WoodWorks™
WOOD PRODUCTS COUNCIL

Essential Design & Detailing Aspects of Mid-Rise Wood-Frame Construction

Marc Rivard, PE, SE
Regional Director - MA, CT, ME, NH, RI, VT

Momo Sun, PE
Regional Director – NY, NJ, PA

Terry Pattillo, AIA
Regional Director – DC, DE, WV, VA

Photo: Brett Drury

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Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

Course Description

- This seminar will focus on structural design strategies for mid-rise wood-frame projects. As modern multi-family living evolves to achieve greater urban density while accommodating more amenities and long-term value, projects are growing larger, taller and incorporating more open space with views of the surrounding neighborhood. As the material of choice for many of these mid-rise projects, wood framing is well suited to accomplish these evolving trends in multi-family and mid-rise construction. However, along with a shift in the aesthetic and programmatic layout of these buildings has come a need to better frame the spaces and that onus is on the structural engineers. In addition, more and more of a building's fire and life safety design considerations are becoming a joint effort with architect and engineer.
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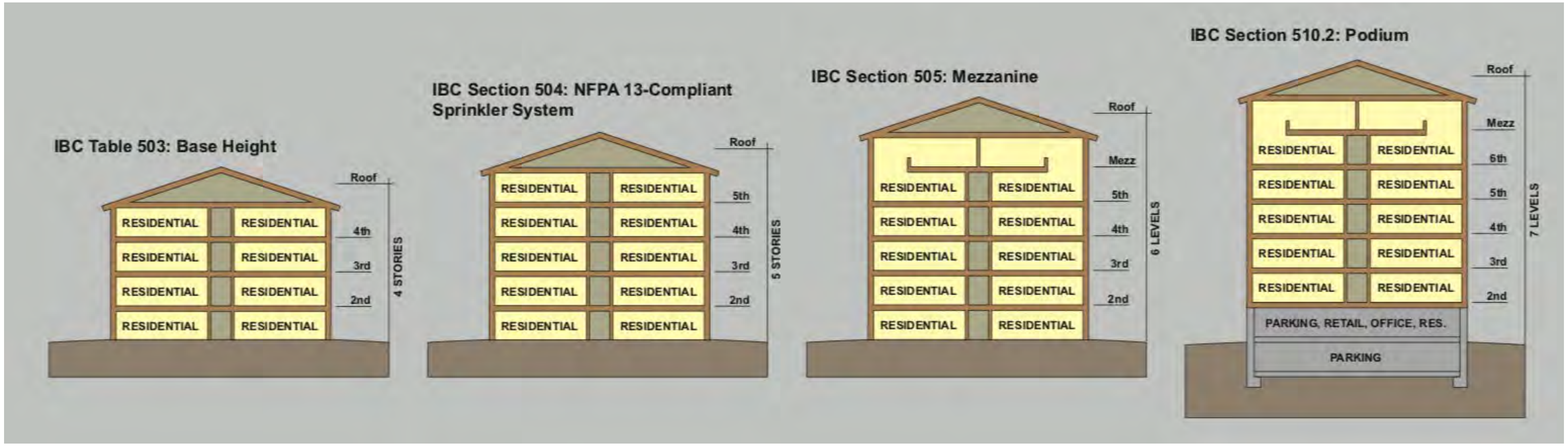
- This half-day seminar will address a number of topics that structural engineers will need to understand in order to cost effectively design and detail mid-rise and mid-rise over podium projects.

Evolution of Mid-Rise



Credit: WoodWorks

Heights and Areas – IBC 2015

IBC 2015: Table 504.3 provides base & increased heights

TABLE 504.3[a]
ALLOWABLE BUILDING HEIGHT IN FEET ABOVE GRADE PLANE

OCCUPANCY CLASSIFICATION	SEE FOOTNOTES	TYPE OF CONSTRUCTION								
		TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
		A	B	A	B	A	B	HT	A	B
A, B, E, F, M, S, U	NS[b]	UL	160	65	55	65	55	65	50	40
	S	UL	180	85	75	85	75	85	70	60
H-1, H-2, H-3, H-5	NS[c, d]	UL	160	65	55	65	55	65	50	40
	S									
H-4	NS[c, d]	UL	160	65	55	65	55	65	50	40
	S	UL	180	85	75	85	75	85	70	60
I-1 Condition 1, I-3	NS[d, e]	UL	160	65	55	65	55	65	50	40
	S	UL	180	85	75	85	75	85	70	60
I-1 Condition 2, I-2	NS[d, f, e]	UL	160	65	55	65	55	65	50	40
	S	UL	180	85						
I-4	NS[d, g]	UL	160	65	55	65	55	65	50	40
	S	UL	180	85	75	85	75	85	70	60
R	NS[d, h]	UL	160	65	55	65	55	65	50	40
	S13R	60	60	60	60	60	60	60	60	60
	S	UL	180	85	75	85	75	85	70	60

Heights and Areas – IBC 2015

IBC 2015: Table 504.4 provides base & increased stories

TABLE 504.4[a, b]
ALLOWABLE NUMBER OF STORIES ABOVE GRADE PLANE

OCCUPANCY CLASSIFICATION		TYPE OF CONSTRUCTION								
	SEE FOOTNOTES	TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
		A	B	A	B	A	B	HT	A	B
A-1	NS	UL	5	3	2	3	2	3	2	1
	S	UL	6	4	3	4	3	4	3	2
A-2	NS	UL	11	3	2	3	2	3	2	1
	S	UL	12	4	3	4	3	4	3	2
A-3	NS	UL	11	3	2	3	2	3	2	1
	S	UL	12	4	3	4	3	4	3	2
R-1	NS[d, h]	UL	11	4	4	4	4	4	3	2
	S13R	4	4						4	3
	S	UL	12	5	5	5	5	5	4	3
R-2	NS[d, h]	UL	11	4	4	4	4	4	3	2
	S13R	4	4	4					4	3
	S	UL	12	5	5	5	5	5	4	3
R-3	NS[d, h]	UL	11	4	4	4	4	4	3	3
	S13R	4	4						4	4
	S	UL	12	5	5	5	5	5	4	4

Sloped Sites

HEIGHT, BUILDING. The vertical distance from *grade plane* to the average height of the highest roof surface.

GRADE PLANE. A reference plane representing the average of finished ground level adjoining the building at *exterior walls*. Where the finished ground level slopes away from the *exterior walls*, the reference plane shall be established by the lowest points within the area between the building and the *lot line* or, where the *lot line* is more than 6 feet (1829 mm) from the building, between the building and a point 6 feet (1829 mm) from the building.

626 Dekalb Avenue, Atlanta, GA
Matt Church - Davis Church Structural Engineers

Mid-Rise vs. High-Rise Definition – IBC 202

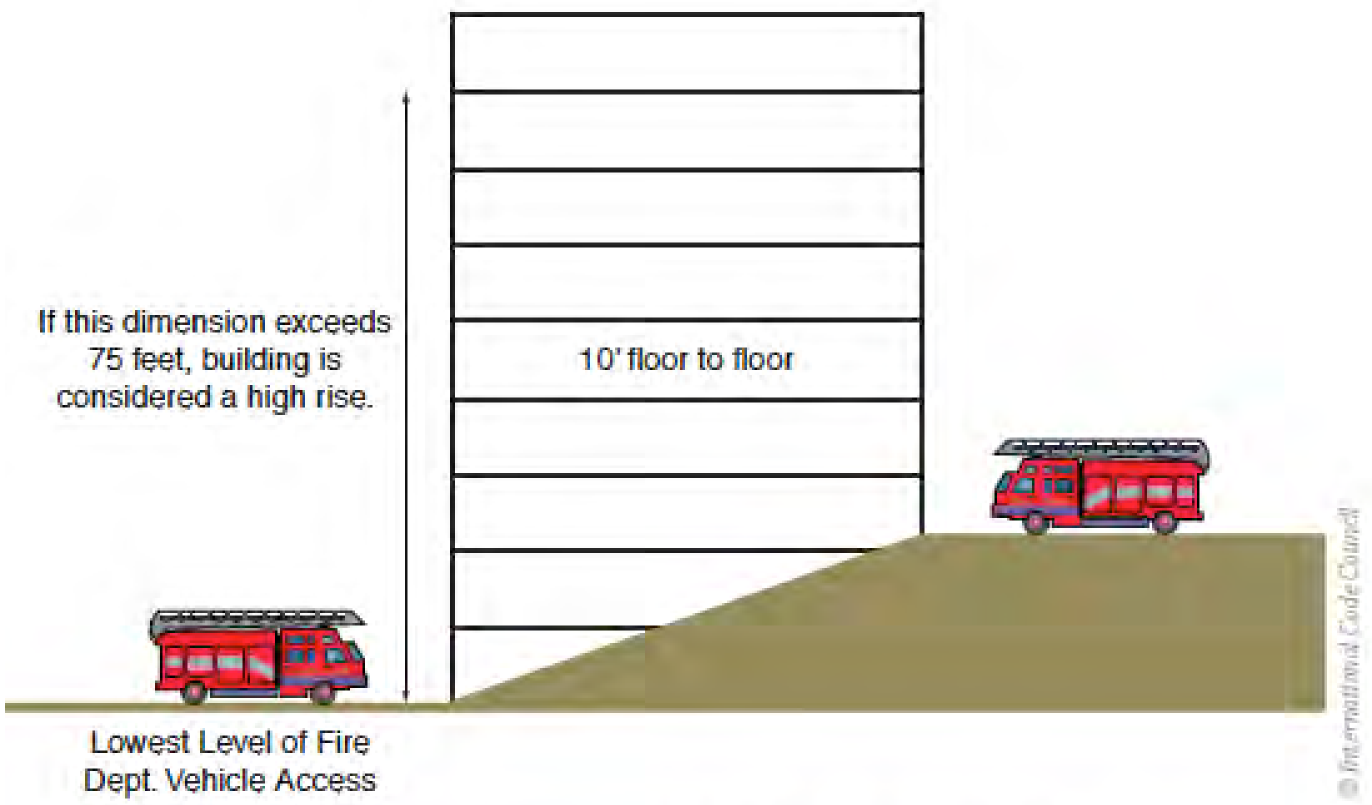
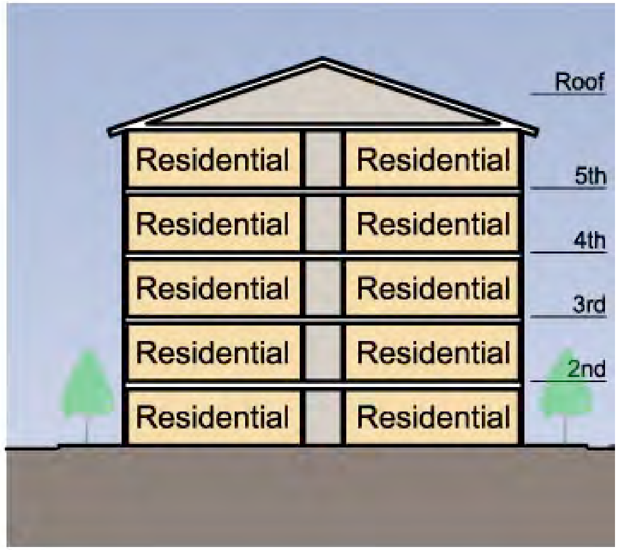


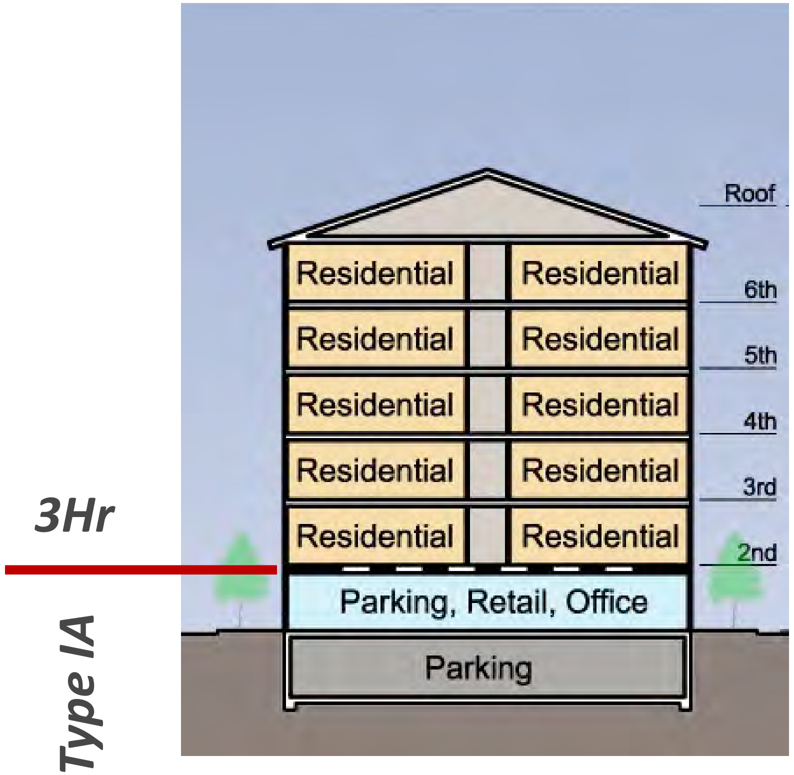
FIGURE 6-6 Determination of high-rise building

IBC 202: High-Rise Building: A building with an occupied floor located more than 75 feet above the lowest level of fire department vehicle access.

IBC Podium Provisions



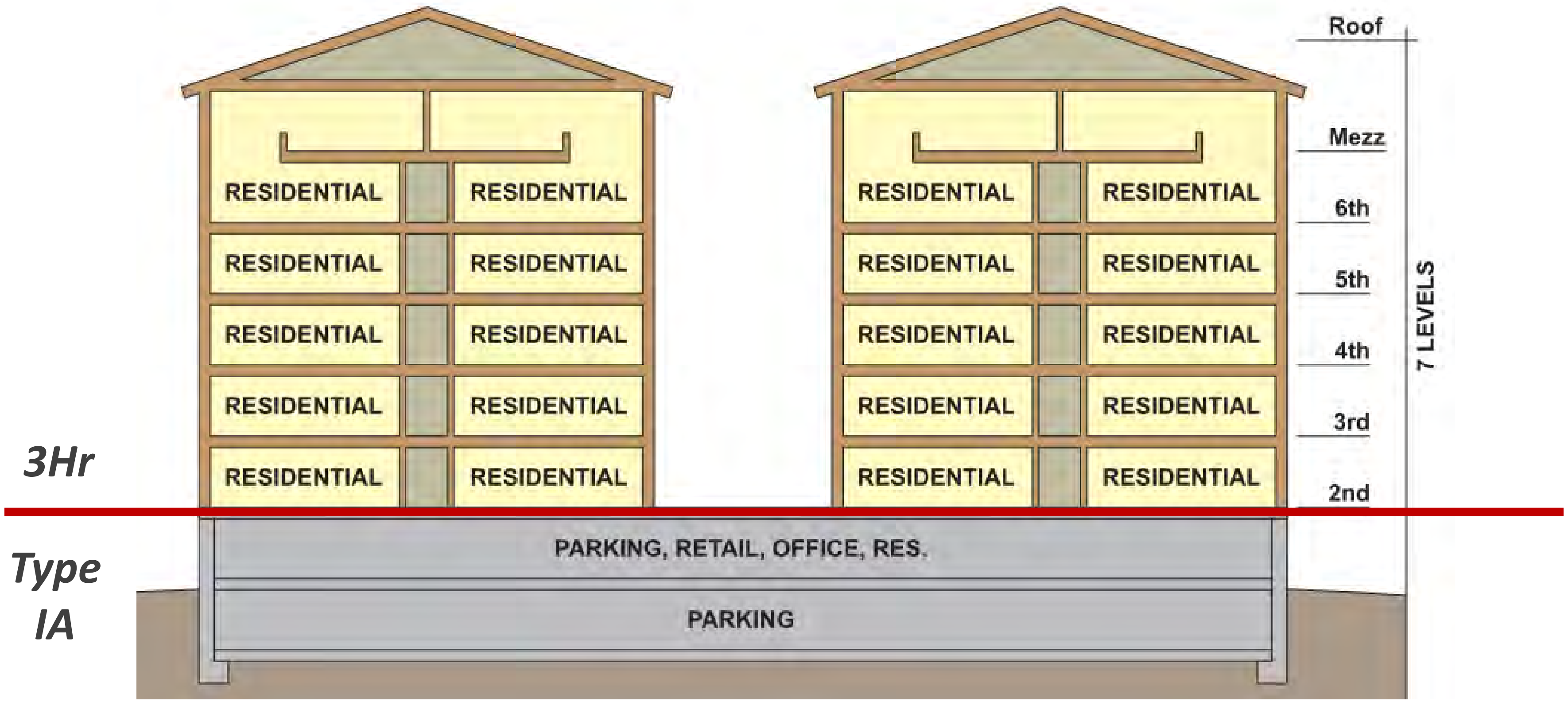
5 story Type III Building



5 story Type III Building
On Top of a Type IA Podium

Special Provisions for Podiums in IBC 2012 510.2
Increases allowable stories… not allowable building height

IBC Podium Provisions



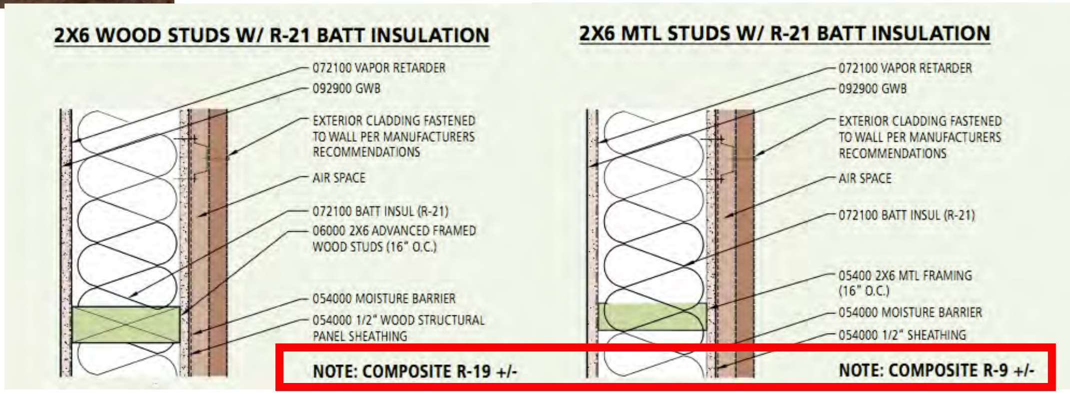
Multiple Buildings over one Podium
See Special Provisions for Podiums in IBC 2012 510.2

Wood Within Podium Level(s)

Credit: WoodWorks

FRTW is permitted in non-bearing, non-rated exterior walls in types I & II (IBC 603.1)

Thermal/building envelope benefits, as well as consistent exterior wall detailing



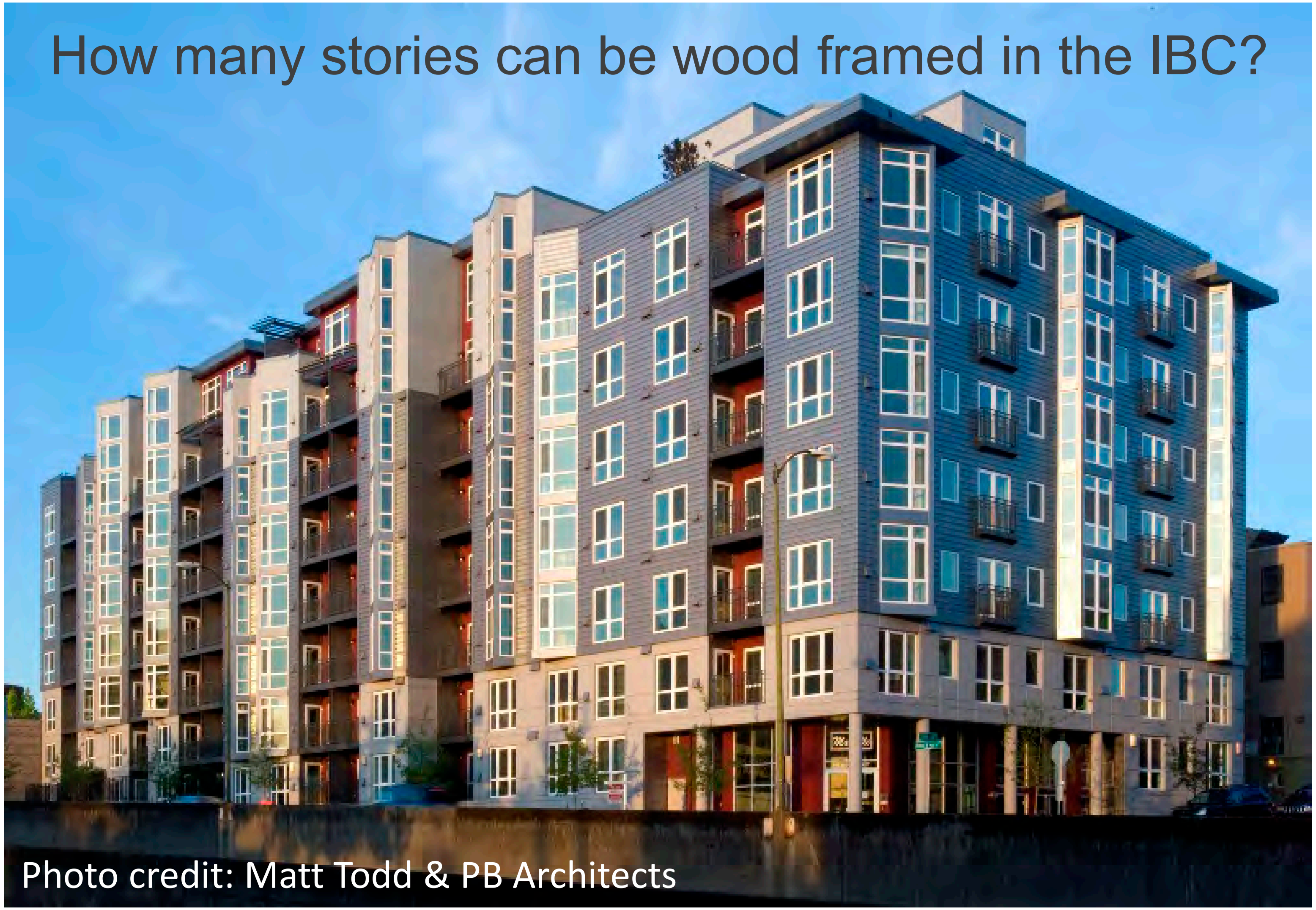
Source: Mahlum Architects

Wood Within Podium Level(s)

Credit: WoodWorks

2021 IBC allows stairs below the podium to be framed with wood if building above podium is type III, IV or V

Wood Mid-Rise Construction



Marselle Condos, Seattle, WA

6 stories for Offices, 5 stories for Residential
+ Mezzanine + Multi-Story Podium

Mid-Rise Construction Types

- **Type V**
- All building elements are any allowed by code

- **Type III**
- Exterior walls non-combustible (may be FRTW)
- Interior elements any allowed by code

- Types III and V can be subdivided to A (protected) or B (unprotected)

Type V Construction

Multi-family

Restaurants

Retail

Office

Type VB Heights & Areas

Occupancy	# of Stories	Height	Area per Story	Building Area
A-2	2	60 ft	18,000 SF	36,000 SF
B	3	60 ft	27,000 SF	81,000 SF
M	2	60 ft	27,000 SF	54,000 SF
R-2	3	60 ft	21,000 SF	63,000 SF

Stories/Heights/Areas include allowable increases for sprinklers, but exclude potential frontage increase

1-story retail and restaurants

2 to 3-story residential/office

No fire resistance ratings required*

Type VA Heights & Areas

Occupancy	# of Stories	Height	Area per Story	Building Area
A-2	3	70 ft	34,500 SF	103,500 SF
B	4	70 ft	54,000 SF	162,000 SF
M	4	70 ft	42,000 SF	126,000 SF
R-2	4	70 ft	36,000 SF	108,000 SF

Stories/Heights/Areas include allowable increases for sprinklers, but exclude potential frontage increase

3 to 4-story residential/office

1-hour fire resistance rating required for most building elements

Type III Construction

Multi-family

K-12/Higher Ed

Hospitality

Office

Type IIIB Construction

Credit: Lever Architecture

Occupancy	# of Stories	Height	Area per Story	Building Area
A-2	3	75 ft	28,500 SF	85,500 SF
B	4	75 ft	57,000 SF	171,000 SF
M	3	75 ft	37,500 SF	112,500 SF
R-2	5	75 ft	48,000 SF	144,000 SF

Stories/Heights/Areas include allowable increases for sprinklers, but exclude potential frontage increase

4-story office / 5-story residential

2-hour fire resistance rating required for exterior bearing walls only (noncombustible or FRT construction)

Type IIIA Construction

Credit: Christian Columbres

Occupancy	# of Stories	Height	Area per Story	Building Area
A-2	4	85 ft	42,000 SF	126,000 SF
B	6	85 ft	85,500 SF	256,500 SF
M	5	85 ft	55,500 SF	166,500 SF
R-2	5	85 ft	72,000 SF	216,000 SF

Stories/Heights/Areas include allowable increases for sprinklers, but exclude potential frontage increase

5-story residential / 6-story office

2-hour rating for exterior bearing walls

1-hour rating for other building elements

Building Configuration Options

Many buildings utilize a higher construction type than necessary due to traditional practice. This can have an impact on fire ratings, materials and ultimately cost.


