear Wall Deformations and Story Drift

- Considerations when evaluating drift from wind loads:
  - *IBC wind deflection criteria* – IBC Table 1604.3 (footnote f) permits using the 10-year mean return interval basic wind speed.
  - *ASCE 7 appendix C code commentary (CC)* – ASCE 7 does not mandate a specific allowable story drift for wind but provides non-mandatory serviceability guidelines in its commentary. Drift limits commonly used in practice range from about H/600 to H/400 of the building or story height (0.17% – 0.25% drift per story).
  - *Griffis Method* – Griffis Method for Evaluating Wind Drift (AISC, 1993), which is referenced in the ASCE 7 commentary, uses a reduced wind load of 0.42*W* (equivalent to a 10-year event) or a representative return period for a given wind event. The Griffis paper also provides discussion on perceived motion by occupants and tabulated values for drift limits and associated serviceability concerns such as ceiling cracks and water leaks.

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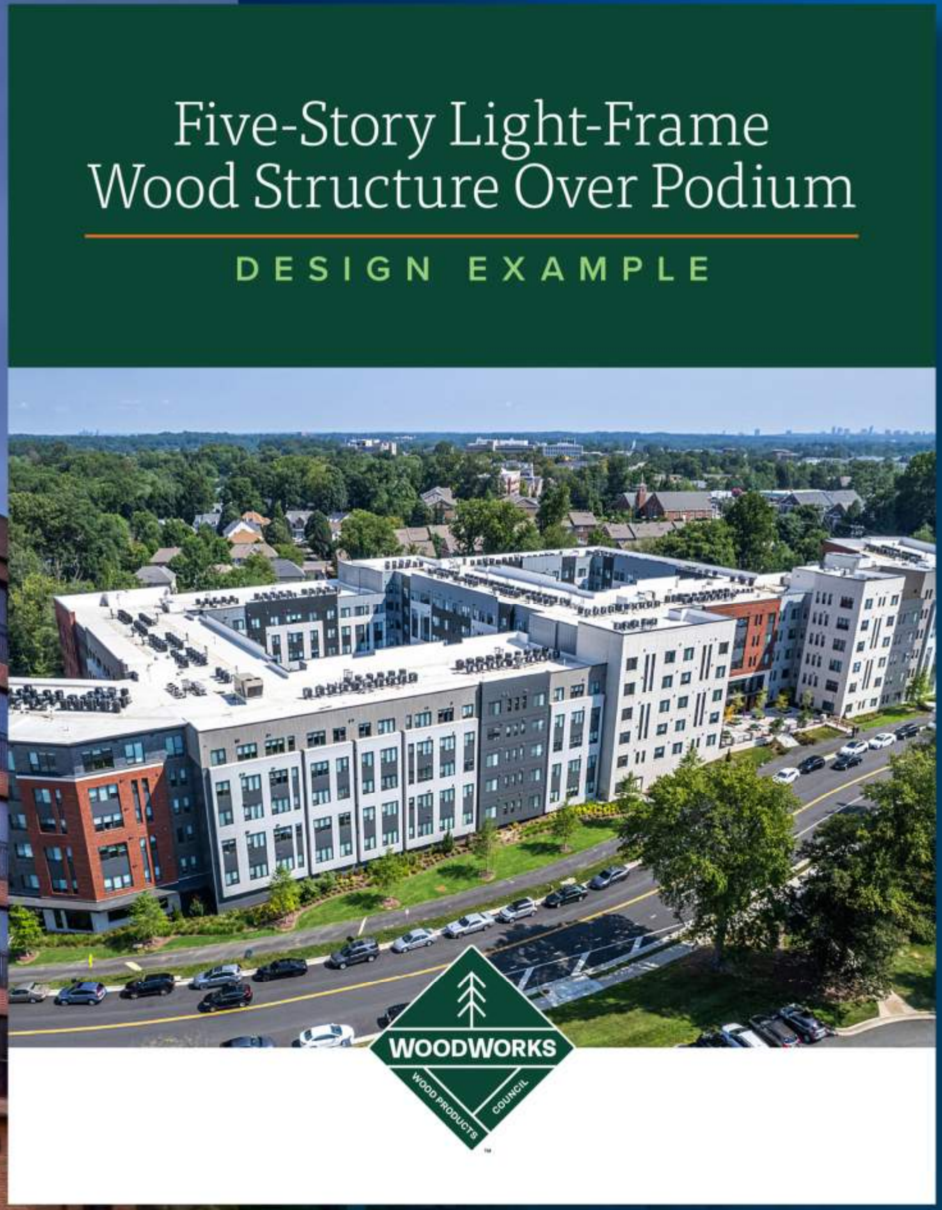

QUESTIONS?

This concludes The American Institute of Architects Continuing Education Systems Course

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