RBS

Building Configuration Options

Mixed-use occupancies on 1st floor of residential buildings often require longer spans for open areas (parking, retail, assembly). Structurally, this may require steel or concrete framing. This doesn't mean that it has to be a Type IA podium, can use these materials in <u>any</u> construction type (IBC 602.1.1)

Credit: Brett Drury

Building Configuration Options

Example:

5 story building

1st floor: parking

2nd-5th floors residential

Options:

4-story, type VA over 1 story type IA (podium provision – IBC 510.2)

4 Stories of type VA over 1 story type IV (open) or type I (IBC 510.4) no ”podium” req’d

5 stories of type III (enclosed parking only) sep. or non-sep. occupancies

Building Configuration Options

Example:

7 story building

(6 above grade)

Basement: parking

1st-6th floors: residential

Options:

5-story, type III over 1 story type IA (podium provision – IBC 510.2)

4-story, type VA over 2 story podium (podium provision 2015 IBC 510.2)

6-story type IIIA (IBC 510.5 – requires 3000 ft^2 max areas & other limitations)

Building Configuration Options

Example:

5 story hotel

1^{st} floor: lobby, restaurant, fitness center, conference rooms, residential

2^{nd}-5^{th} floors residential

<u>Option 1:</u>

4-story, type VA over 1 story type IA (podium provision – IBC 510.2)

Mixed-use on 1^{st} floor handled with separated/non-separated occupancies considering that floor only

Building Configuration Options

Example:

5 story hotel

1^{st} floor: lobby, restaurant, fitness center, conference rooms, residential

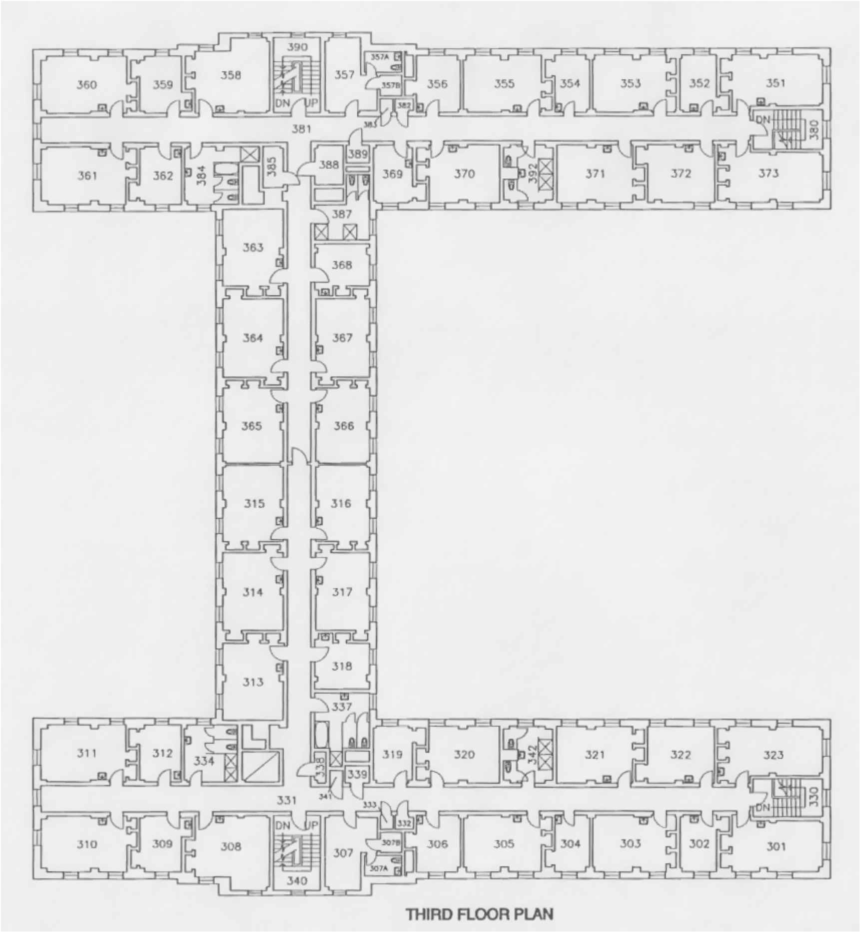
2^{nd}-5^{th} floors residential

<u>Option 2:</u>

5-story, type III

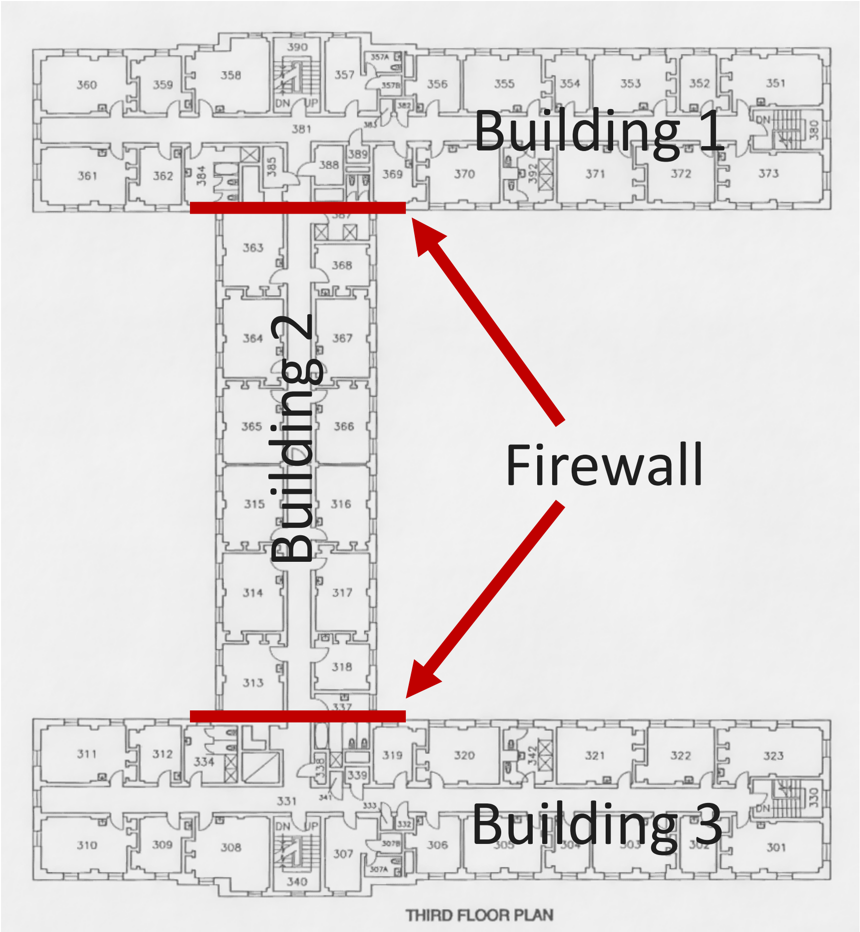
Mixed-use on 1^{st} floor handled with separated/non-separated occupancies considering <u>all</u> floors

Building Configuration Options



Example:
T- and L-shaped buildings – common in hotels, often with large floor areas

Building Configuration Options



These building configurations may lend themselves well to use of firewalls at building intersections.

Minimize length/impact of firewall while maximizing allowable building area

may allow lower construction type (i.e. type IIIB instead of IIIA)

Let’s Talk Structure (architecturally)

Type III Exterior Walls – FRT

Type III Construction - IBC Section 602.3:
Fire-retardant-treated wood framing complying with Section 2303.2 shall be permitted within exterior wall assemblies of a 2-hour rating or less

What does this FRTW requirement include?

- Wall Framing (Studs & Plates) – Yes
- Headers – Yes
- Wall Sheathing – Yes
- Floor sheathing - ?
- Rim Joist- ?
- Floor Joists- ?


Credit: WoodWorks

Exterior Walls – Intersecting Floors

Does the floor framing & sheathing that extends into the exterior wall need to FRT?

Choosing Fire Rated Assemblies

Common tested assemblies (ASTM E119) per IBC 703.2:

- UL Listings
- Gypsum Catalog
- Proprietary Manufacturer Tests
- **Industry Documents: such as AWC's DCA3**

Alternate Methods per IBC 703.3

- Prescriptive designs per IBC 721.1
- Calculated Fire Resistance per IBC 722
- Fire-resistance designs documented in sources
- Engineering analysis based on a comparison
- Fire-resistance designs certified by an approved agency

Exterior Walls – Intersecting Floors

AWC's DCA3 provides floor to wall intersection detailing options

Addresses both continuity provisions and requirements for FRT elements in exterior wall plane

Credit: AWC

Fire-Resistance-Rated Wood-Frame Wall and Floor/Ceiling Assemblies

Building Code Requirements

For occupancies such as stores, apartments, offices, and other commercial and industrial uses, building codes commonly require floor/ceiling and wall assemblies to be fire-resistance rated in accordance with standard fire tests. This document is intended to aid in the design of various wood-frame walls and wood-frame floor/ceiling assemblies, where such assemblies are required by code to be fire-resistance-rated.

Depending on the application, wall assemblies may need to be fire-resistance-rated for exposure from either one side or both sides. Exterior walls are required to be rated for both interior and exterior fire exposure where the wall has a fire separation distance of 10 feet or less. For exterior walls with a fire separation distance of greater than 10 feet, the required fire-resistance-rating applies only to exposure from the interior. The designer should note that some state and local building code amendments may require fire resistance rating for exposure from both sides of exterior walls, regardless of fire separation distance; however,

Fire Tested Assemblies

Fire-resistance-rated wood-frame assemblies can be found in a number of sources including the *International Building Code (IBC)*, Underwriters Laboratories (UL) *Fire Resistance Directory*, Intertek Testing Services' *Directory of Listed Products*, and the Gypsum Association's *Fire Resistance Design Manual* (GA 600). The American Wood Council (AWC) and its members have tested a number of wood-frame fire-resistance-rated assemblies (see photos). Descriptions of successfully tested lumber wall assemblies are provided in Table 1 for one-hour fire-resistance-rated wall assemblies and Table 2 for two-hour fire-resistance-rated wall assemblies. Lumber shall be identified by the grade mark of a lumber grading or inspection agency that has been approved by an accreditation body that complies with the *American Softwood Lumber Standard (PS 20)*. The fire-resistance-rated assemblies described in this document, as well as those listed in other sources are not species- or grade-specific unless specifically noted as such.

Exterior Walls – Intersecting Floors

Credit: AWC

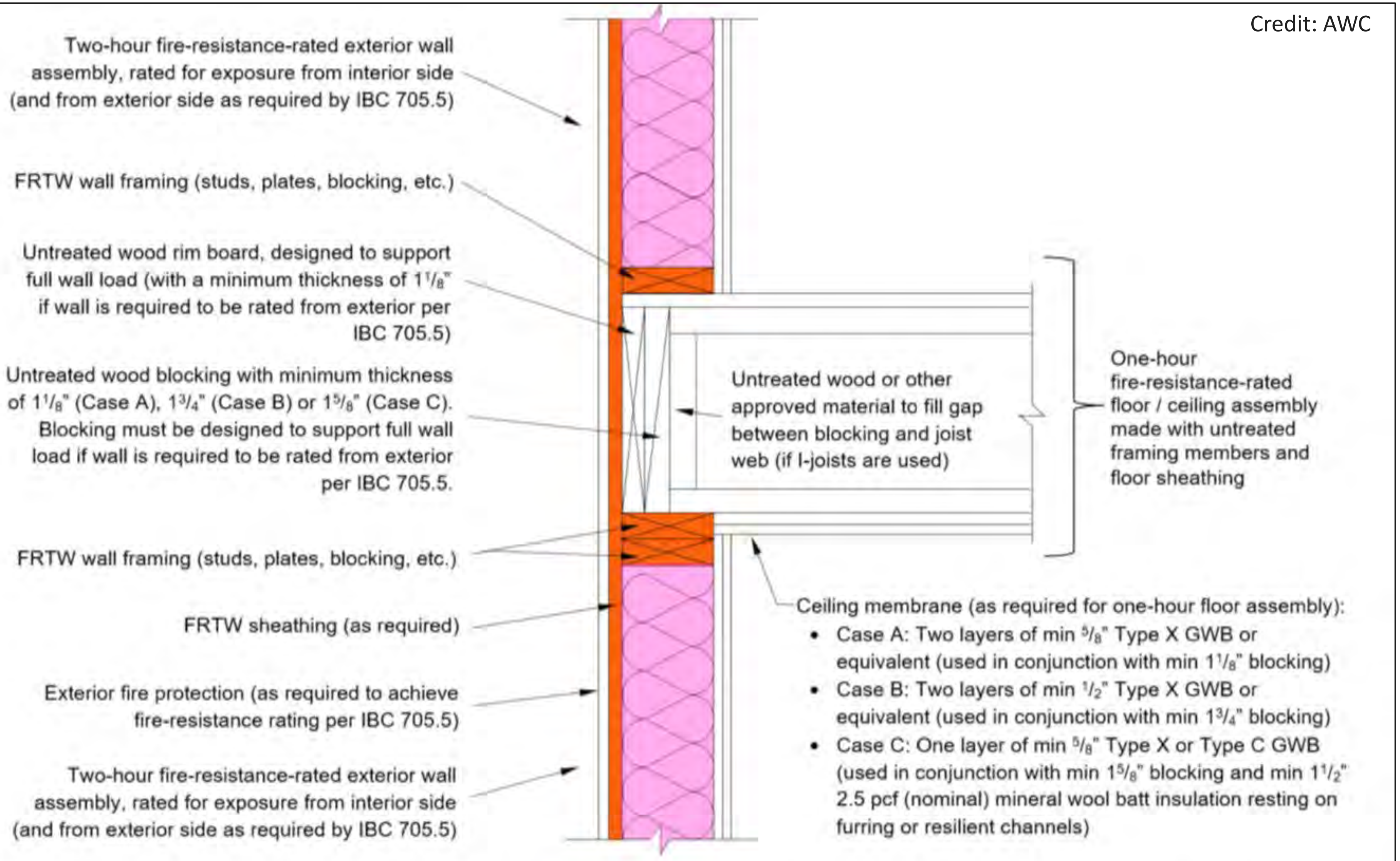
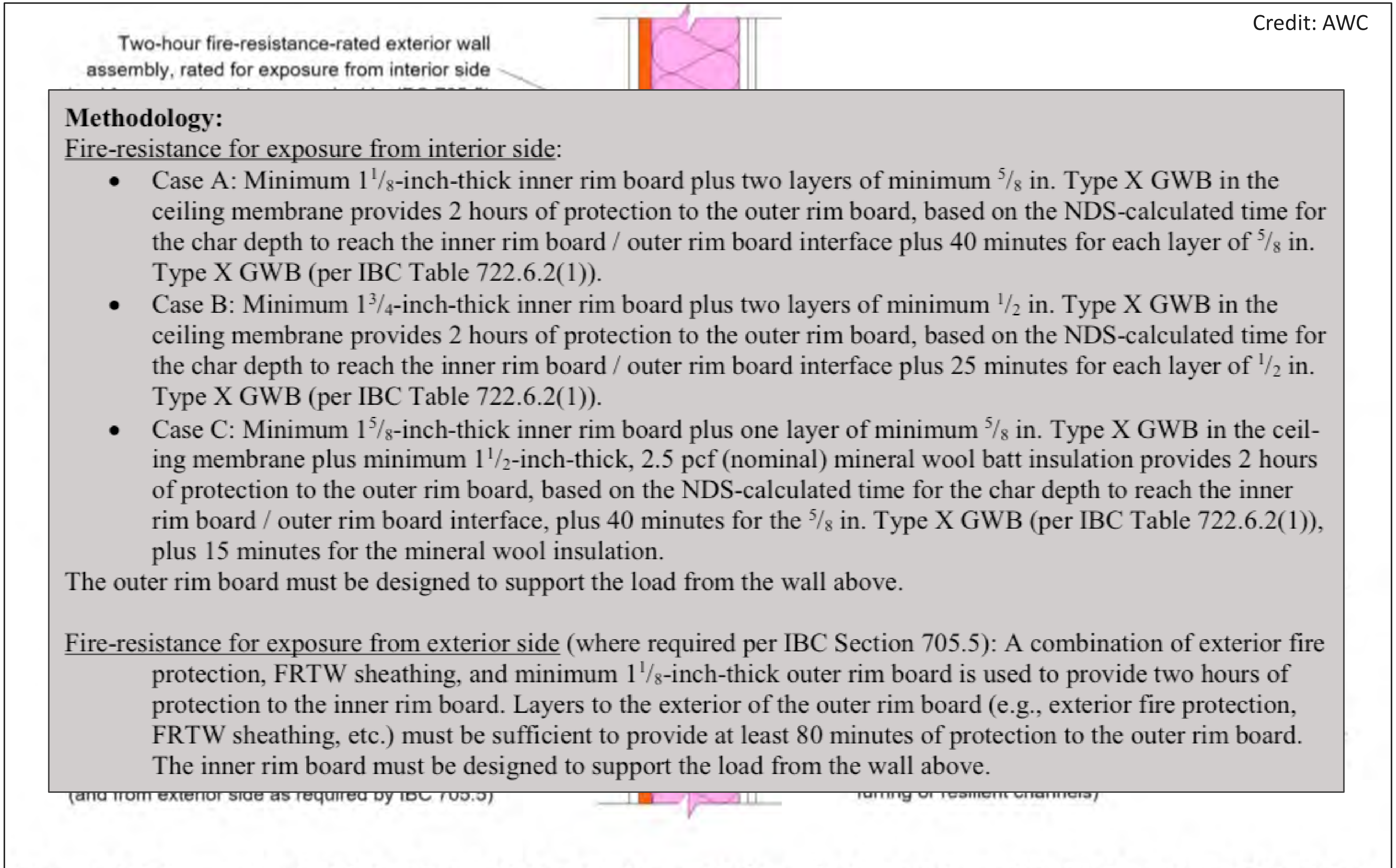


Figure 1A: Example detail for Type III-A exterior wall–floor intersection with rim board and blocking

Exterior Walls – Intersecting Floors

Credit: AWC



Methodology:

<u>Fire-resistance for exposure from interior side</u>:

- Case A: Minimum $1^1/_8$-inch-thick inner rim board plus two layers of minimum $^5/_8$ in. Type X GWB in the ceiling membrane provides 2 hours of protection to the outer rim board, based on the NDS-calculated time for the char depth to reach the inner rim board / outer rim board interface plus 40 minutes for each layer of $^5/_8$ in. Type X GWB (per IBC Table 722.6.2(1)).
- Case B: Minimum $1^3/_4$-inch-thick inner rim board plus two layers of minimum $^1/_2$ in. Type X GWB in the ceiling membrane provides 2 hours of protection to the outer rim board, based on the NDS-calculated time for the char depth to reach the inner rim board / outer rim board interface plus 25 minutes for each layer of $^1/_2$ in. Type X GWB (per IBC Table 722.6.2(1)).
- Case C: Minimum $1^5/_8$-inch-thick inner rim board plus one layer of minimum $^5/_8$ in. Type X GWB in the ceiling membrane plus minimum $1^1/_2$-inch-thick, 2.5 pcf (nominal) mineral wool batt insulation provides 2 hours of protection to the outer rim board, based on the NDS-calculated time for the char depth to reach the inner rim board / outer rim board interface, plus 40 minutes for the $^5/_8$ in. Type X GWB (per IBC Table 722.6.2(1)), plus 15 minutes for the mineral wool insulation.

The outer rim board must be designed to support the load from the wall above.

<u>Fire-resistance for exposure from exterior side</u> (where required per IBC Section 705.5): A combination of exterior fire protection, FRTW sheathing, and minimum $1^1/_8$-inch-thick outer rim board is used to provide two hours of protection to the inner rim board. Layers to the exterior of the outer rim board (e.g., exterior fire protection, FRTW sheathing, etc.) must be sufficient to provide at least 80 minutes of protection to the outer rim board. The inner rim board must be designed to support the load from the wall above.

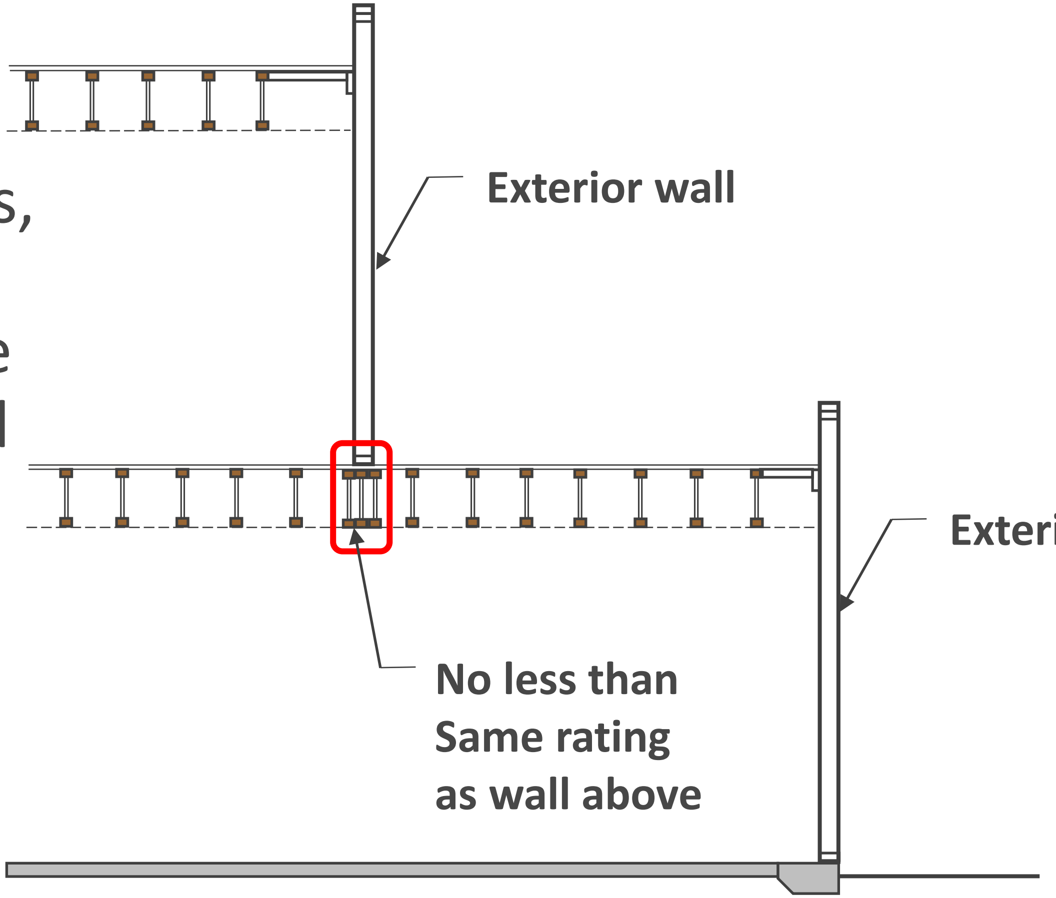
Figure 1A: Example detail for Type III-A exterior wall–floor intersection with rim board and blocking

Exterior Walls – Vertical Offsets

There is no requirement for an exterior wall to extend to the foundation in a stepped building.

Posts, beams or walls, that support a rated exterior wall must be fire –resistance rated not less than the rating of the supported wall

(IBC -2015 705.6)

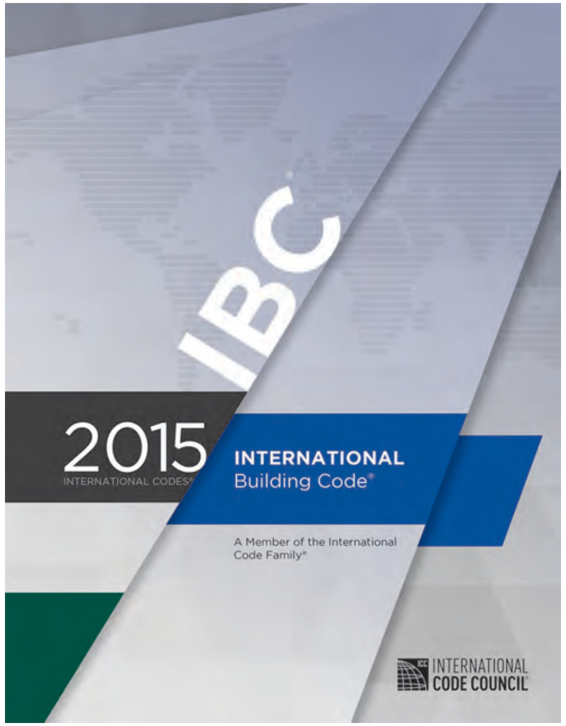


Accommodating Wood Shrinkage

Credit: Brett Drury

Shrinkage Code Requirements

- **2304.3.3 Shrinkage.** Wood walls and bearing partitions shall not support more than two floors and a roof unless an analysis satisfactory to the building official shows that <u>**shrinkage of the wood framing will not have adverse effects on the structure or any plumbing, electrical or mechanical systems, or other equipment installed**</u> therein due to excessive shrinkage or differential movements caused by shrinkage. The analysis shall also show that the roof drainage system and the foregoing systems or equipment will not be adversely affected or, as an alternative, such systems shall be designed to accommodate the differential shrinkage or movements.



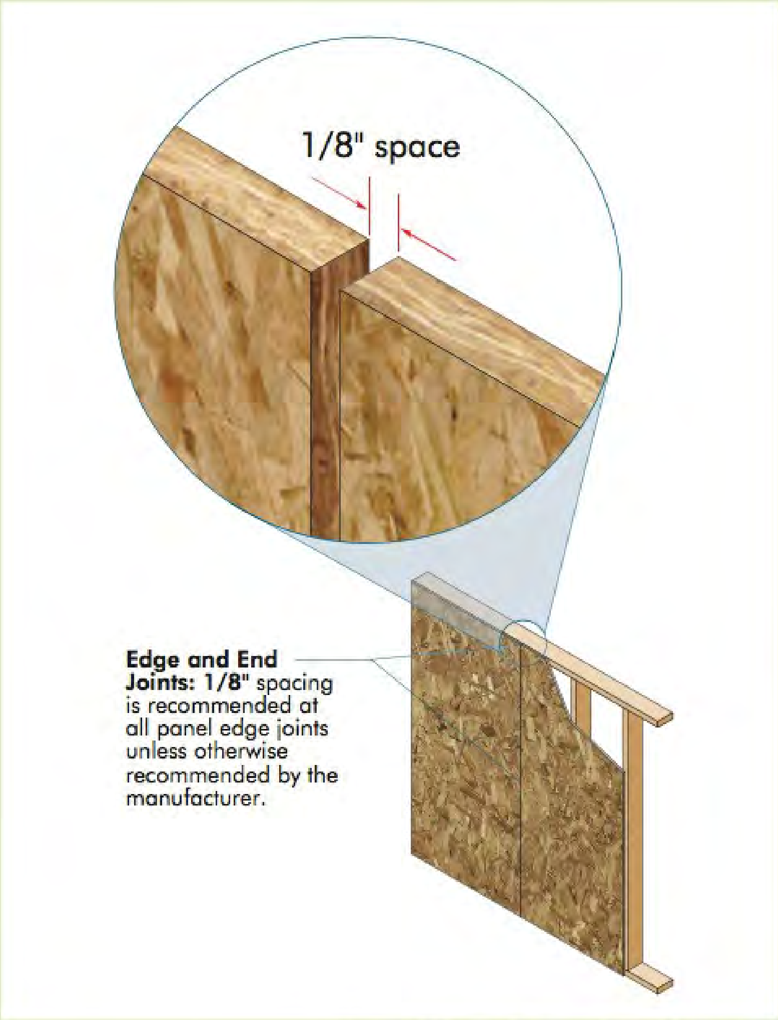
Shrinkage Calculations

Product	Moisture Content
Lumber – S-Dry	19% or less
Lumber – S-Green	Usually over 19%
Panel products (OSB, plywood)	4-8%
I-Joists	4-16%

$M_i = 19\%$

$M_i = 28\%$

Example Shrinkage Calculation



Source: APA – The Engineered Wood Association

Shrinkage Design Considerations

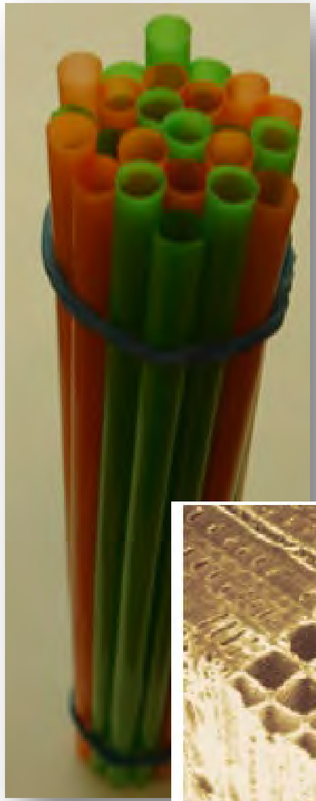
Designing and detailing to accommodate shrinkage is a design criteria but it doesn't need to be difficult

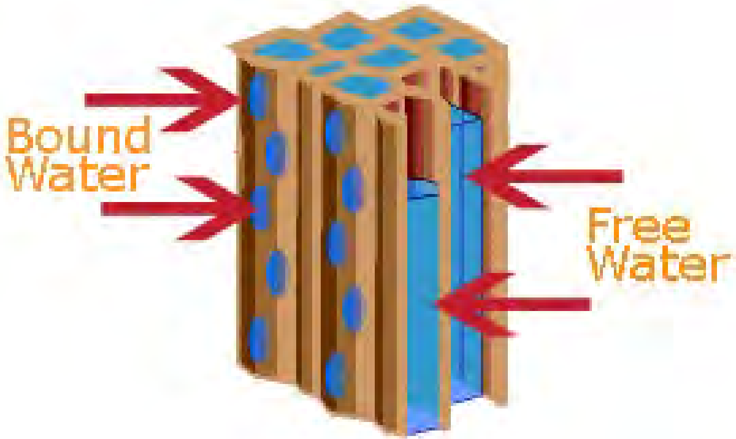
With proper calculations, detailing & an understanding of how and why wood shrinks, it simply becomes a very approachable design topic

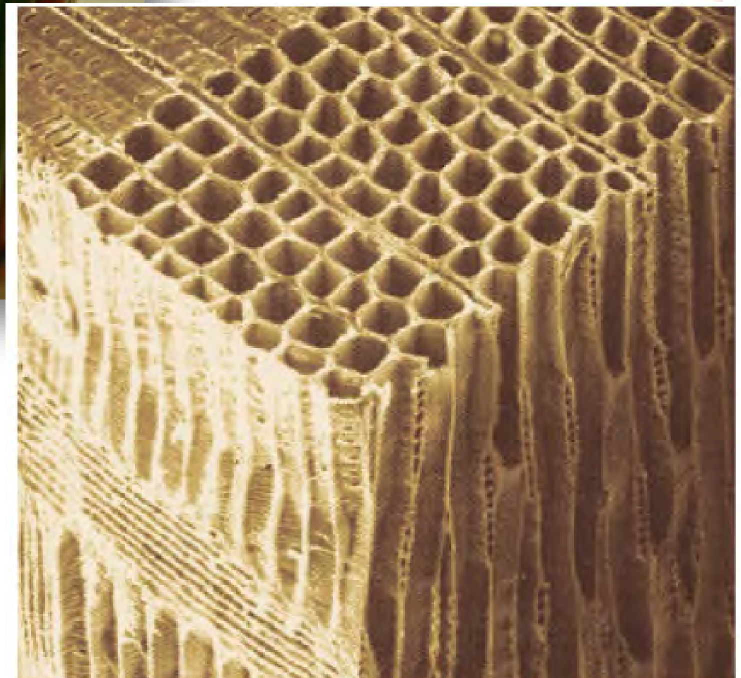
Shrinkage Design Considerations

Image: Schaefer

Wood Science – Moisture in Wood







When does wood shrink?

- After MC drops below FSP – bound water is removed

Why does wood shrink?

- Loss of moisture bound to cell wall changes thickness of cell wall

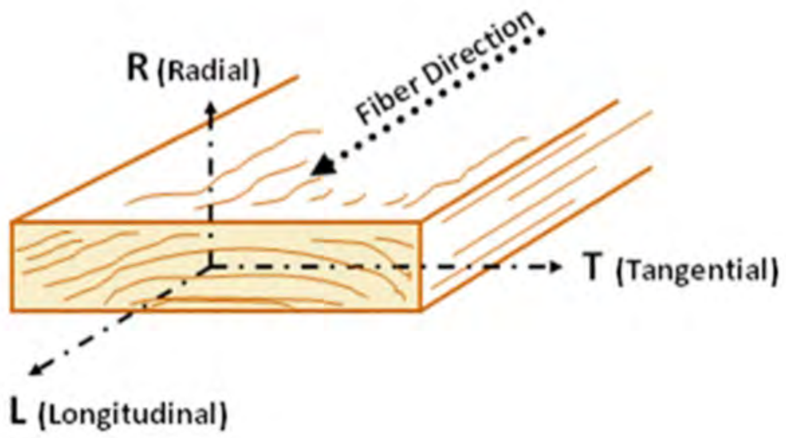
Is shrinkage uniform across all dimensions of a piece of lumber?

- No…

Wood Science

Wood is orthotropic, meaning it behaves differently in its three orthogonal directions: Longitudinal (L), Radial (R), and Tangential (T)

- Longitudinal shrinkage is negligible
- Can assume avg. of radial & tangential or assume all tangential



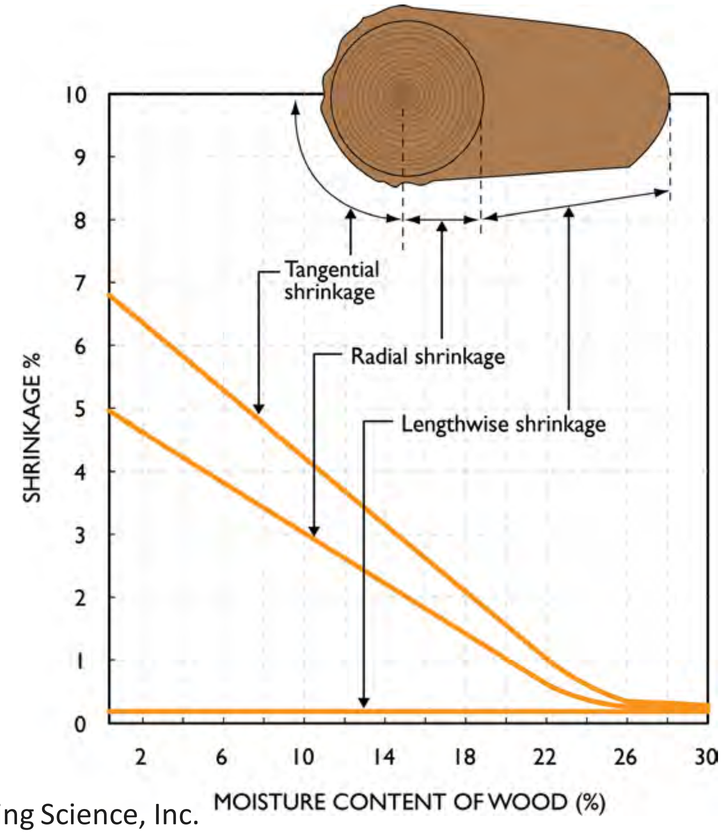


Image: RDH Building Science, Inc.

Shrinkage Calculations

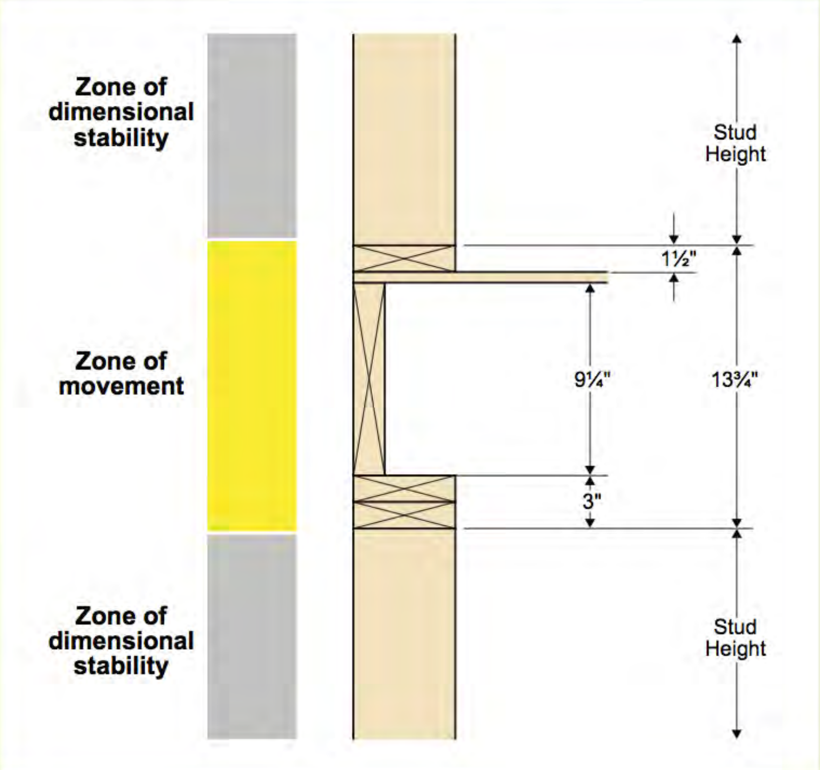
- **Three variables influence amount of shrinkage:**
- Installed moisture content (MC)
- In-service equilibrium moisture content (EMC)
- Cumulative thickness of cross-grain wood contributing to shrinkage

- Wood species has relatively little impact since most species used in commercial construction have similar shrinkage properties.

Shrinkage Calculations – Cross Grain Wood

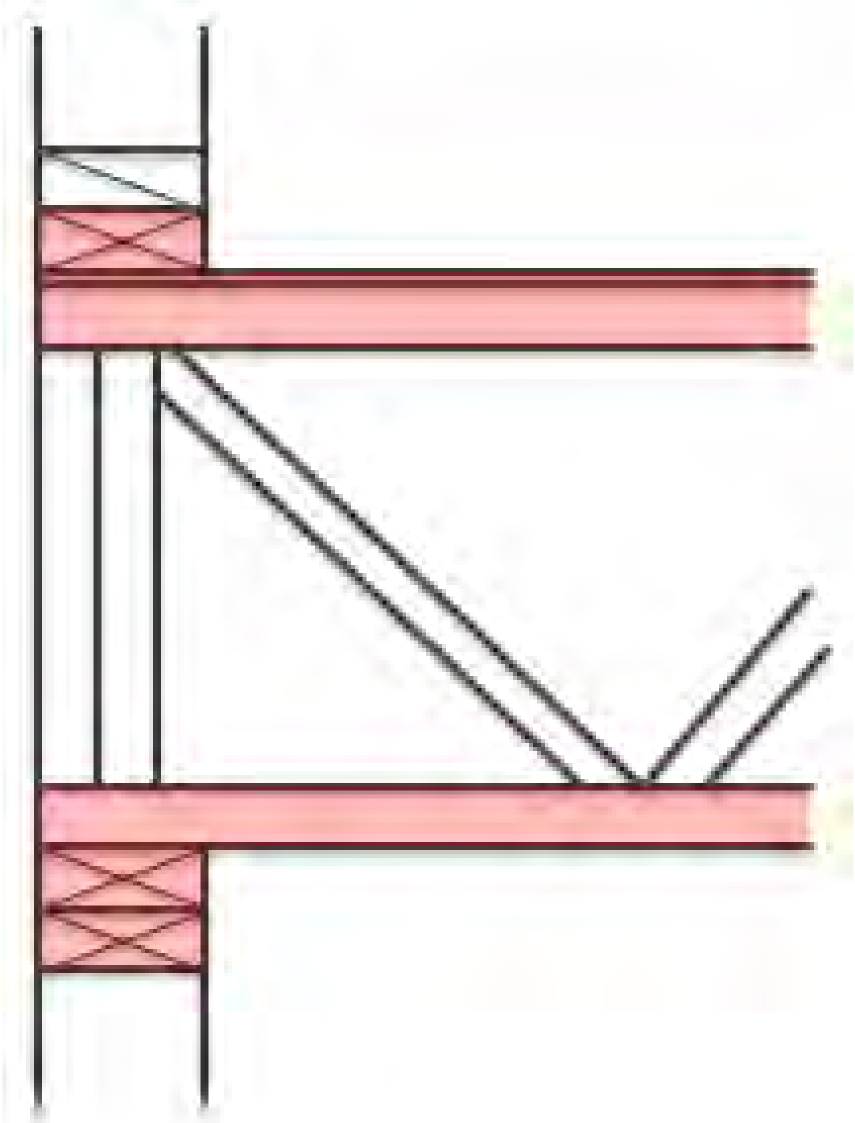
- Shrinkage occurs in cross-grain, but not longitudinal, wood dimensions
  - Primarily in horizontal members
  - Wall plates
  - Floor/rim joists
  - Engineering judgement required when determining what to include in shrinkage zone
  - Should Sheathing, I-Joists, Trusses, other products manufactured with low MC be included?

FIGURE 5:
Shrinkage zone in platform-framed detail



Shrinkage Calculations – Cross Grain Wood

In parallel chord trusses, only chords contribute to shrinkage, vertical and diagonal webs don't



Shrinkage Calculations – Running the Numbers

- **Simplified Method:**

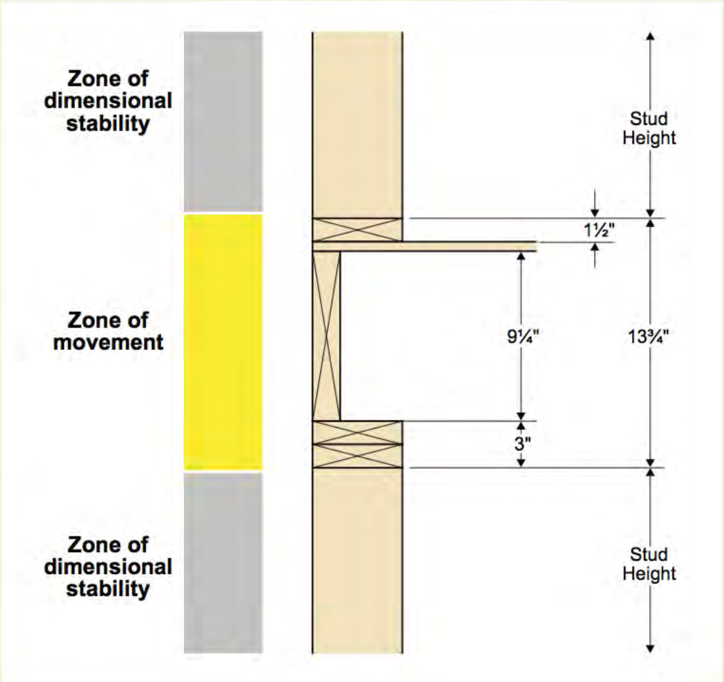
- S = 0.0025 in / inch of cross grain wood / % MC change

- Example: 13.75” shrinkage zone
- Installed MC = 19%
- EMC = 12%

- S = (0.0025)(13.75”)(12-19) = **-0.24”**

- (note: Negative value due to loss
- in cross section)

FIGURE 5:
Shrinkage zone in platform-framed detail



Minimizing Shrinkage

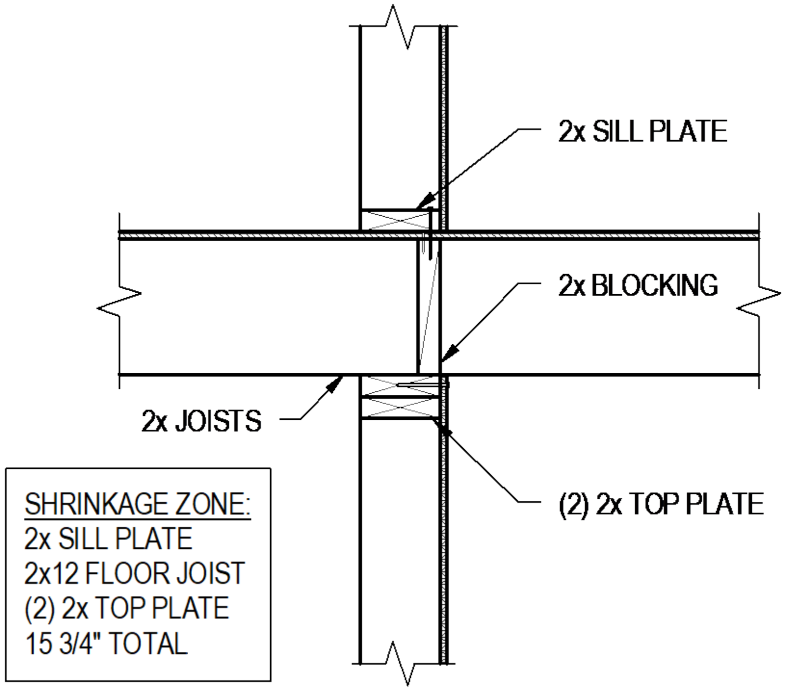
Recalling the three variables that influence amount of shrinkage:

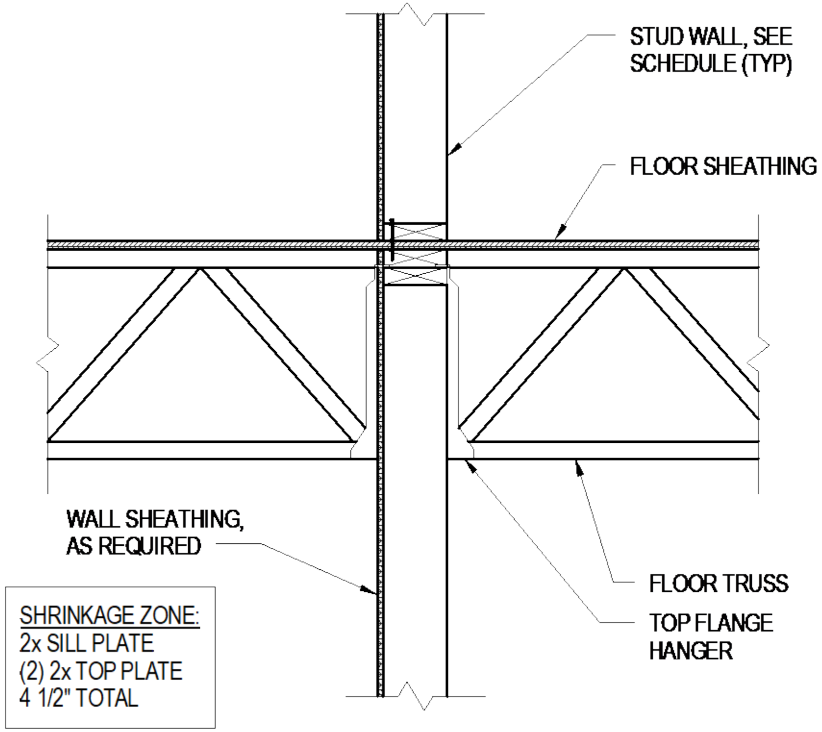
- Installed moisture content (MC)
- In-service equilibrium moisture content (EMC)
- Cumulative thickness of cross-grain wood contributing to shrinkage

As designers, we can impact 2 of these 3 variables

Our specifications and details, hand in hand with on-site protection measures and proper installation, can greatly minimize the magnitude and effects of shrinkage

Minimizing Shrinkage – Detailing





Images: Schaefer

Minimizing Shrinkage – Detailing

- **Platform Detail:**
- 15.75” Shrinkage Zone
- 19% MC Initial
- 12% EMC

- S = (0.0025)(15.75”)(12-19) = **0.28”**

- 5-story building: **1.4” total**

Semi-Balloon Detail:

4.5” Shrinkage Zone

19% MC Initial

12% EMC

S = (0.0025)(4.5”)(12-19) = **0.08”**

5-story building: **0.4” total**

In-Service Moisture Content

Shrinkage will continue to occur linearly until the wood's equilibrium moisture content (EMC) has been reached.

EMC is the point at which the wood is neither gaining nor losing moisture. However, this is a dynamic equilibrium as it is a function of temperature and relative humidity

USDA FPL "Wood Handbook"

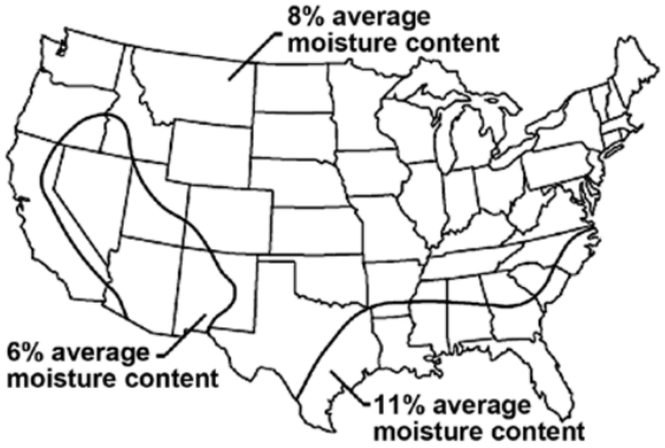


Figure 13–1. Recommended average moisture content for interior use of wood products in various areas of the United States.

WWPA Technical Report 10

Table 1. Average Outdoor and Indoor EMC

Location	Average Outdoor EMC (%)	Average Indoor EMC (%)
Los Angeles, CA	10	9
San Diego, CA	12	10
Twentynine Palms, CA	6	6
San Francisco Bay Area	13	9
Sacramento Valley (CA)	11	8
N. Coast Red. (CA)	14	9
Sierra Nevada (CA)	11	7
San Joaquin Valley (CA)	11	8

Differential Movement

- **Need to consider differential movement between wood frame elements and other materials that…**
- Expand due to moisture or thermal changes
- Do not change with moisture but do change with thermal fluctuations
- Shrink much less than wood




Differential Movement

Wood Framing & Veneer:

- Veneer Type Transitions
- Openings (Sill, Head, Jambs)

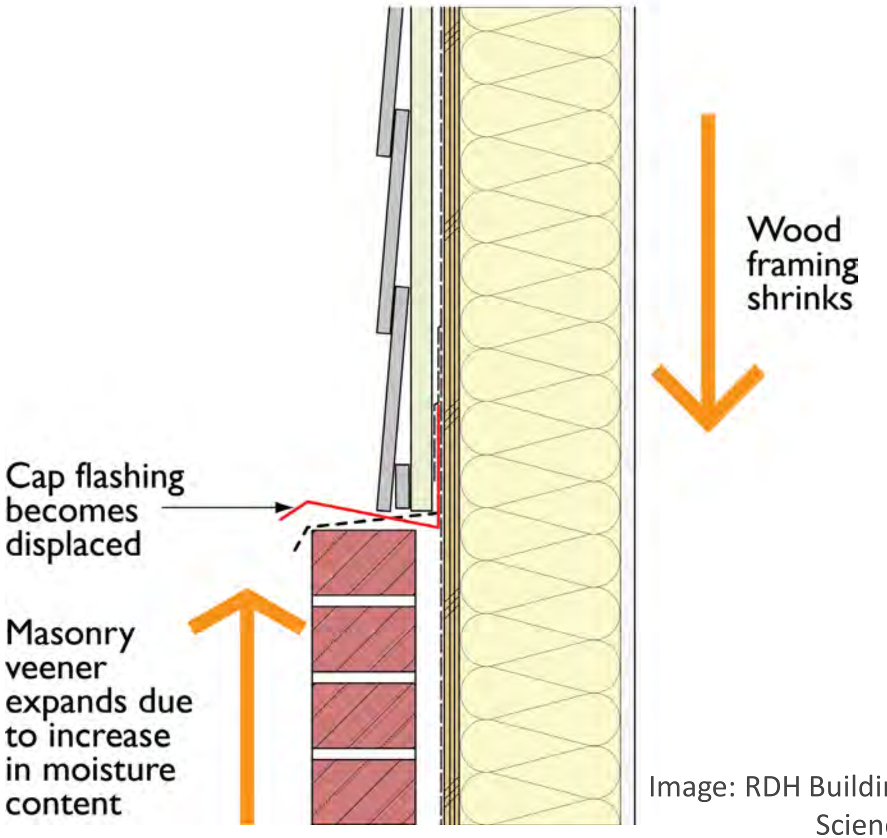


Image: RDH Building Science

Differential Movement – Veneer Transition

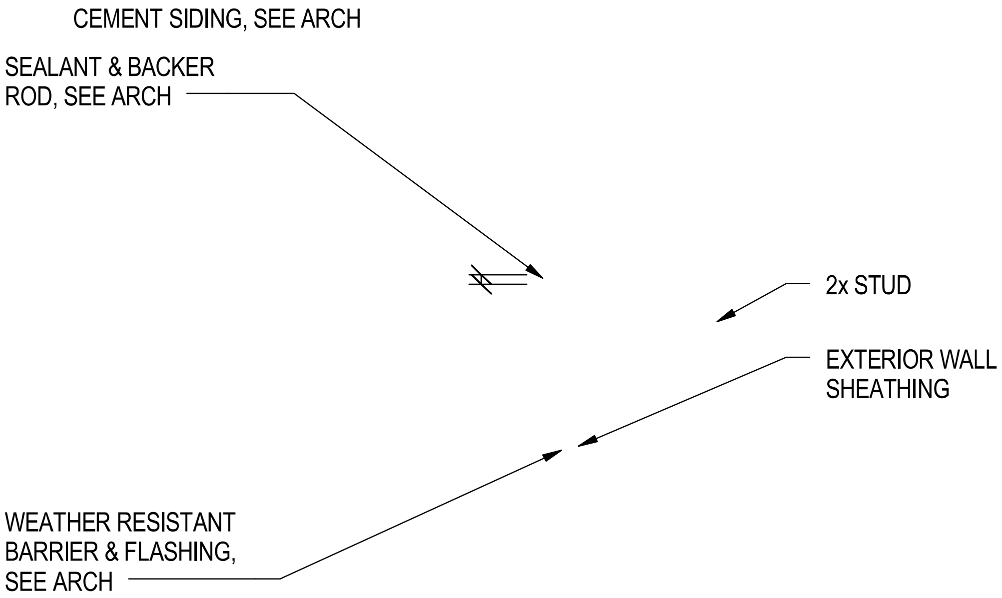
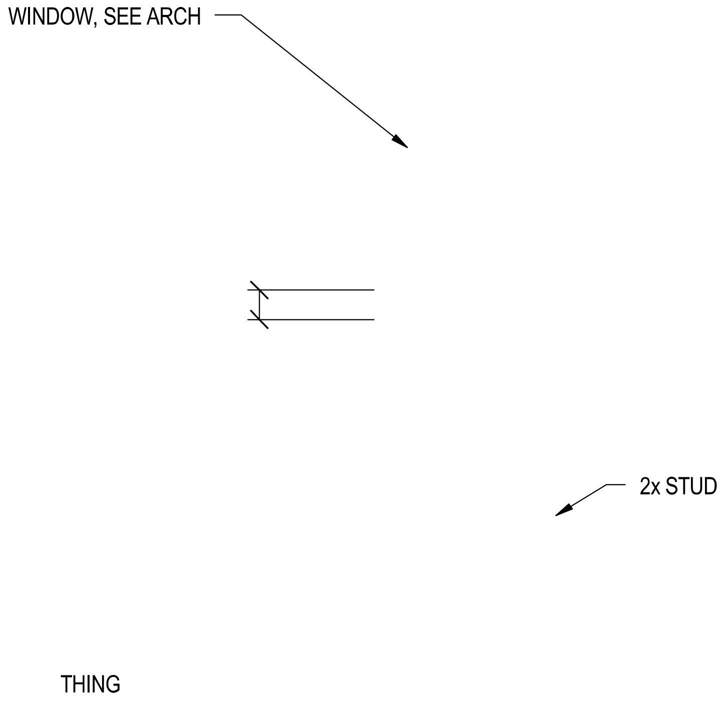


Image: Schaefer

Differential Movement – Veneer Opening



Images: Schaefer

Differential Movement – Veneer Opening

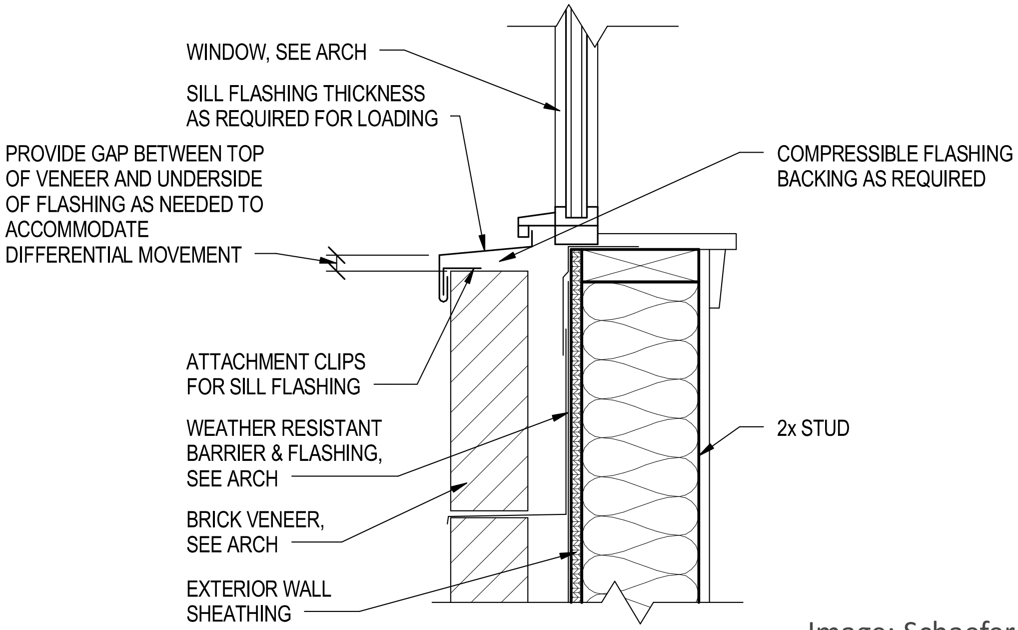


Image: Schaefer

Differential Movement – Veneer Opening

Image: RDH Building Science

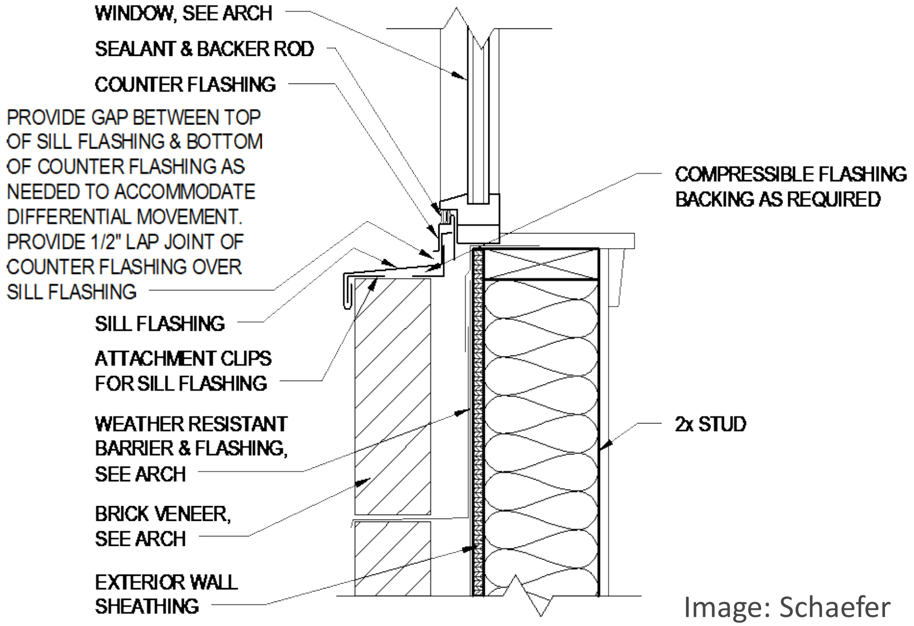


Image: Schaefer

Differential Movement - MEP

MEP main runs often start at base or top of structure, extend throughout height, with horizontal tees at each floor.

Horizontal tees often installed in wood stud partitions

Differential Movement - MEP

- Vertically slotted holes in studs allow differential movement
- Verify structural adequacy of studs

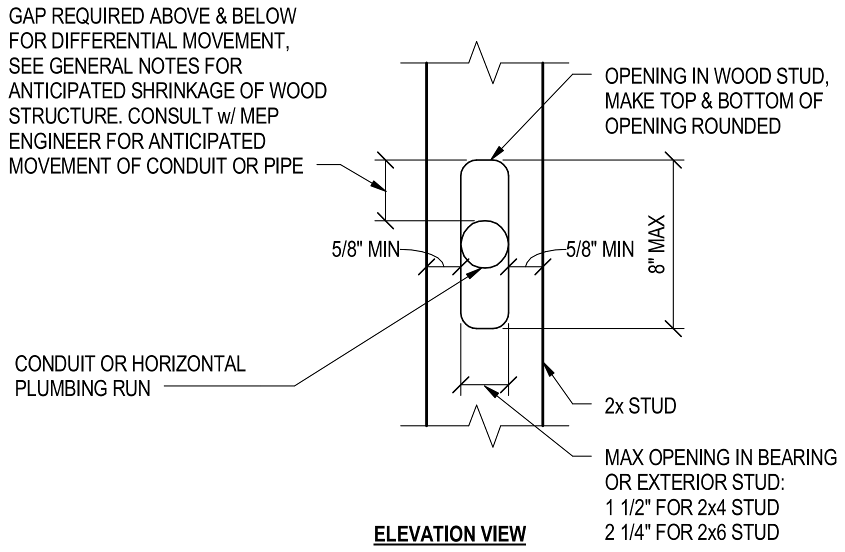


Image: Schaefer

NOTE: ENGINEER SHALL REVIEW LOADING CONDITIONS ON WALL FOR ALLOWABLE SIZE OF PENETRATION

Image: Louisiana-Pacific Corporation

Oval cutout options for Horizontal Pipe

Differential Movement - MEP

Wood framing shrinks, vertical MEP runs remain stationary or expand with thermal fluctuations

Differential movement should be allowed for

Helpful to wait as late as possible after wood framing is erected to install MEP

Note anticipated wood shrinkage at each level on construction documents – MEP contractor should provide methods of accommodating

Differential Movement - MEP

A variety of expansion or slip joint connectors are available – allow vertical MEP runs to move with the wood structure

Vertical Stacks – Compensation Devices Installed

Shrinkage Resource

Code provisions, detailing options, calculations and more for accommodating differential material movement in wood structures

Free resource at woodworks.org

WoodWorks™
WOOD PRODUCTS COUNCIL

Accommodating Shrinkage in Multi-Story Wood-Frame Structures

Richard McLain, MS, PE, SE, Technical Director, 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.

The Brooklyn Riverside
Jacksonville, Florida
Architect: Dwell Design Studio
Structural Engineer: M2 Structural Engineering

Photo: Pollack Shores, Matrix Residential

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.

Let’s Take a Break

Let's Talk Structure

Structure and Fire & Life Safety

Can't Live in Separate Bubbles

Structure and Fire & Life Safety

In any project, but particularly wood-frame mid-rise construction, efficiency in structural framing layout, assembly selection and detailing must also account for ”architectural” requirements such as:

- Fire-resistance ratings
- Acoustics
- Materials permitted (construction type)

In other words, you’re not just an engineer anymore


Credit: Brett Drury

Exterior Wall – Bearing vs. Non Bearing

Non loading-bearing exterior walls may have lower fire resistance rating requirements than bearing walls in certain situations. IBC Chapter 2 defines load bearing walls as:

[BS] WALL, LOAD-BEARING. Any wall meeting either of the following classifications:

1. Any metal or wood stud wall that supports more than 100 pounds per linear foot (1459 N/m) of vertical load in addition to its own weight.

[BS] **WALL, NONLOAD-BEARING.** Any wall that is not a *load-bearing wall*.

Exterior Wall – Bearing vs. Non Bearing

Why is this important? **Fire-Resistance Ratings and $**

Fire Rating of Structural Elements	IIA	IIB	IIIA	IIIB	IV	VB
IBC Table 601						
• Exterior bearing walls *(hours)*	1	0	2	2	2	0
• Interior bearing walls *(hours)*	1	0	1	0	1	0
• All other elements *(hours)*	1	0	1	0	HT	0
IBC Table 602						
• X < 10 feet	1	1	1	1	1	1
• 10 ft ≤ X < 30 feet	1	0	1	0	1	0
• X ≥ 30 feet	0	0	0	0	HT	0

Credit: WoodWorks

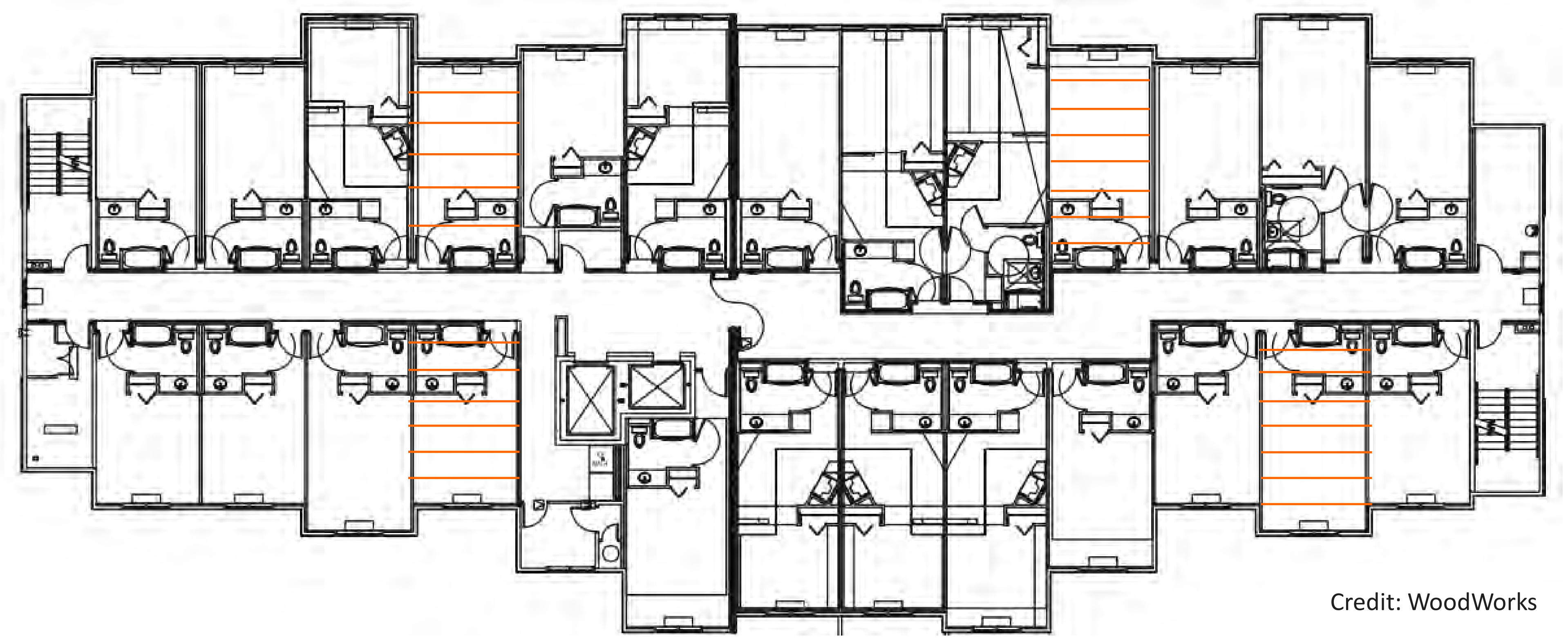
Type III:

Exterior Bearing Wall = **2-hours**

Exterior non-Bearing Wall = varies but often **0-hours**

Exterior Walls – Bearing vs. Non-Bearing

If framing parallel to long exterior walls is possible, minimizes area of load bearing exterior walls



Credit: WoodWorks

Type III Exterior Walls – FRT

Type III Construction - IBC Section 602.3:
Fire-retardant-treated wood framing complying with Section 2303.2 shall be permitted within exterior wall assemblies of a 2-hour rating or less


Credit: WoodWorks

Type III Exterior Walls – FRT

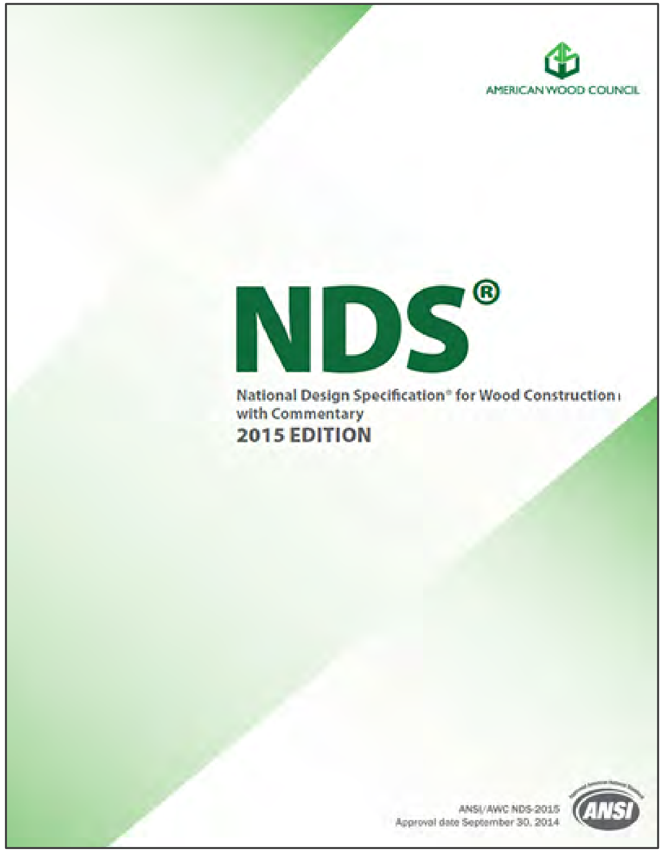
Long Span Headers in Type III

When a multi-ply 2x is inadequate due to load and span, what are the options?

- FRT EWP availability?
- Non-FRT wood options?
- Non-combustible materials?

FRT Wood Design Values

NDS 2.3.4: Adjusted design values, including adjusted connection design values, for lumber and structural glued laminated timber pressure-treated with fire retardant chemicals shall be obtained from the company providing the treatment and redrying service.



FRT Wood Design Values

FRT manufacturers provide reduction values in literature, ICC ESR's, etc.

Example FRT manufacturer's ESR reduction values:

TABLE 2—DESIGN VALUE ADJUSTMENT FACTORS FOR PYRO-GUARD® TREATED LUMBER

PROPERTY	PYRO-GUARD® WALL/FLOOR SERVICE TEMPERATURE TO 100°F/38°C			PYRO-GUARD® ROOF FRAMING, SERVICE TEMPERATURE TO 150° F/66° C,					
	Douglas fir	Southern pine	Other species	Douglas fir			Southern pine		
				Climate Zone			Climate Zone		
				1A	1B	2	1A	1B	2
Extreme fiber stress in bending, F_b	0.97	0.91	0.88	0.90	0.93	0.96	0.80	0.85	0.89
Tension parallel to grain F_t	0.95	0.88	0.83	0.80	0.87	0.93	0.80	0.84	0.88
Compression parallel to grain, F_c	1.00	0.94	0.94	0.94	0.98	1.00	0.94	0.94	0.94
Horizontal shear F_v	0.96	0.95	0.93	0.95	0.95	0.96	0.92	0.93	0.94
Modulus of elasticity, E	0.96	0.95	0.94	0.96	0.96	0.96	0.95	0.95	0.95
Compression perp. to grain F_{cz}	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Fasteners/connectors	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90

FRT Wood Design Values

Shear wall capacity reduction typically handled by increasing sheathing thickness

When fire-retardant-treated plywood is used in a shear wall, the thickness must be one standard size thicker than that determined in the tabulated allowable shear values contained in Section 4.3 of ANSI/AWC Special Design Provisions for Wind and Seismic (SDPWS) or as shown in the tables referenced in Section 2306.3 of the IBC (2306.4 of the 2009 and 2006 IBC). Thickness to be used for FRT plywood compared to untreated plywood shear walls are shown below:

FRT Plywood Thickness (inches)	Untreated Plywood Thickness (inches)
$^{3}/_{8}$	$^{5}/_{16}$
$^{7}/_{16}$	$^{3}/_{8}$
$^{15}/_{32}$	$^{7}/_{16}$
$^{1}/_{2}$	$^{15}/_{32}$

PT Sole Plate vs FRT Continuity

In type III construction with FRT studs, what happens where the sole plate is in contact with concrete?

- FRTW is required
- PT wood is required

FRT contains about 10x borate compound found in PT (borate is water soluble)

Can specify a product tested to do both

Floor Vibration Design

Structural Floor Design

Common Wood Floor Assembly:

LW Concrete Topping
Acoustical Mat
Wood Floor Sheathing
Wood Trusses/I-joists
Batt Insulation
Resilient Channel
Gypsum Ceiling

Structural Floor Design - Vibration

The code is silent on floor vibration criteria & analysis

TABLE 1604.3
DEFLECTION LIMITS[a, b, c, h, i]

CONSTRUCTION	L	S or W[f]	$D + L$[d, g]
Roof members:[e]			
Supporting plaster or stucco ceiling	$l/360$	$l/360$	$l/240$
Supporting nonplaster ceiling	$l/240$	$l/240$	$l/180$
Not supporting ceiling	$l/180$	$l/180$	$l/120$
Floor members	$l/360$	—	$l/240$
Exterior walls and interior partitions:			
With plaster or stucco finishes	—	$l/360$	—
With other brittle finishes	—	$l/240$	—
With flexible finishes	—	$l/120$	—
Farm buildings	—	—	$l/180$
Greenhouses	—	—	$l/120$

Structural Floor Design - Vibration

AMERICAN WOOD COUNCIL

Where can I find criteria for vibration control for wood members?

- Dolan and Woeste developed some information on controlling vibration published in *Structural Engineer* magazine.
- APA Technical Note called *Minimizing Floor Vibration by Design and Retrofit* http://www.apawood.org/SearchResults.aspx?q=E710&tid=1
- *Wood Design Focus* paper by Dolan and Kalkert called "Overview of Proposed Wood Floor Vibration Design Criteria" (Vol. 5, #3). http://www.forestprod.org/buy_publications/wood_design_focus_past_articles.php#volume5

Structural Floor Design - Vibration

IS A "SPRING IN YOUR STEP" CAUSING PROBLEMS?

June 2007 » Feature Article

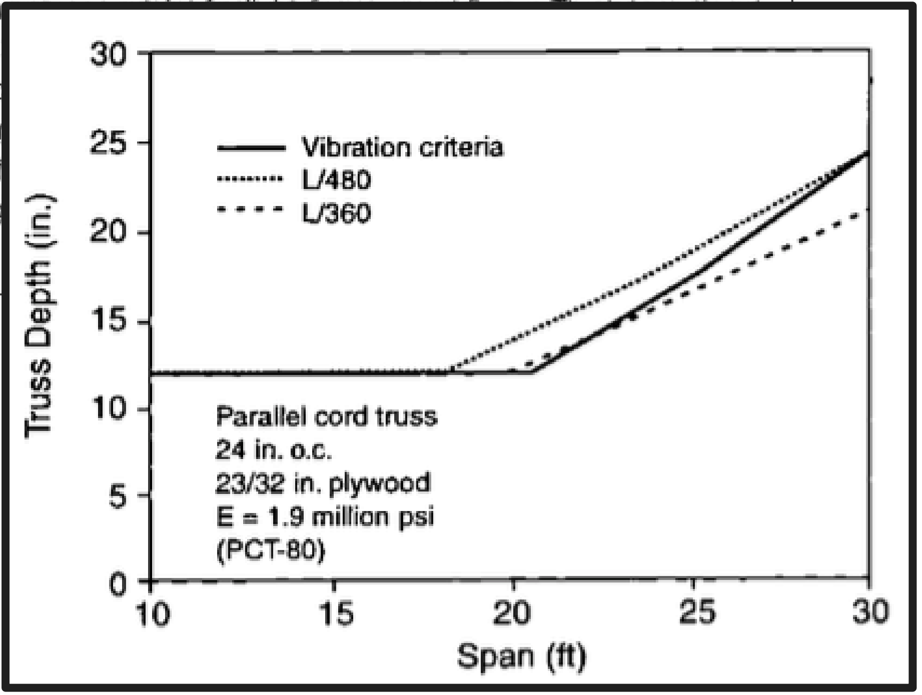
Annoying vibration is probably the most common performance complaint for light-frame wood floors.

Frank Woeste, Ph.D., P.E., and Daniel Dolan, Ph.D., P.E.

Recommendations to minimize annoying wood-floor vibrations

Annoying vibration is probably the most common perform
Code Council's 2006 International Residential Code (IRC)
this issue, yet the engineer-of-record for a project may fac
engineer may be engaged to determine the cause of an a
under the prescriptive provisions of the IRC. While wood f
deserves attention by the design professional at the desig
impossible to fix.

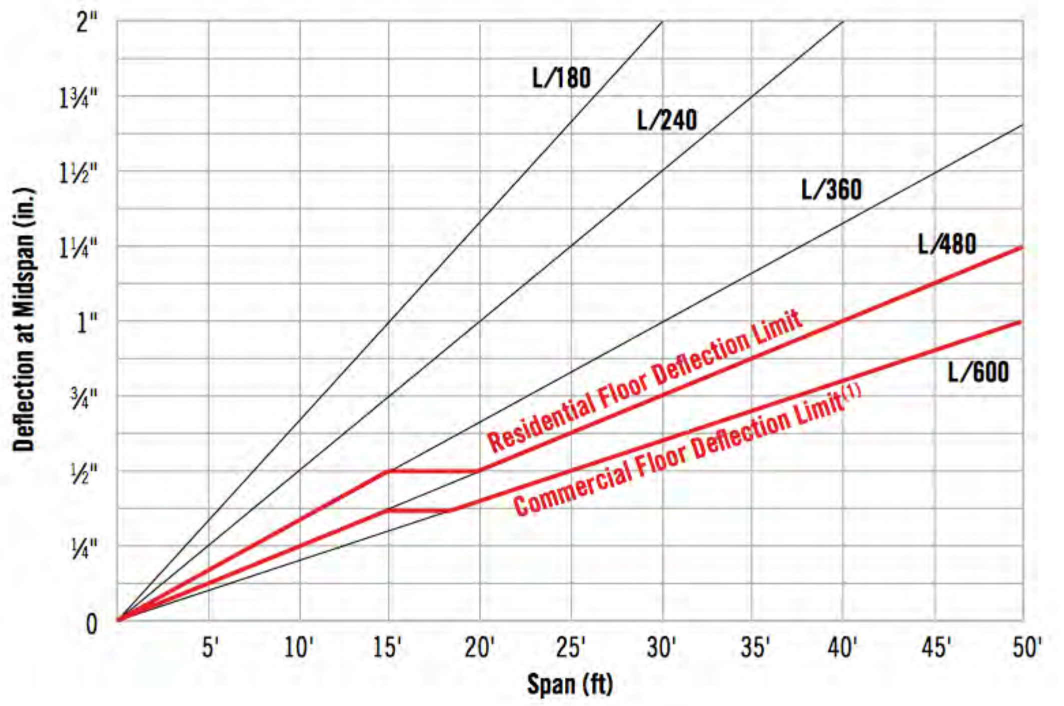
$$f = 1.57\sqrt{\frac{386EI}{WL^3}} \qquad \text{(Equation 1)}$$



Floor Design: Occupant Comfort

Vibration & Deflection Control

Multi-family floor spans in the 24’-30’ range work well from a layout perspective. Floor design of wood members in this span range are often governed by vibration and/or deflection control, not structural capacity.

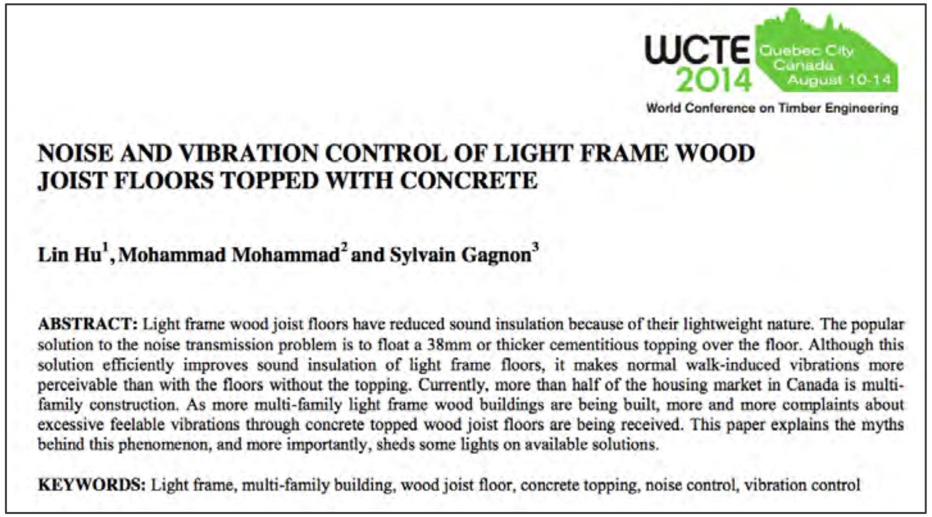


Live Load Deflection Chart, Courtesy: Redbuilt

Floor Design: Occupant Comfort

Tools available to designers

Vibration Analysis: FP Innovations (Spreadsheet available upon request)



WCTE 2014 Quebec City Canada August 10-14
World Conference on Timber Engineering

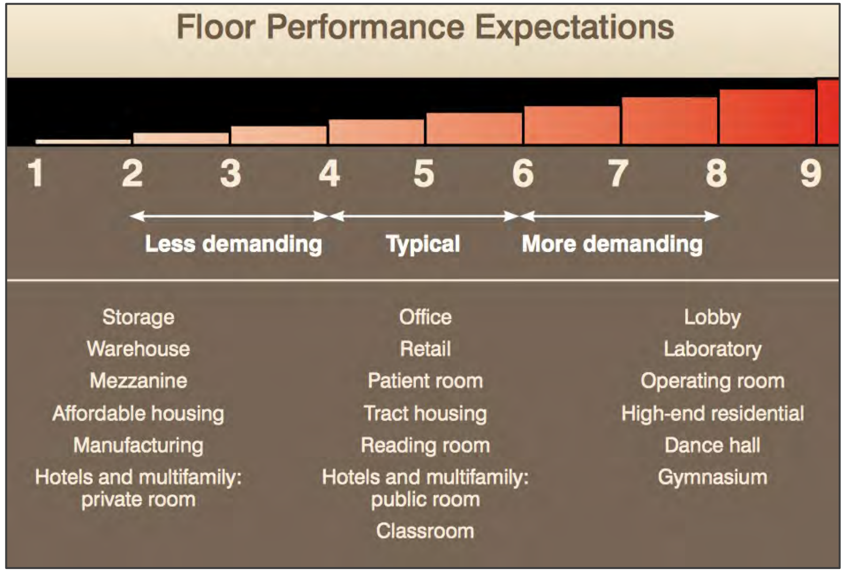
NOISE AND VIBRATION CONTROL OF LIGHT FRAME WOOD JOIST FLOORS TOPPED WITH CONCRETE

Lin Hu[1], Mohammad Mohammad[2] and Sylvain Gagnon[3]

ABSTRACT: Light frame wood joist floors have reduced sound insulation because of their lightweight nature. The popular solution to the noise transmission problem is to float a 38mm or thicker cementitious topping over the floor. Although this solution efficiently improves sound insulation of light frame floors, it makes normal walk-induced vibrations more perceivable than with the floors without the topping. Currently, more than half of the housing market in Canada is multi-family construction. As more multi-family light frame wood buildings are being built, more and more complaints about excessive feelable vibrations through concrete topped wood joist floors are being received. This paper explains the myths behind this phenomenon, and more importantly, sheds some lights on available solutions.

KEYWORDS: Light frame, multi-family building, wood joist floor, concrete topping, noise control, vibration control

Joist Manufacturer's Rating Systems



What methods exist for checking floor vibration of light-frame wood structures?

Vibration of light-frame wood floor construction can be a significant occupant comfort issue. However, achieving acceptable levels of floor vibration is not a code requirement. As such, it is possible to design a code-compliant wood floor structure that produces annoying or unacceptable levels of vibration due to standard foot traffic.

A variety of factors can affect a floor's vibration performance, including:

- Presence of concrete topping or other massing materials
- Thickness/stiffness of floor sheathing
- Stiffness, spacing and span of floor joists/trusses
- Presence, size and spacing of blocking/bridging/strong backs
- Presence of direct-applied ceiling
- Stiffness of joist supporting elements (i.e., beams, bearing walls)
- Presence of partition walls

Several vibration analysis methods have been published, each of which takes into account some or all of these variables.

View All Expert Tips

Project Assistance

Our technical experts offer free project support from design through construction, on issues ranging from allowable heights and areas to structural design, lateral systems and fire- or acoustical-rated assemblies.

Get Assistance >

Ask an Expert

Q: When is blocking/bracing within wood-frame walls required? What is considered adequate bracing for wood wall studs in their weak axis?

A: Wood studs used in light-frame wall construction may require horizontally-oriented blocking for a number of reasons—including blocking at shear panel edges, fire blocking, and buckling restraint when subject to axial loads. Structural Blocking Purposes Blocking to Reduce Stud Slenderness Ratio Section 3

- Learn More

Have a question? Email Us >

Feature Project

Stacked Bearing Wall Design

Bearing Wall Studs: Stacking Loads

In mid-rise structures, bearing wall loads accumulate – may result in increased stud requirements at lower levels

Example: 5 Story Building, Exterior Bearing Wall Supports 28' Span Trusses

Roof: DL = 20 psf, SL = 40 psf
Floor: DL = 30 psf, LL = 40 psf
Wall: DL = 10 psf

Total Bearing Wall Load at Lowest Level = 4650 plf or 6200 lbs per stud @ 16" o.c.

Need 2-2x6 studs @ 16" o.c.

Bearing Wall Studs: Stacking Loads

Options for lower level, stacked bearing wall studs:

- Specify SP or DF plates – up to 40% increase in allowable loads
  - Fc perp= 565 psi to 625psi
- Specify LSL or LVL plates – 75% increase in capacity
- Decrease stud spacing from 16" o.c. to 12” o.c. - 33% increase in capacity
- Double studs – 100% increase in capacity
- Increase the depth of the wall – 2x6 at upper, 2x8 at lower
- Add interior bearing walls at lower levels

Can live load reduction be used on wood-frame bearing walls?

ASCE 7-10, Section 4.7 permits reduction of live loads on certain structural members that have influence areas of at least 400 sf. ASCE 7-10 defines influence area as $K_{LL}A_T$ where K_{LL} is a live load element factor given in Table 4-2 and A_T is the member's tributary area. Studies have shown that, as a member's influence area increases, the likelihood that the member will experience the full design live load over its entire tributary area decreases. Due to this, ASCE 7-10 equation 4.7-1 can be used to calculate a reduced uniform live load. This reduced live load is not permitted to be less than 50% of the unreduced live load from a single floor, or not less than 40% of the unreduced live load from multiple floors.

When applied to repetitive framing walls, the prevailing consensus in the engineering community is that live load reduction is intended for an individual element—e.g., a header or single stud—and not for the total load on the bearing wall system (for an example, see this article). Few individual members in wood-frame bearing walls will have an influence of at least 400 sf, indicating that live load reduction would not apply. However, should the minimum influence area for an individual element within the wall be reached, a reduction may apply. Rationally, many would consider it excessive to assume that the bearing wall studs on the lowest level of a 4- or 5-story wood-frame building would see 100% of the design live load from all supported levels simultaneously. However, ASCE 7-10 only permits a reduction of design live loads if the 400 sf influence area is reached.

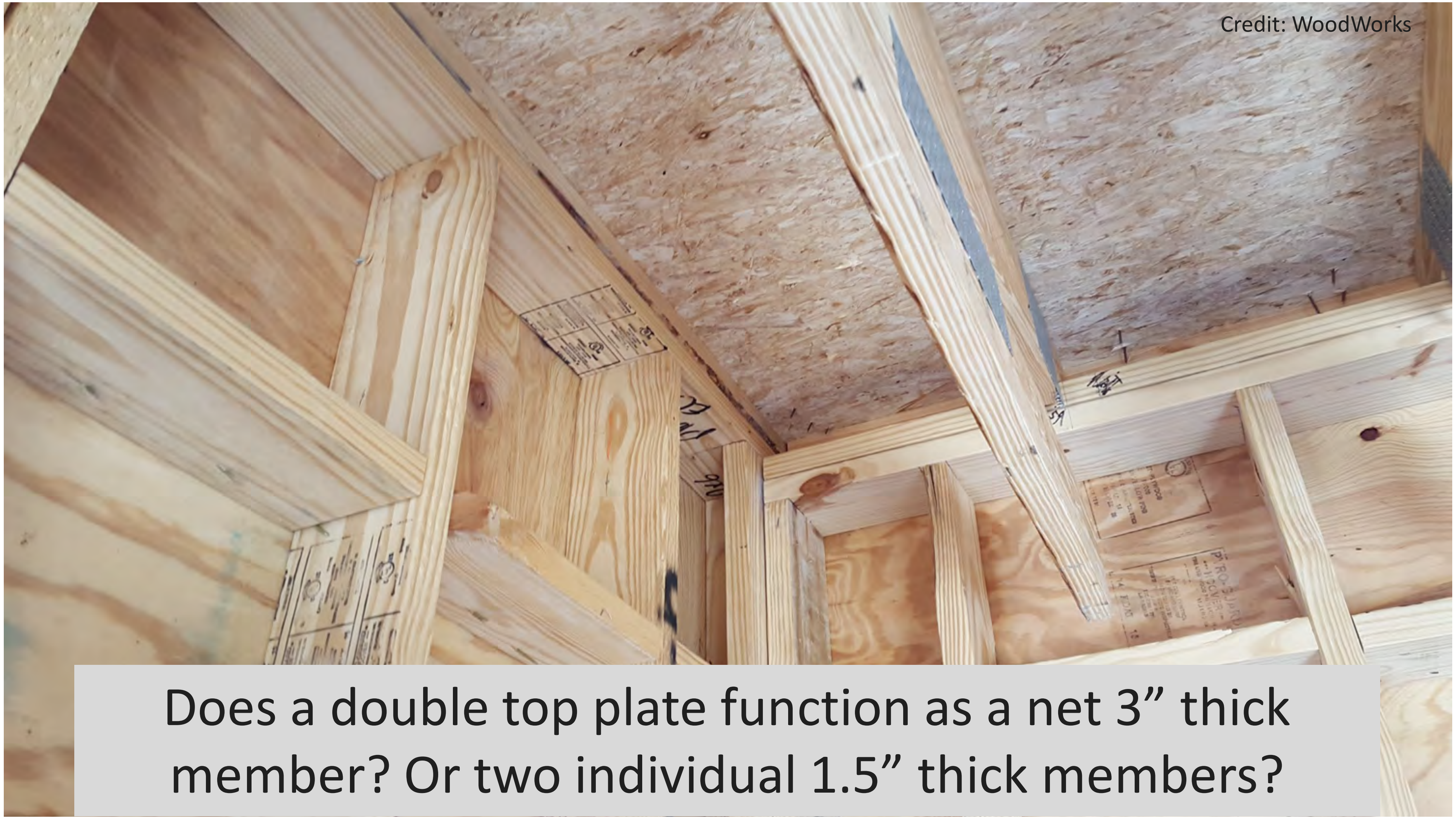
Bearing Wall Studs: Stacking Loads

If Type III Exterior Walls:
Specify the FRT treatment with the lowest Fc perp reduction.
Manufacturers reduction values can vary between 5% and 13%

Bearing Wall Design

When Floor/Roof Framing and Studs don't Align
Design of Top Plates in Bending & Shear

Bearing Wall Design



Credit: WoodWorks

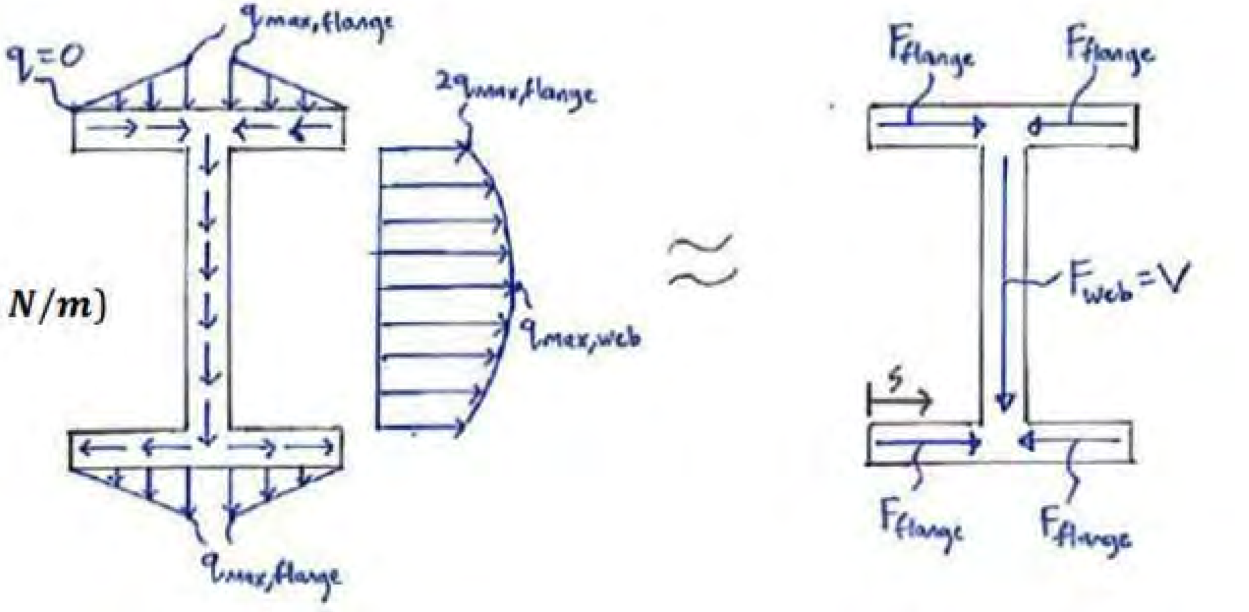
Does a double top plate function as a net 3” thick member? Or two individual 1.5” thick members?

Bearing Wall Design

- Shear flow equations assume rigid connections between members.
- Very difficult to justify this with common dowel fasteners (nailed plate to plate connection)

Shear flow

Formula: $\boxed{q = \frac{VQ}{I}}$ (Units: $\boldsymbol{N/m}$)



Bearing Wall Design

- General consensus is to assume two plates act independently. Half load goes to each (equal deflection)
- A 2-2x6 SPF top plate with studs at 16" o.c. has a truss reaction capacity of approximately 1,000 to 1,400 lb depending on load location

Credit: WoodWorks

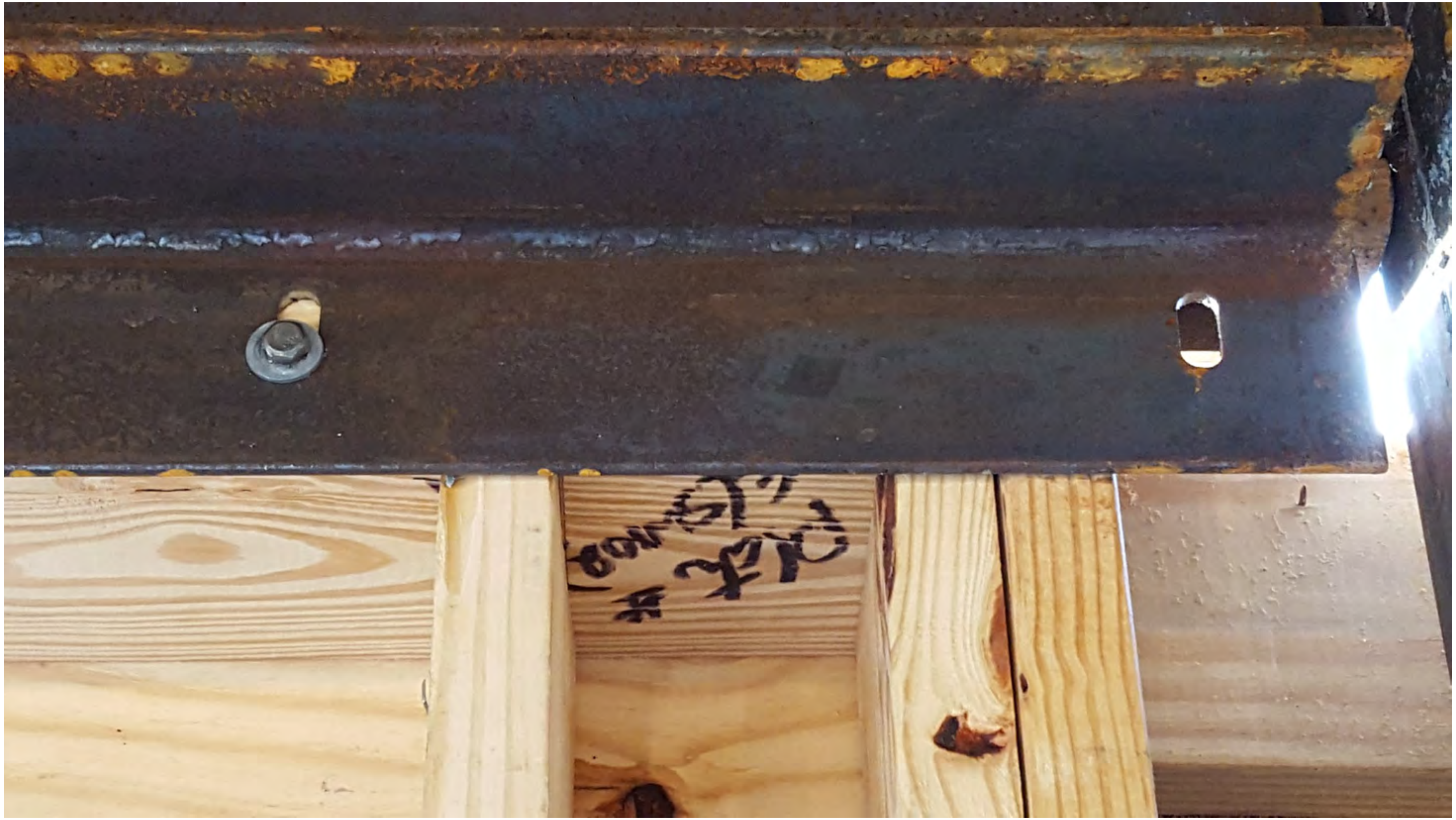
Non-Bearing Wall Design

Credit: WoodWorks

Credit: WoodWorks

Shear Wall to Podium Slab Interface

Shear Wall to Podium Slab Interface



Credit: WoodWorks

What are the options for detailing non-bearing wood partition walls to the underside of floor or roof framing?

In many wood-frame multi-family and commercial buildings, building layouts result in structural floor and roof spans in the range of 15 feet to 30 feet for floors and much longer for roofs. However, interior partition walls are often required to split interior spaces into separate rooms or units. Structural engineers commonly assume that interior partition walls do not act as load-bearing elements, either due to the potential for future partition re-arrangement or structural inefficiency associated with close support spacings. To avoid issues with load resistance at partition walls that were not intended to act as structural elements, careful detailing is required at the intersection of the top of partition and underside of floor or roof framing.

Reasons for ensuring that load transfer to these partition walls does not occur include avoiding partition wall load-bearing inadequacy and altered shear and moment forces in the floor or roof trusses or joists. Although multiple support locations along a framing system may seem like a positive thing, issues

Wall Blocking Requirements


Credit: WoodWorks

Wall Blocking Requirements

- When do you need blocking?
- What is the required blocking capacity?
- What is the required blocking size and orientation?
- Does blocking depth need to match wall stud depth?
- What about unique conditions like staggered stud walls?

Credit: WoodWorks

Wall Blocking Requirements

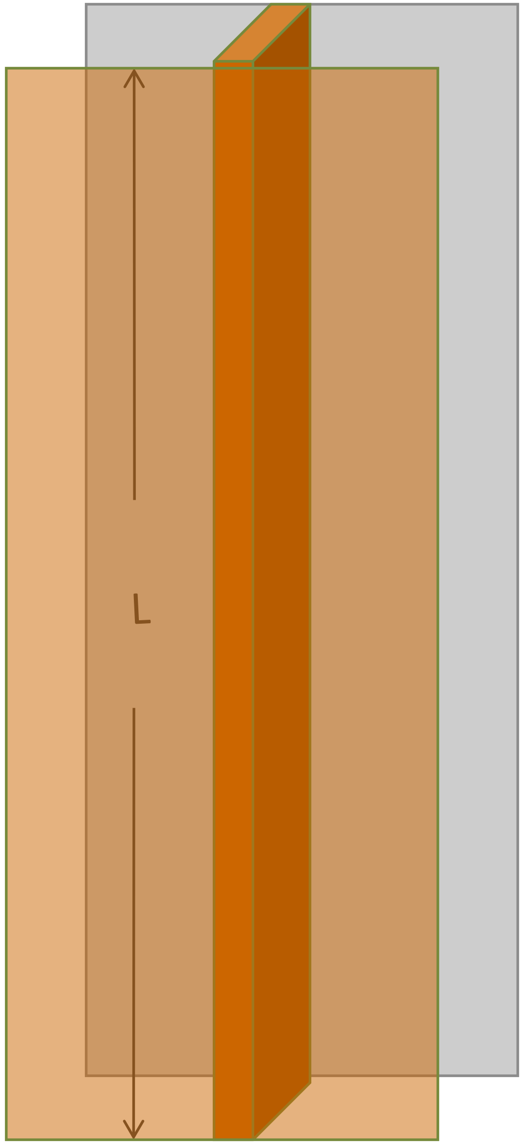
- Slenderness ratio limits
- Weak axis stud buckling
- Shearwall panel edge blocking
- Fire blocking

NDS Appendix A.11.3:

When stud walls in light-frame construction are adequately sheathed on at least one side, the depth, rather than breadth of the stud, shall be permitted to be taken as the least dimension in calculating the le/d ratio. The sheathing shall be shown by experience to provide lateral support and shall be adequately fastened.


Credit: WoodWorks

Wall Blocking Requirements



NDS Commentary:
"Experience has shown that any code allowed thickness of gypsum board, hardwood plywood, or other interior finish adequately fastened directly to studs will provide adequate lateral support of the stud across its thickness irrespective of the type or thickness of exterior sheathing and/or finish used."

Acoustical Design

Noise **Acoustics** **Sound Pollution**

Whatever you call it, it all comes down to one thing:

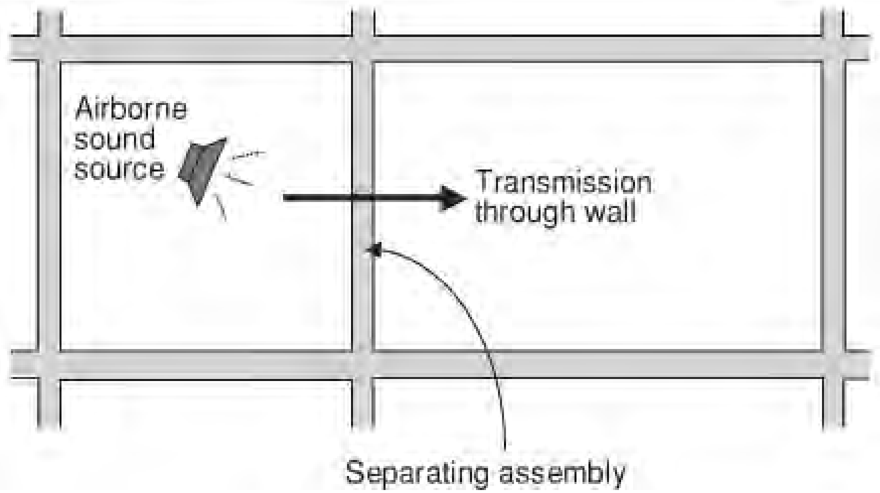
Occupant Comfort

Acoustical Design

<u>Air-Borne Sound:</u>

Sound Transmission Class (STC)

- Measures how effectively an assembly isolates air-borne sound and reduces the level that passes from one side to the other
- Applies to walls and floor/ceiling assemblies

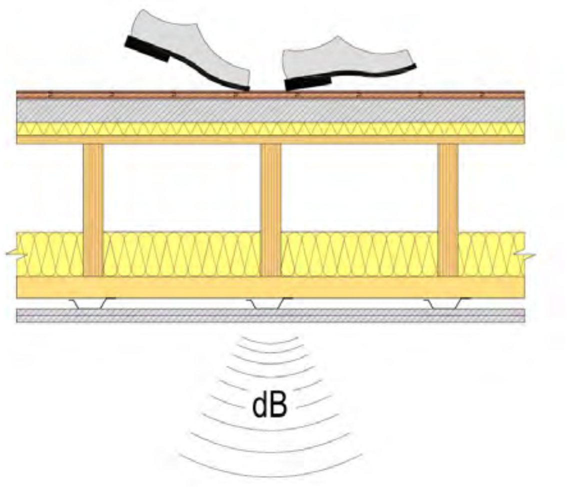


Acoustical Design

<u>Structure-borne sound:</u>

Impact Insulation Class (IIC)

- Evaluates how effectively an assembly blocks impact sound from passing through it
- Only applies to floor/ceiling assemblies



Acoustical Design

Code requirements only address residential occupancies:

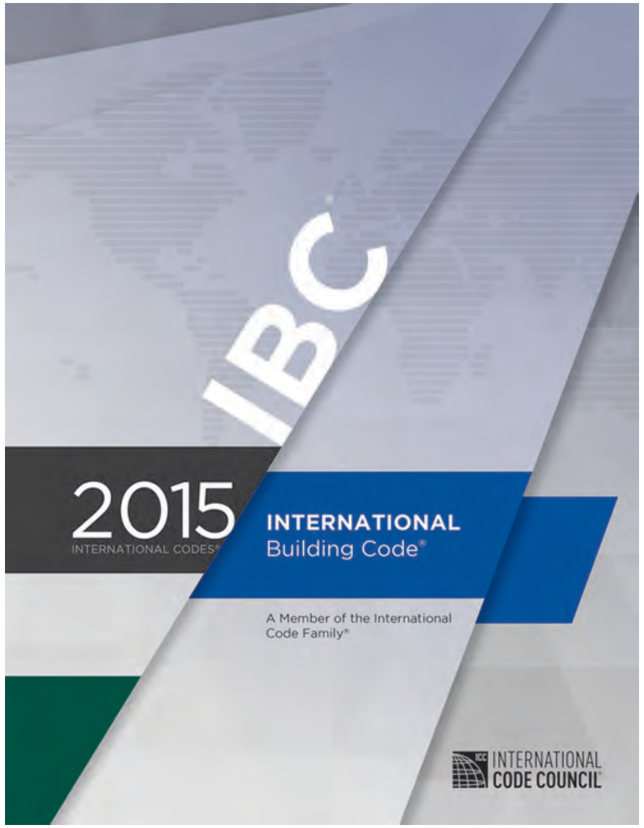
For unit to unit or unit to public or service areas:

Min. STC of 50 (45 if field tested):

- Walls, Partitions, and Floor/Ceiling Assemblies

Min. IIC of 50 (45 if field tested) for:

- Floor/Ceiling Assemblies

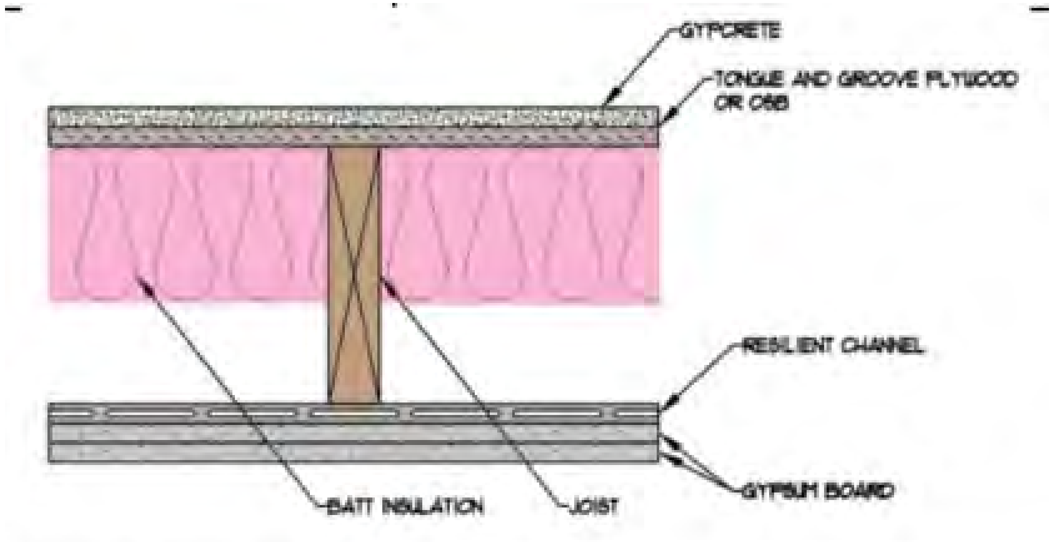


Acoustical Design

When does structure impact the acoustical performance of a wall or floor assembly?

Regardless of the structural materials used in a wall or floor ceiling assembly, there are 3 effective methods of improving acoustical performance:

1. Add Mass
2. Add noise barriers
3. Add decouplers

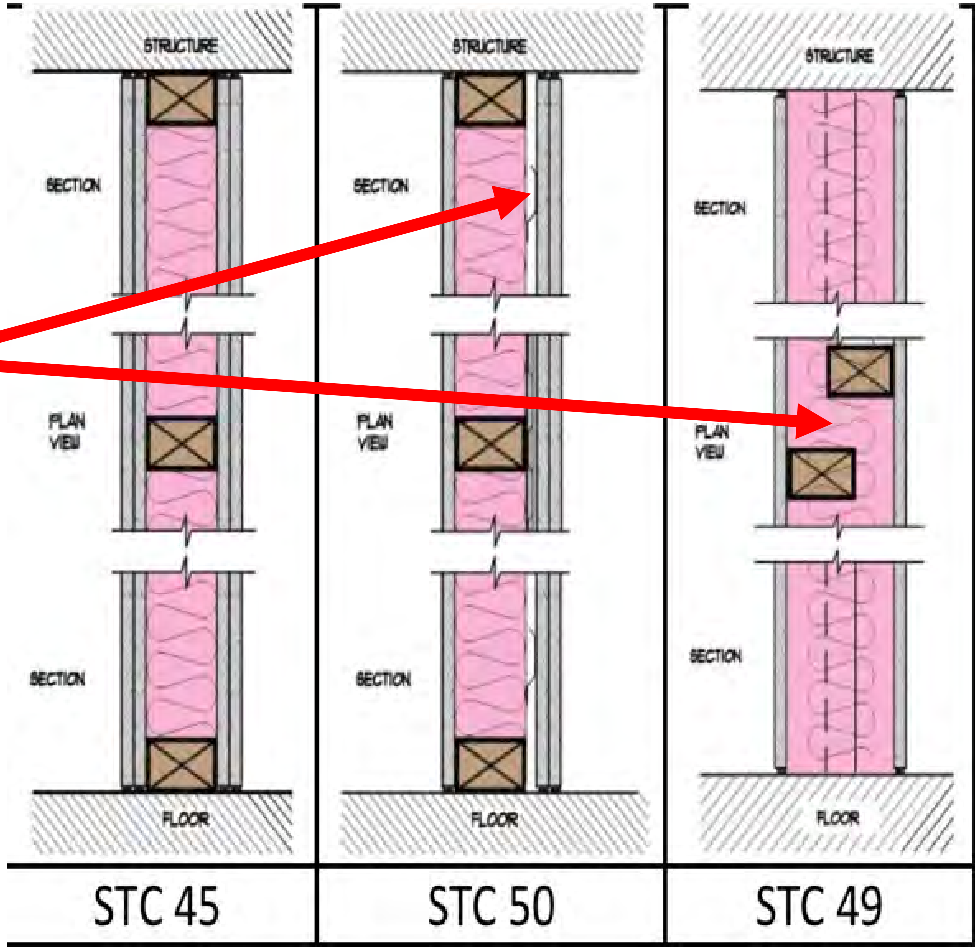


STC 62

Acoustical Design

What does this look like in typical wood-frame construction:

1. Add Mass
2. Add noise barriers
3. Add decouplers

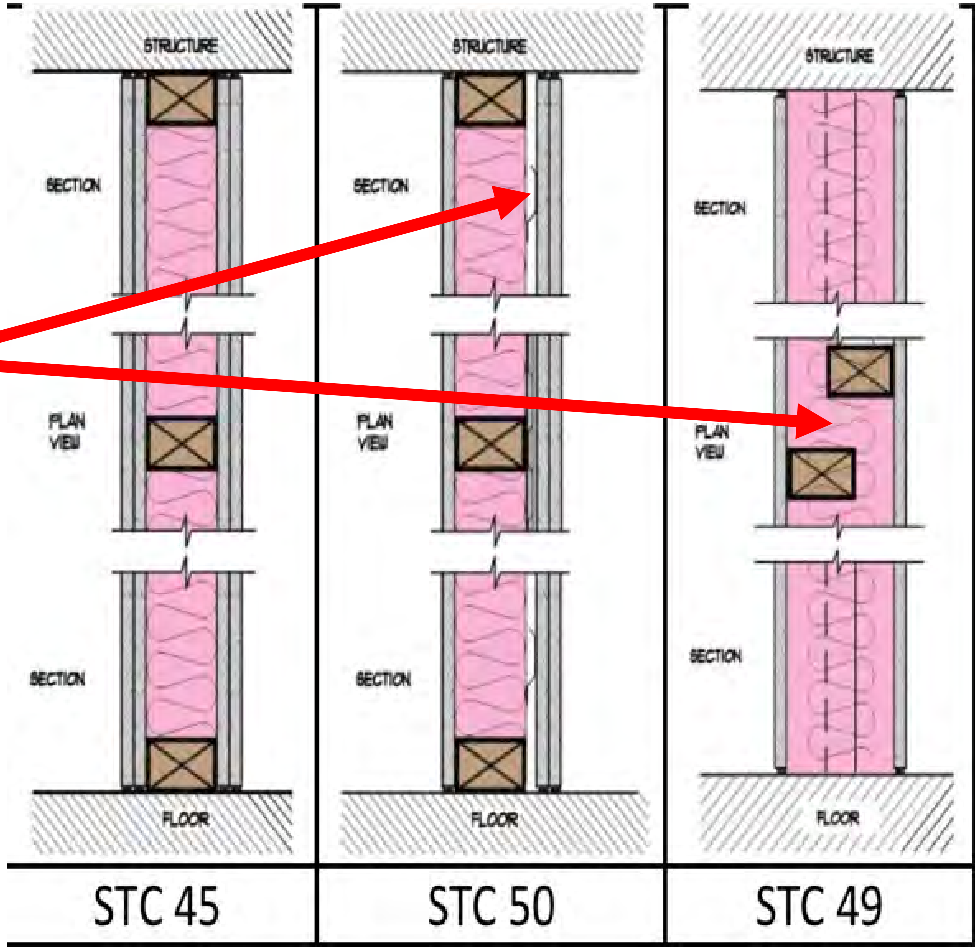


Acoustical Design

What does this look like in typical wood-frame construction:

1. Add Mass
2. Add noise barriers
3. Add decouplers

Make sure that structural elements don't defeat the purpose of these, especially decouplers



Acoustical Design

- My interior, acoustically rated wall also needs to be a shearwall (think unit demising wall)
- Can I add wood structural panels to an acoustically tested wall?

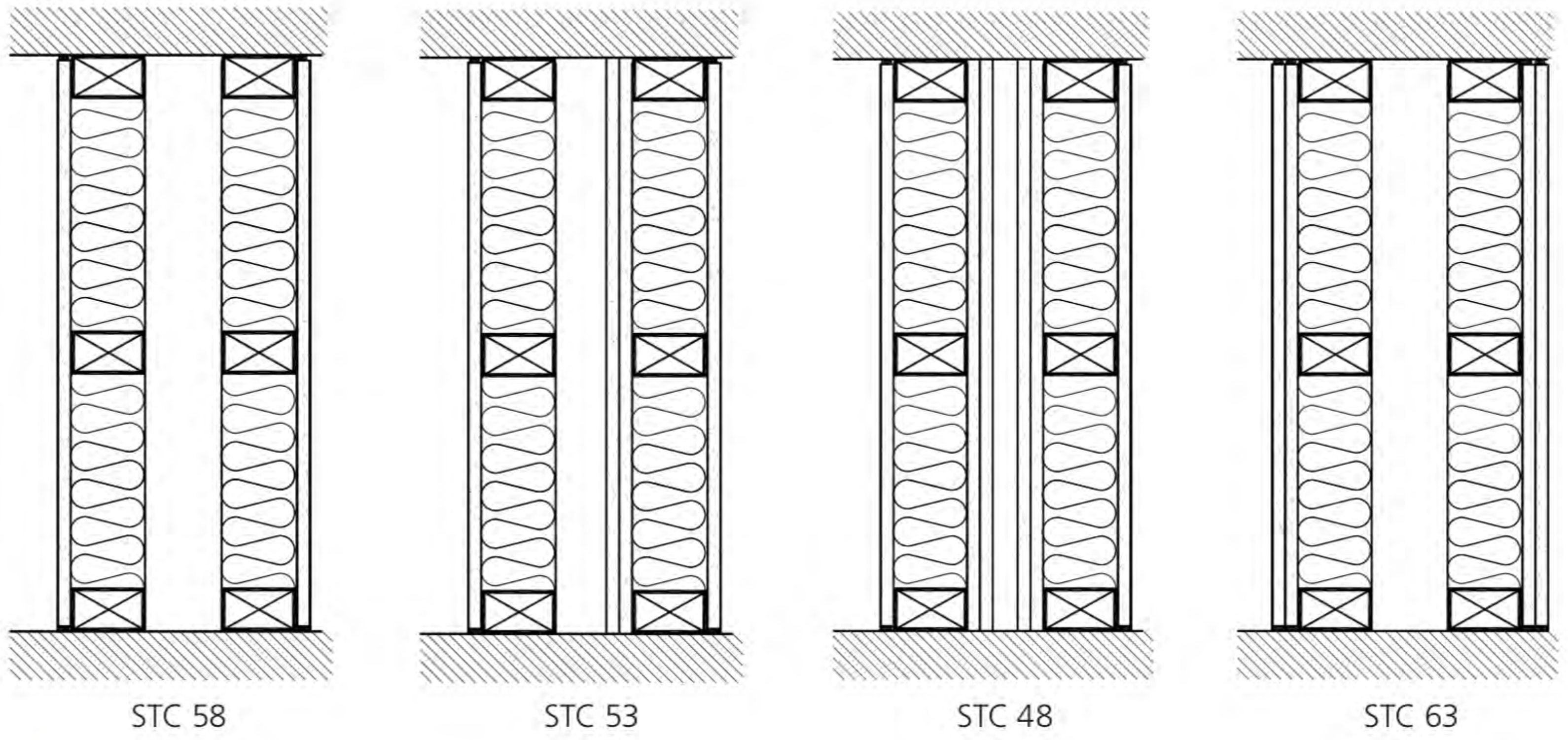
Yes, but placement is very important!

Credit: WoodWorks

Acoustical Design

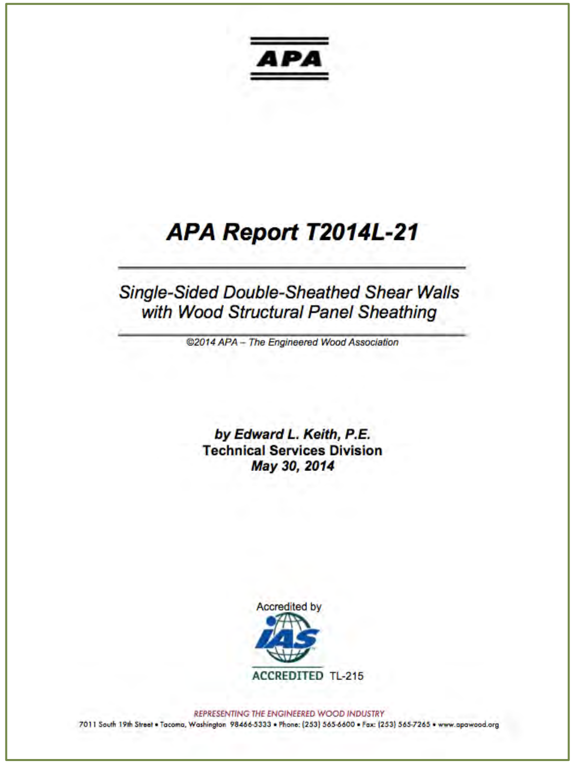
FIGURE 6

Effect of Sheathing Placement on Acoustical Performance (Plan View)



Acoustical Design

- For walls with resilient channels, put WSP on opposite side of wall
- For highly loaded shearwalls, can use double layer of sheathing on same side of wall



APA

APA Report T2014L-21

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

©2014 APA – The Engineered Wood Association

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

Accredited by IAS ACCREDITED TL-215

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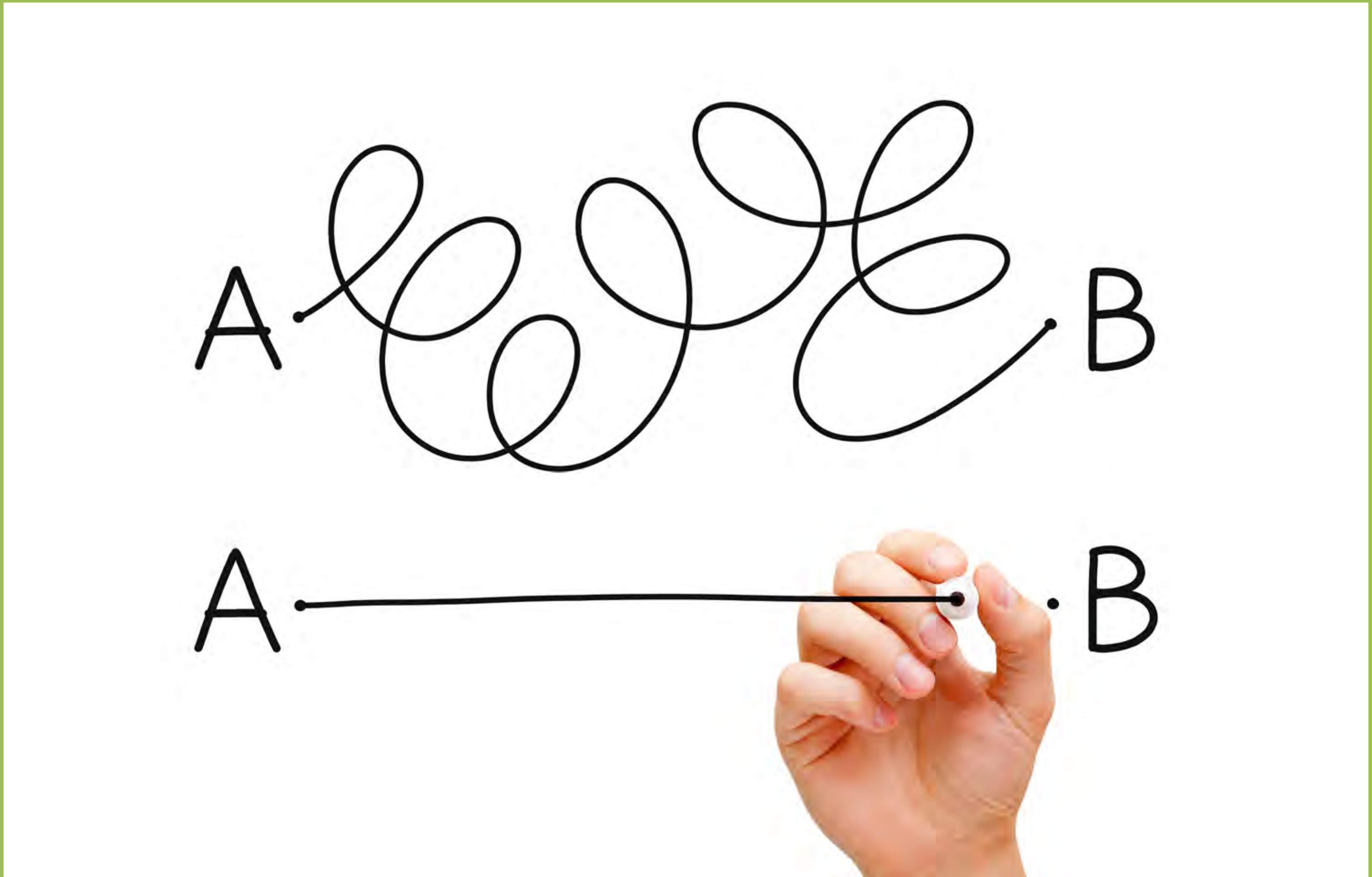
Acoustical Design

- Staggered stud wall condition:
- Blocking bridges finish on one side of wall to studs on opposite side, defeats purpose.
- Solution: use flat blocking in wall (wide face against WSP)

Credit: WoodWorks

Lateral Design Topics

Following the load…

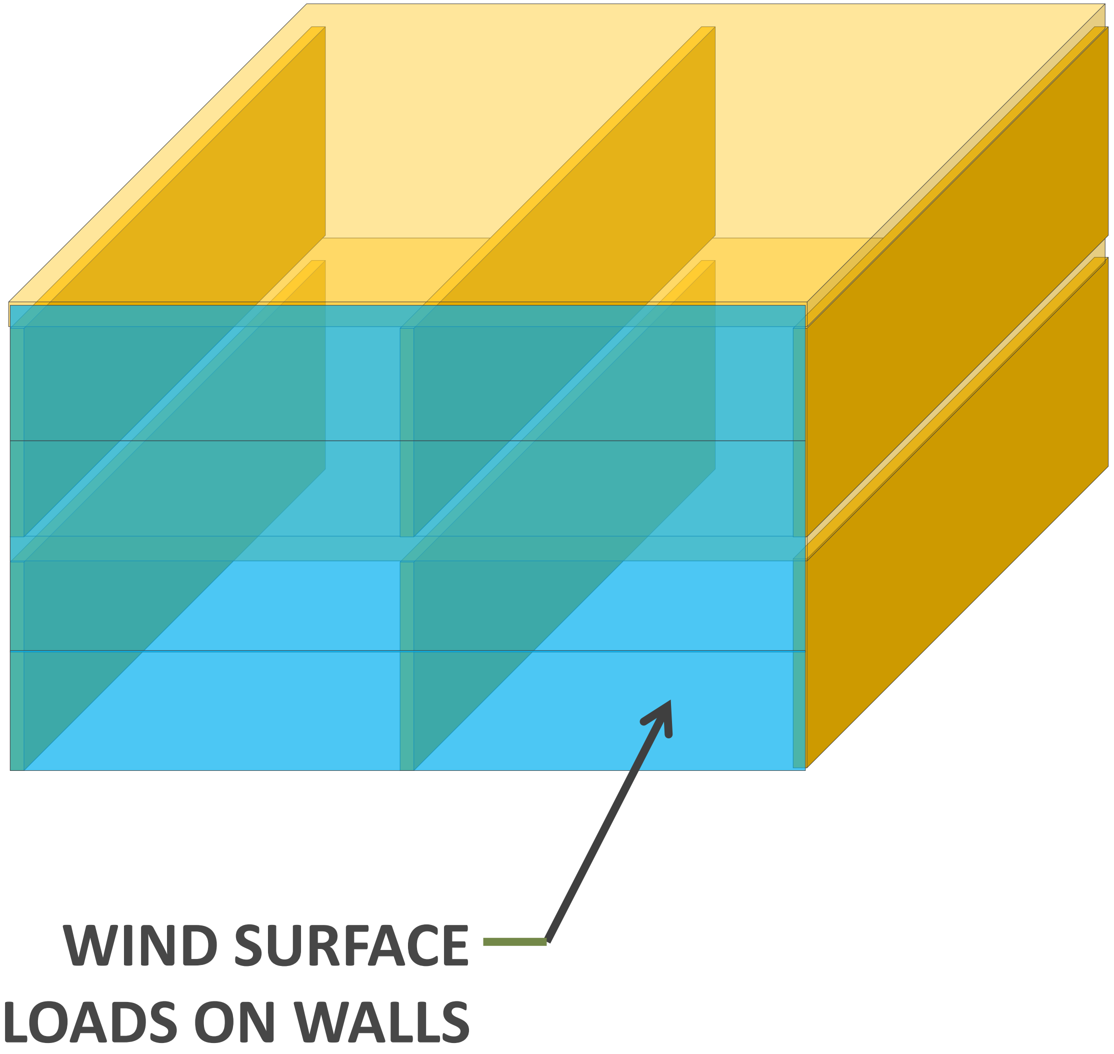


Load Path Continuity

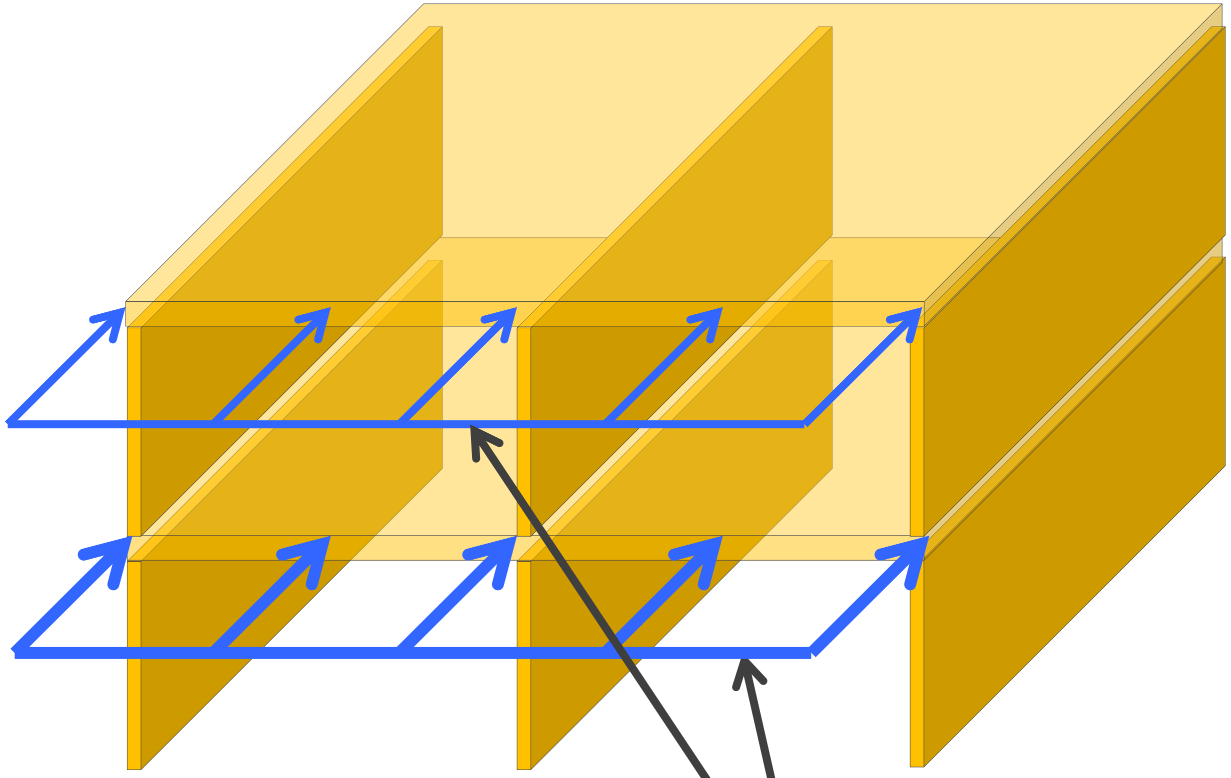
Karuna I
Holst Architecture

Photo: Terry Malone

Multi-Story Wind Load Design

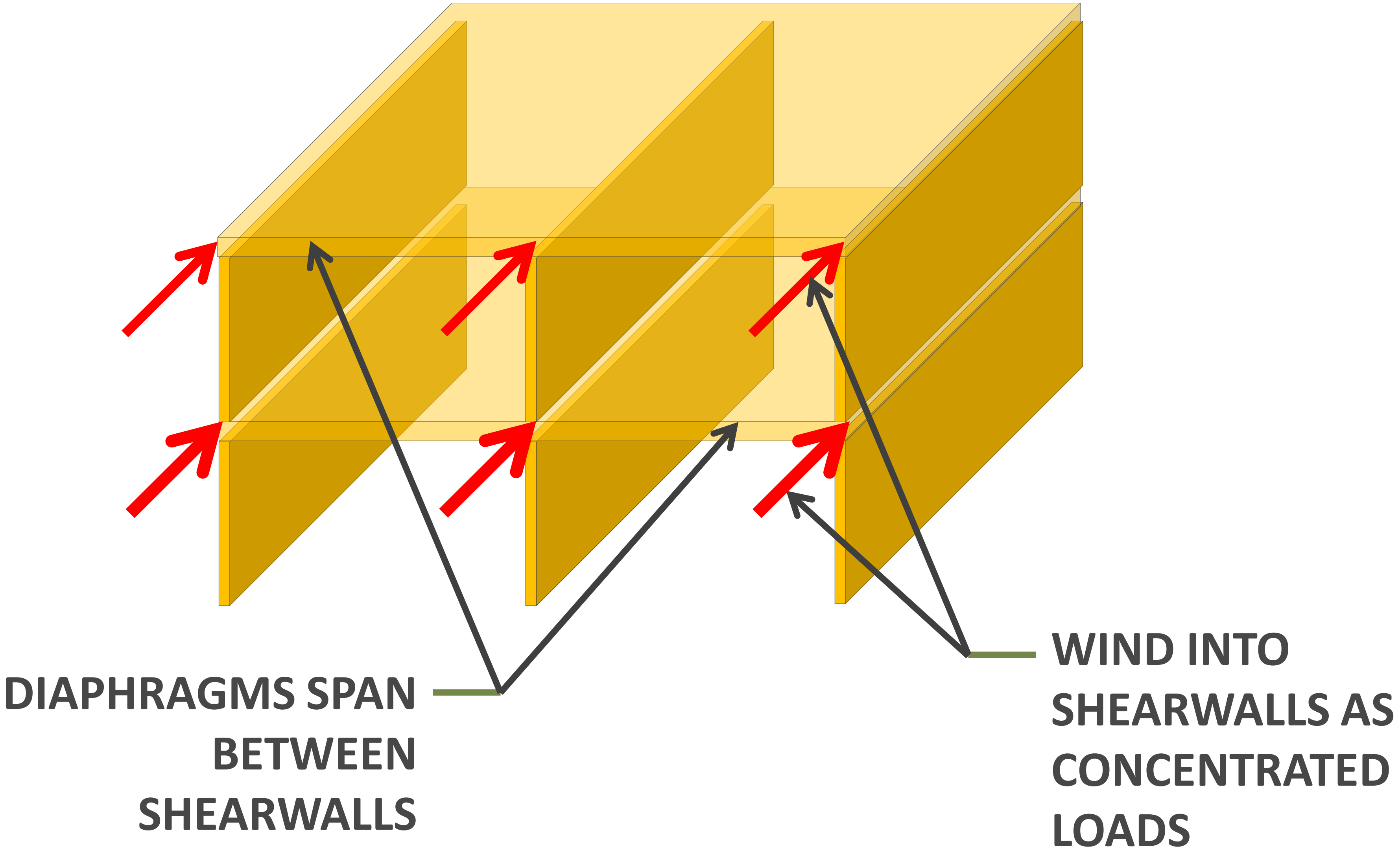


Multi-Story Wind Load Design

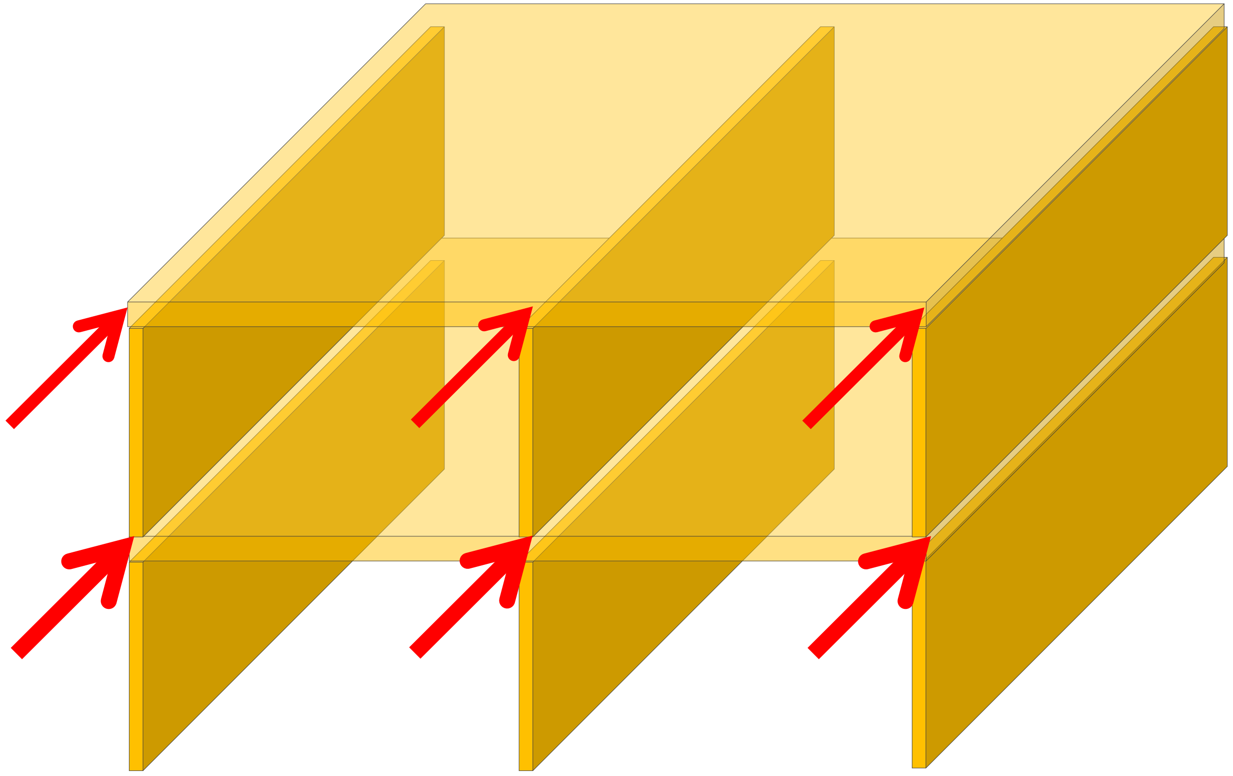


WIND INTO DIAPHRAGMS AS UNIFORM LINEAR LOADS

Multi-Story Wind Load Design



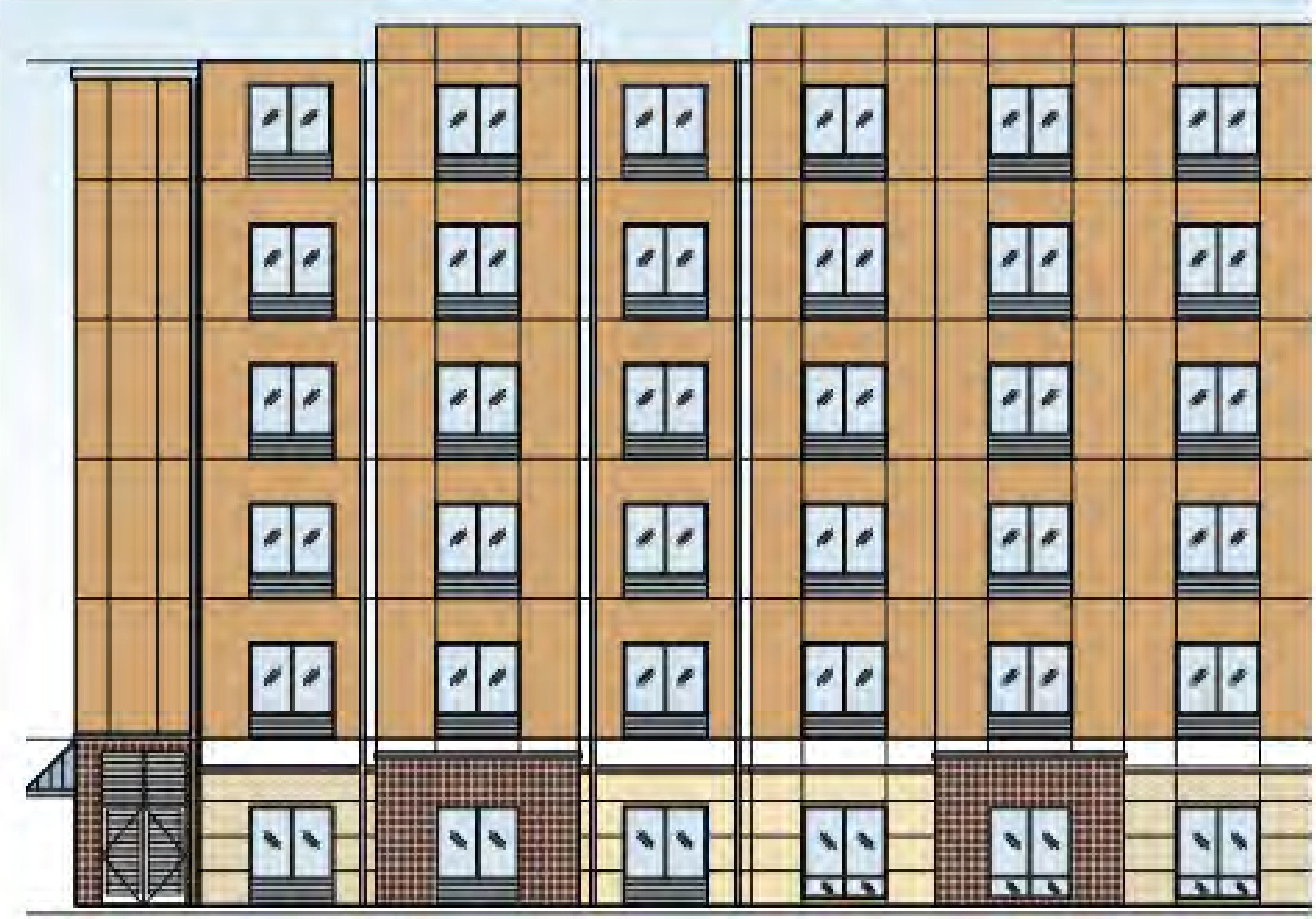
Multi-Story Wind Load Design



DIAPHRAGM WIND FORCES DO NOT ACCUMULATE-THEY ARE ISOLATED AT EACH LEVEL

SHEARWALL WIND FORCES DO ACCUMULATE-UPPER LEVEL FORCES ADD TO LOWER LEVEL FORCES

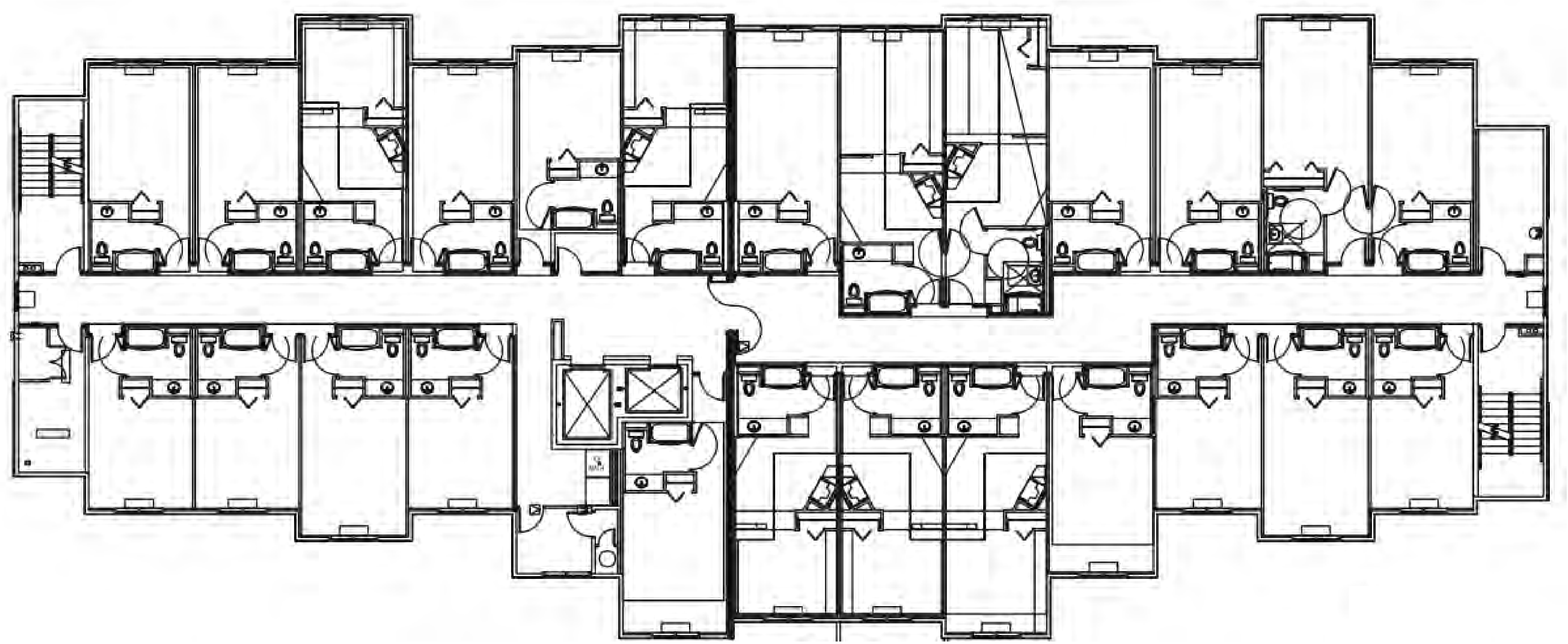
Multi-Story Wind Design



Elevation

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

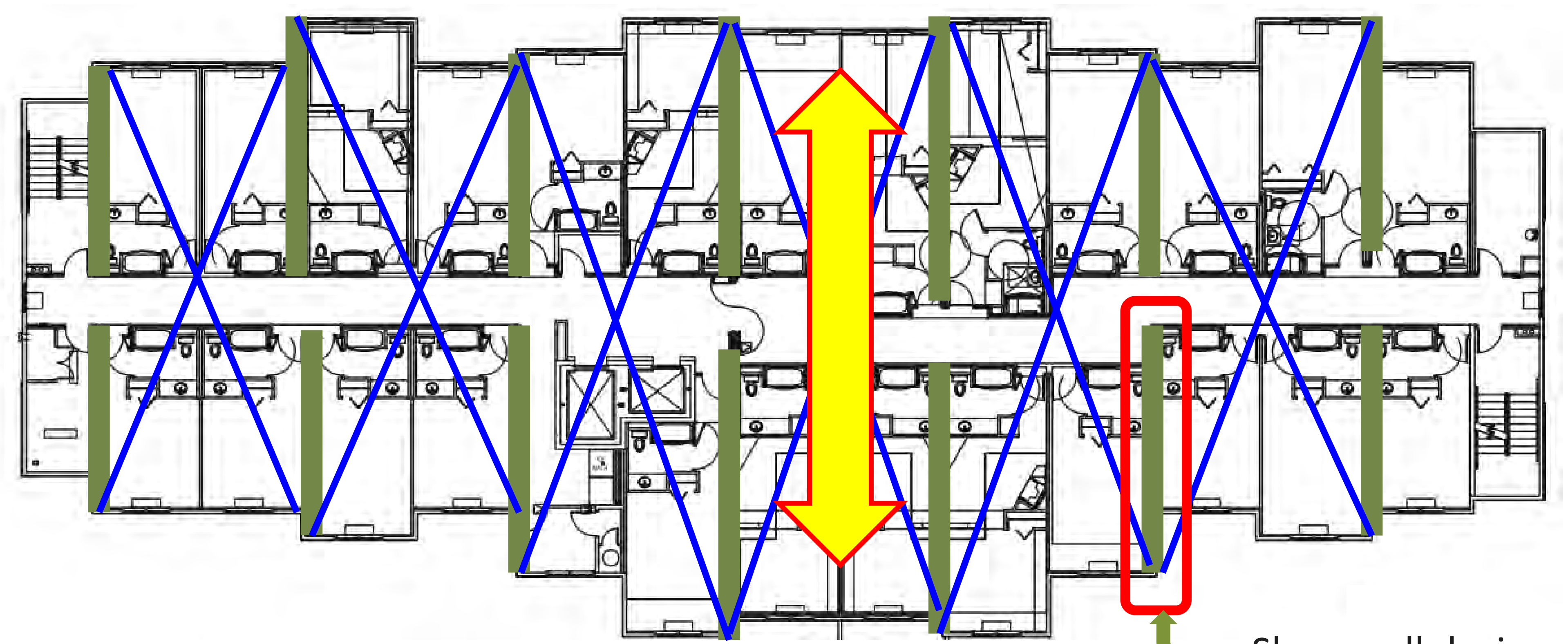
Multi-Story Wind Design



Floor Plan

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

Multi-Story Wind Design

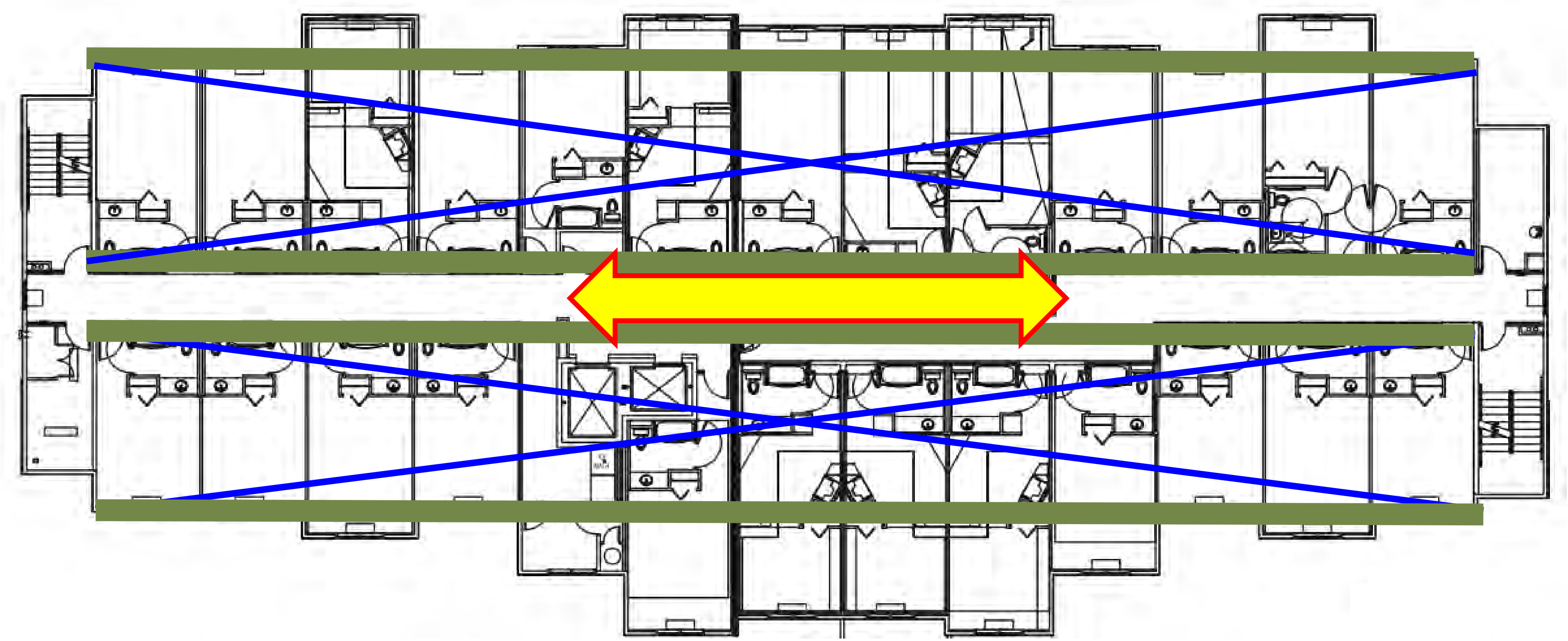


Shearwall design we'll look at

Shearwall Layout

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

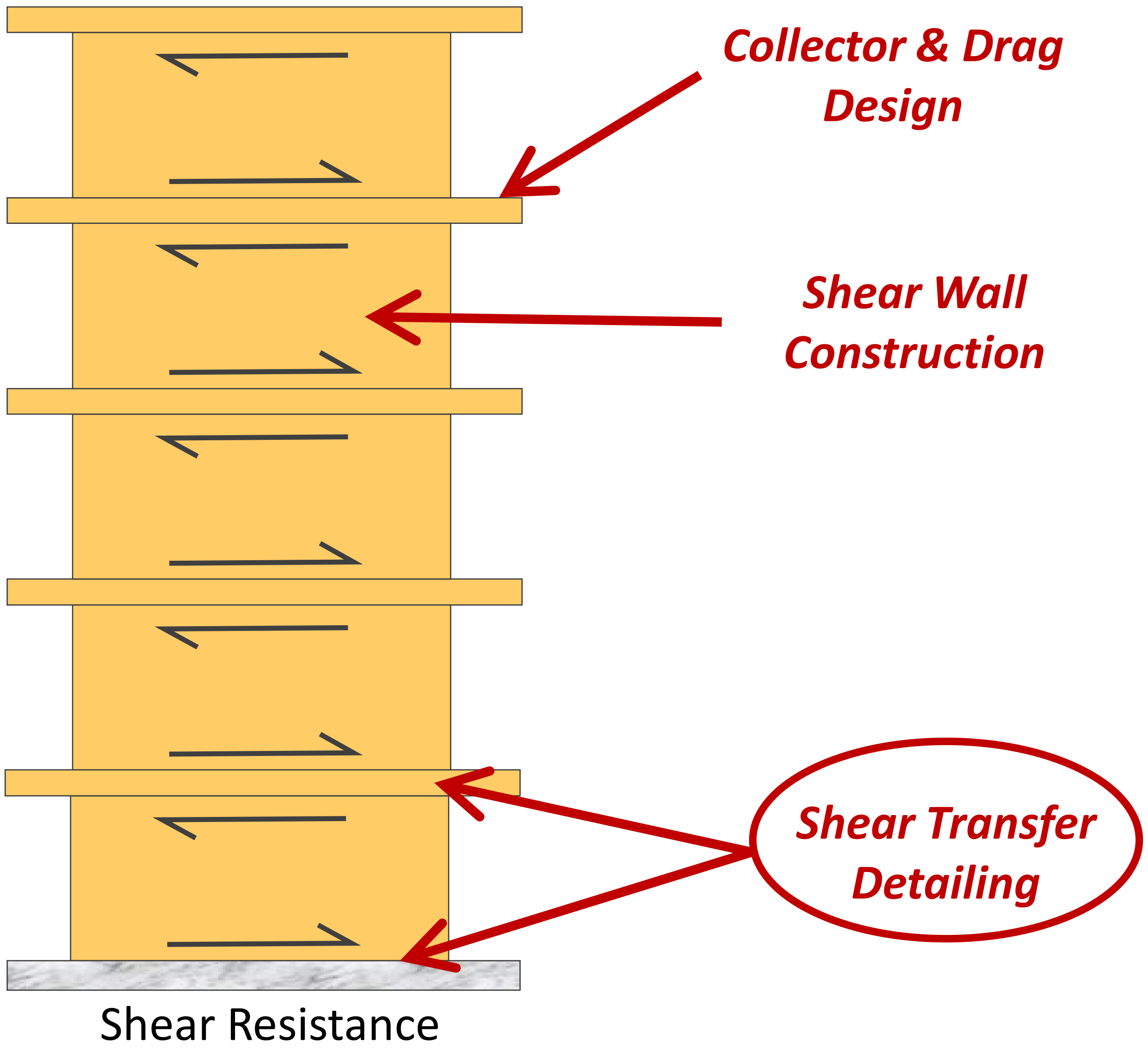
Multi-Story Wind Design



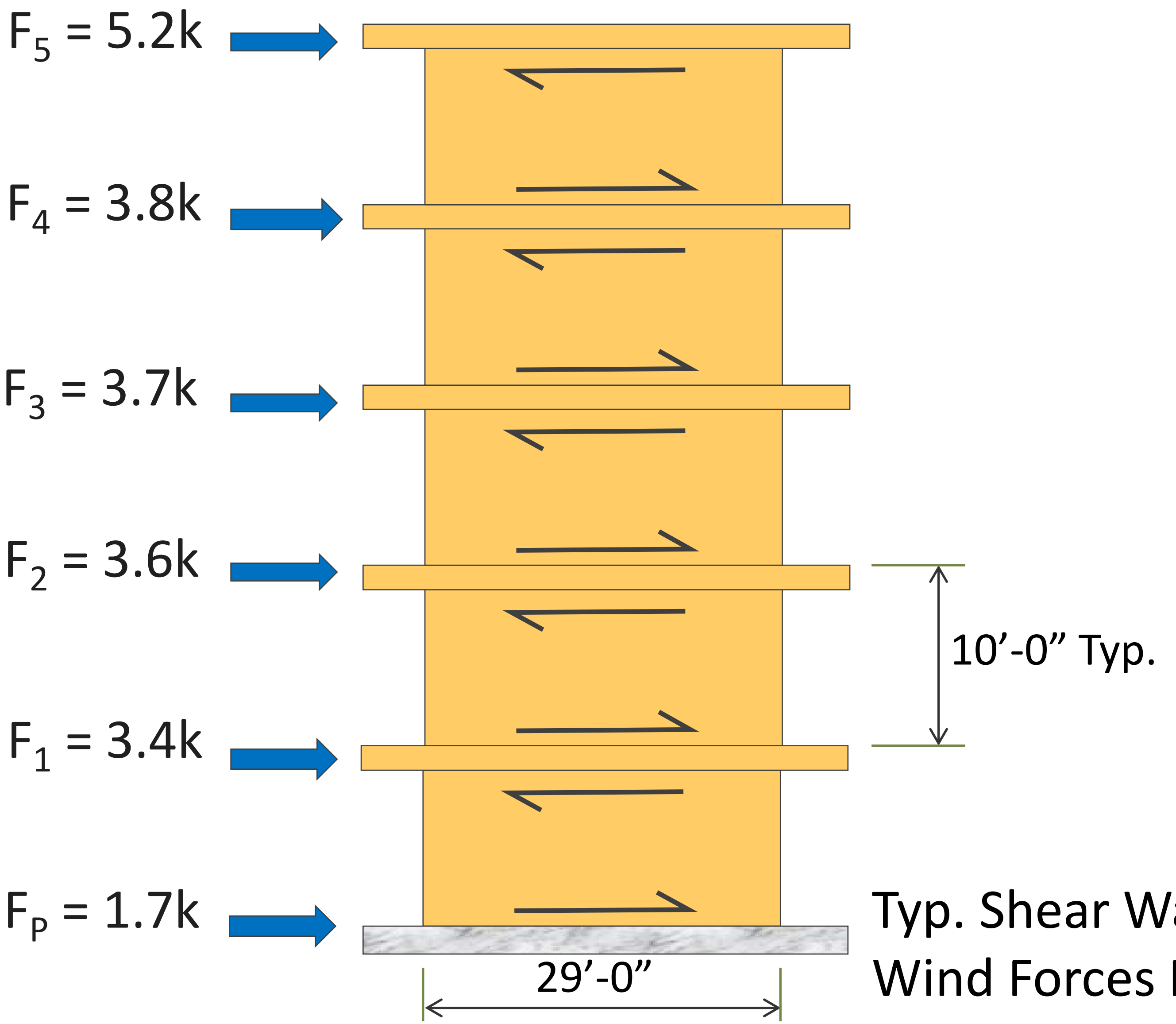
Shearwall Layout

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

Components of Shear Wall Design

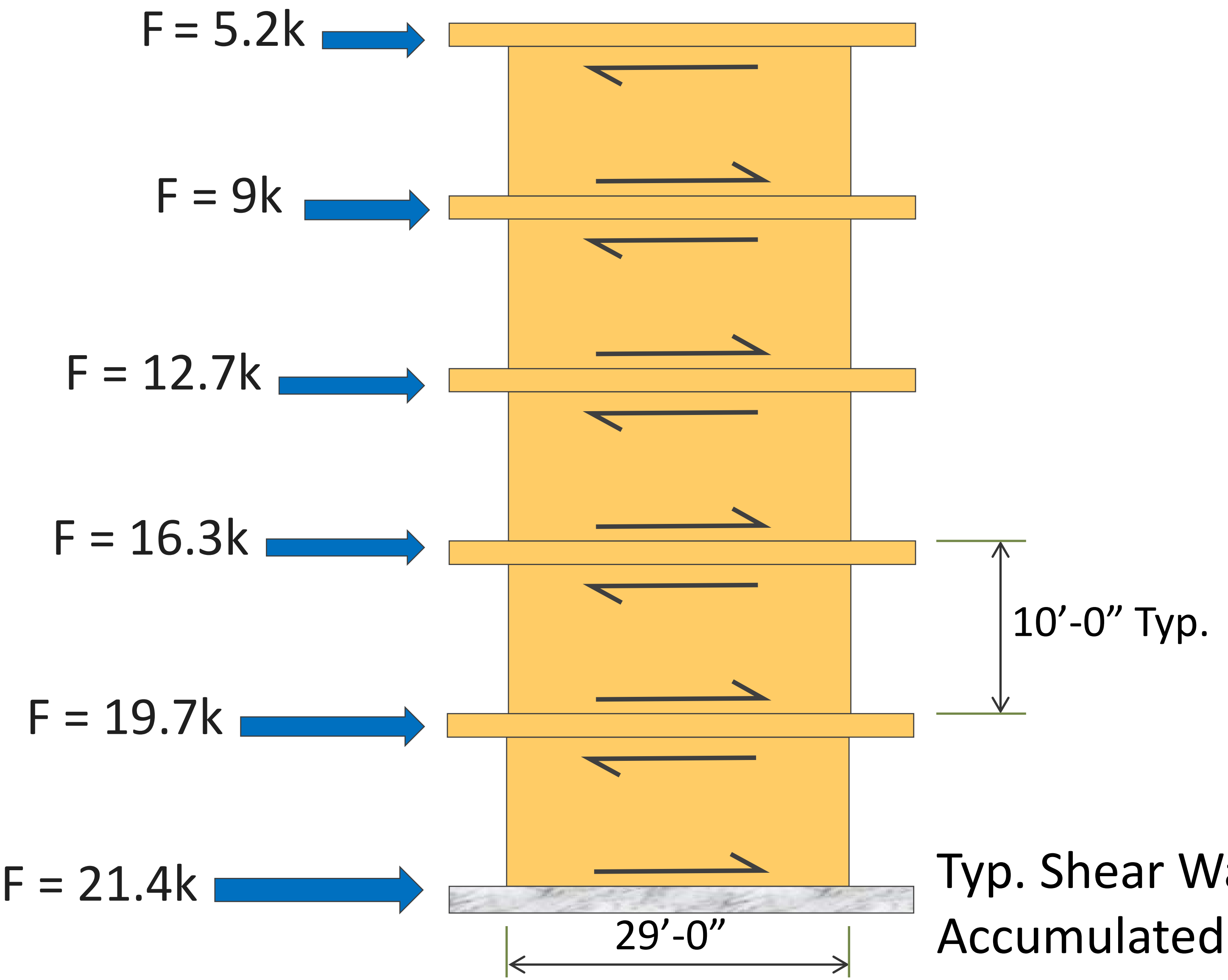


Components of Shear Wall Design



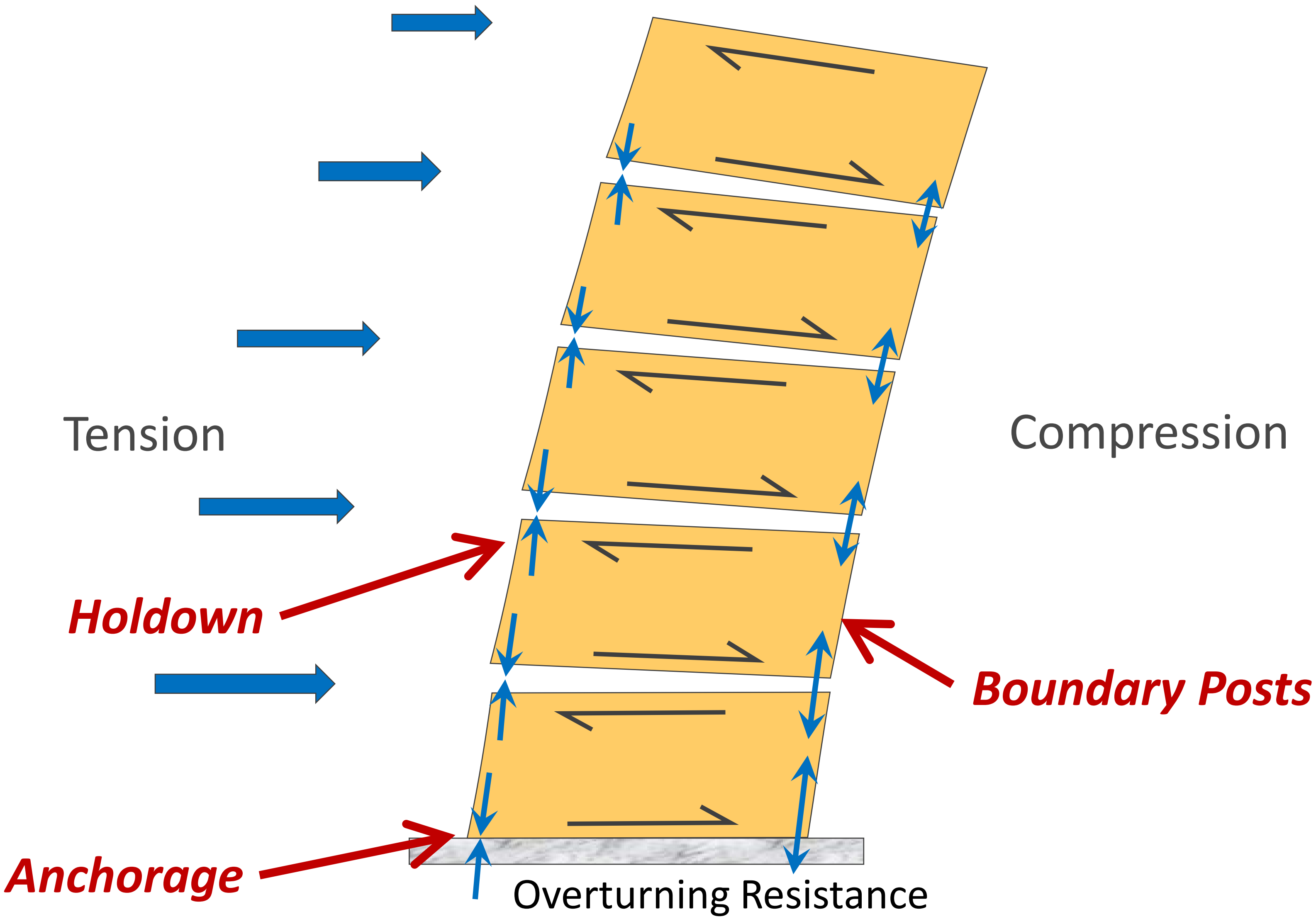
Typ. Shear Wall Elevation
Wind Forces Per Story

Components of Shear Wall Design

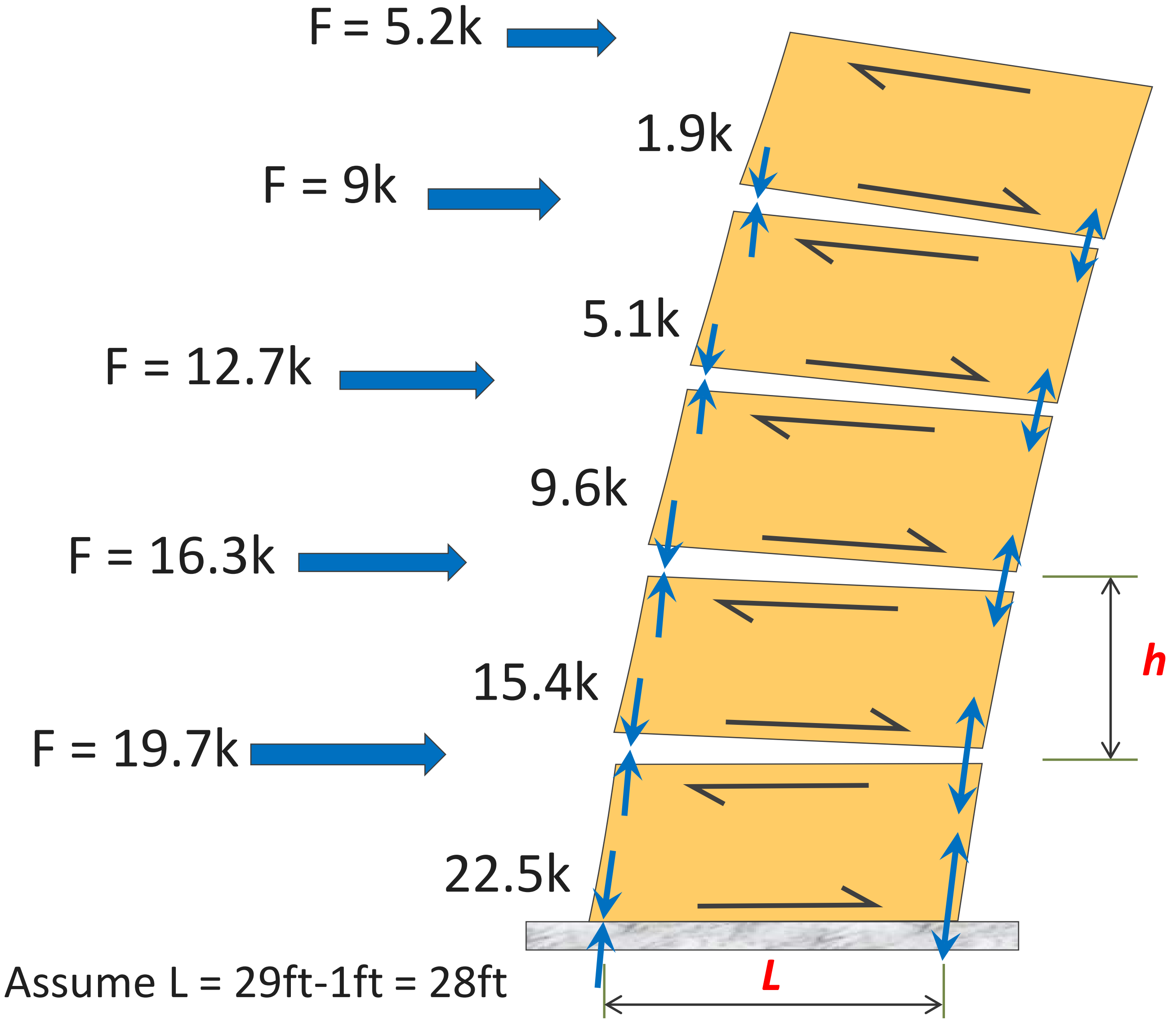


Typ. Shear Wall Elevation
Accumulated Wind Forces

Components of Shear Wall Design



Overturning Force Calculation

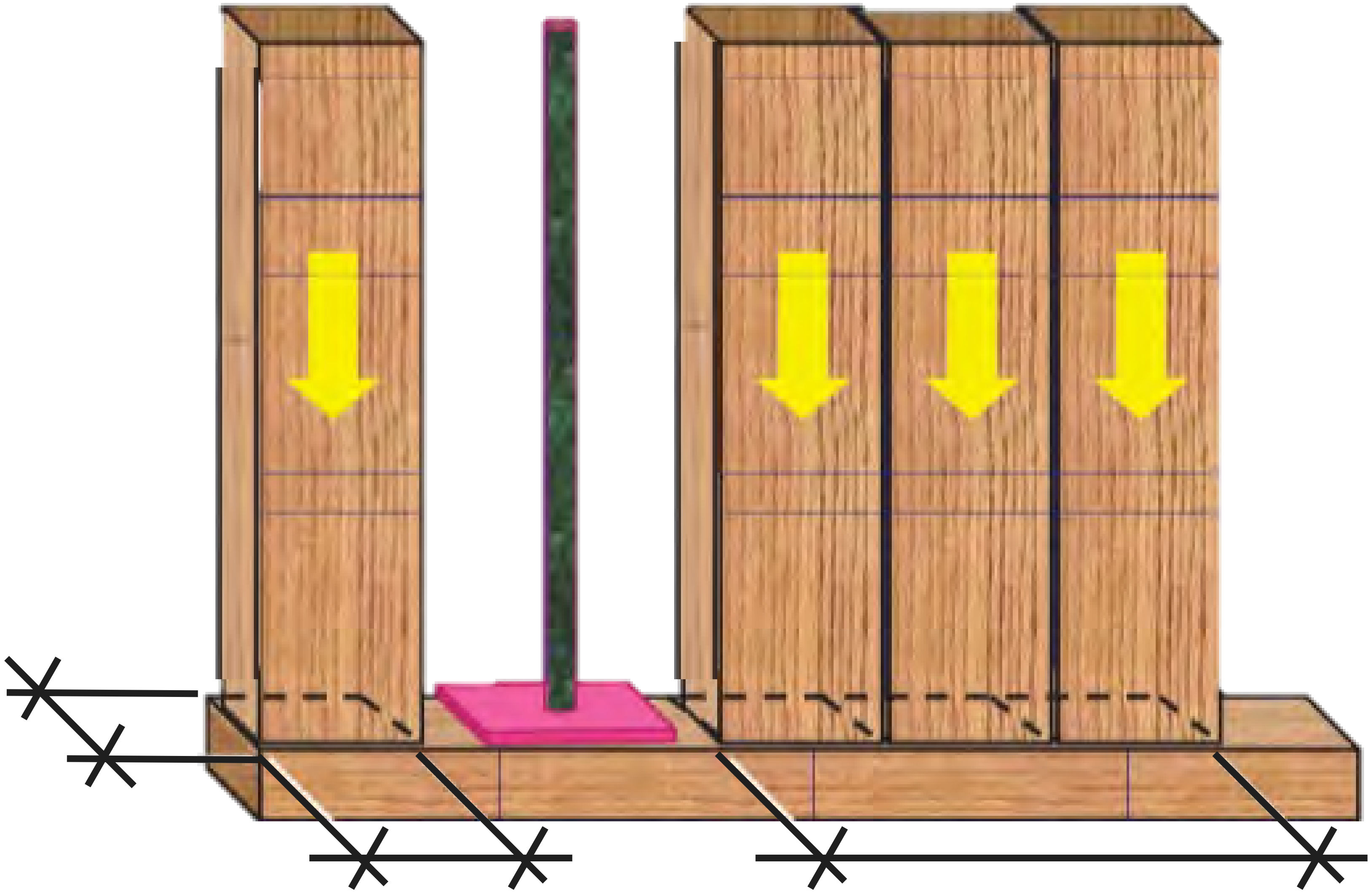


T = C = F*h/L

T & C are cumulative at lower stories

L is moment arm, not entire wall length

Sole Plate Crushing



Sole Plate Crushing

Compression forces perpendicular to grain can cause localized wood crushing. NDS values for $F_{c\perp}$ with metal plate bearing on wood result in a maximum wood crushing of 0.04”. Relationship is non-linear

Eq. 1.0

$f_{c\perp} \le F_{c\perp 0.02\ in}$

$$\Delta = 0.02 \times \left(\frac{f_{c\perp}}{F_{c\perp 0.02\ in}} \right)$$

Eq. 2.0

$F_{c\perp 0.02\ in} < f_{c\perp} < F_{c\perp 0.04\ in}$

$$\Delta = 0.04 - 0.02 \times \frac{1 - \left(\frac{f_{c\perp}}{F_{c\perp 0.04\ in}} \right)}{0.27\ in}$$

Eq. 3.0

$f_{c\perp} > F_{c\perp 0.04\ in}$

$$\Delta = 0.04 \times \left(\frac{f_{c\perp}}{F_{c\perp 0.04\ in}} \right)^3$$

Δ = deformation, in

$f_{c\perp}$ = induced stress, psi

$F_{c\perp 0.04\ in} = F_{c\perp}$ = reference design value at 0.04 in deformation, psi ($F_{c\perp}$)

$F_{c\perp 0.02\ in}$ = reference design value at 0.02 in deformation, psi ($0.73\ F_{c\perp}$)

Compression Post Size & Sole Plate Crush

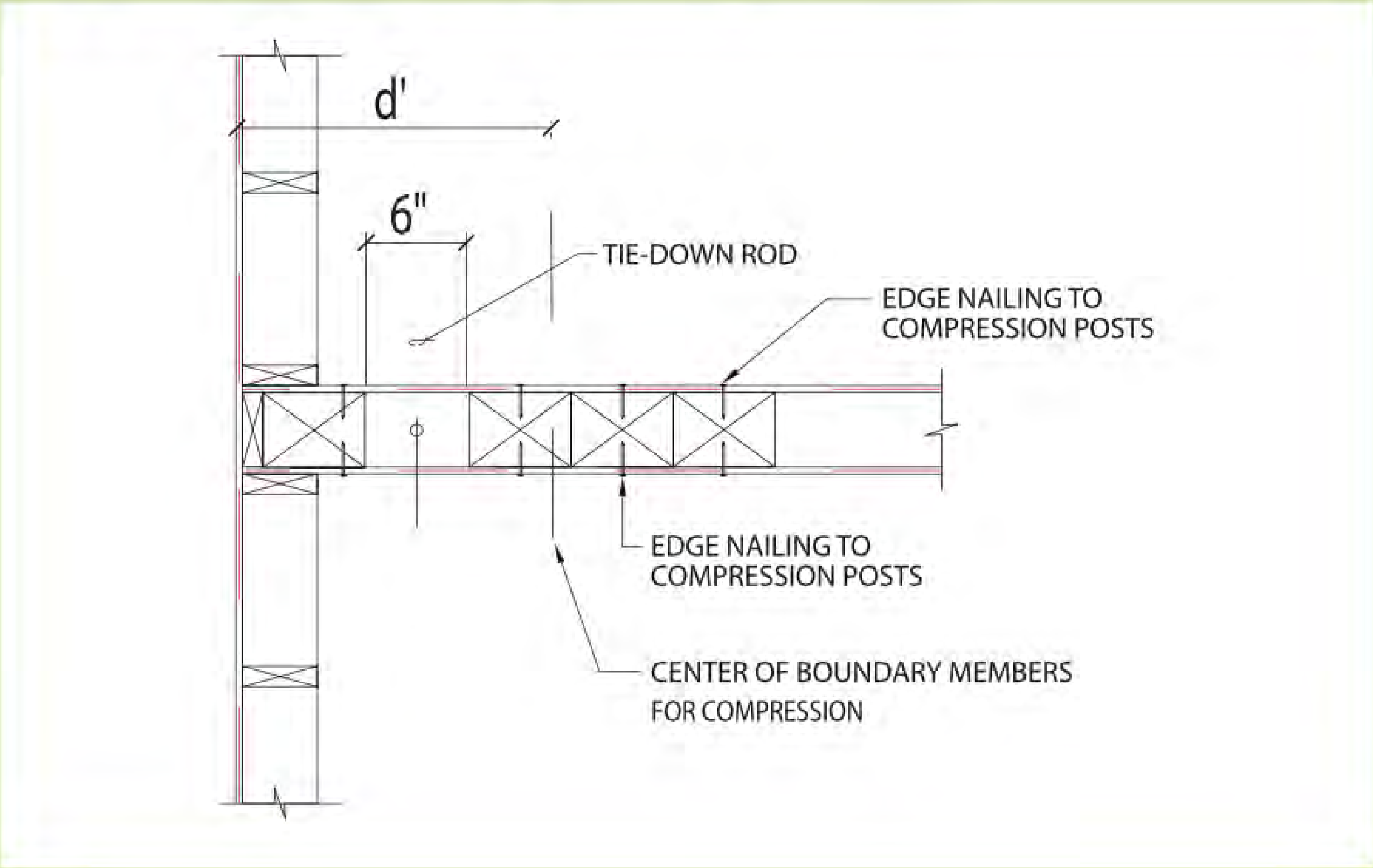
Level	Compression	Required Bearing Area	Post Size	Story Sole Plate Crush	5x Sole Plate Crush
5th Floor	1.9 k	4.4 in^2	(2)-2x4	0.011”	0.057”
4th Floor	5.1 k	11.9 in^2	(2)-4x4	0.013”	0.067”
3rd Floor	9.6 k	22.6 in^2	(2)-4x4	0.034”	0.171”
2nd Floor	15.4 k	36.3 in^2	(3)-4x4	0.039”	0.195”
1st Floor	22.5 k	39.8 in^2	(4)-4x4	0.026”	0.13”

Floors 2-5 use S-P-F #2 Sole Plate, F_{cperp} = 425 psi

Floor 1 use SYP #2 Sole Plate, F_{cperp} = 565 psi

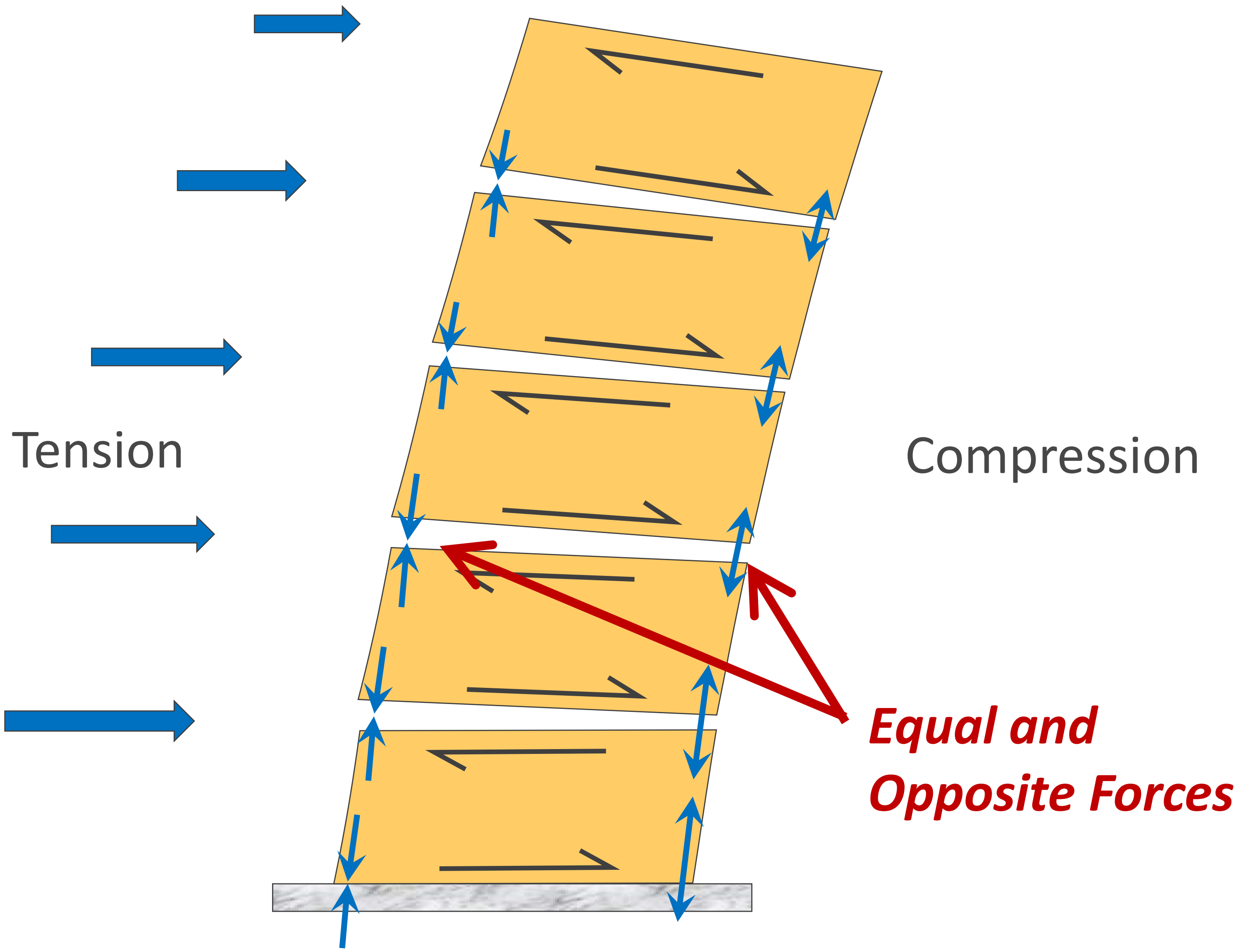
Increasing Compression Post Size

Figure 10. Example Plan Section at Boundary Members

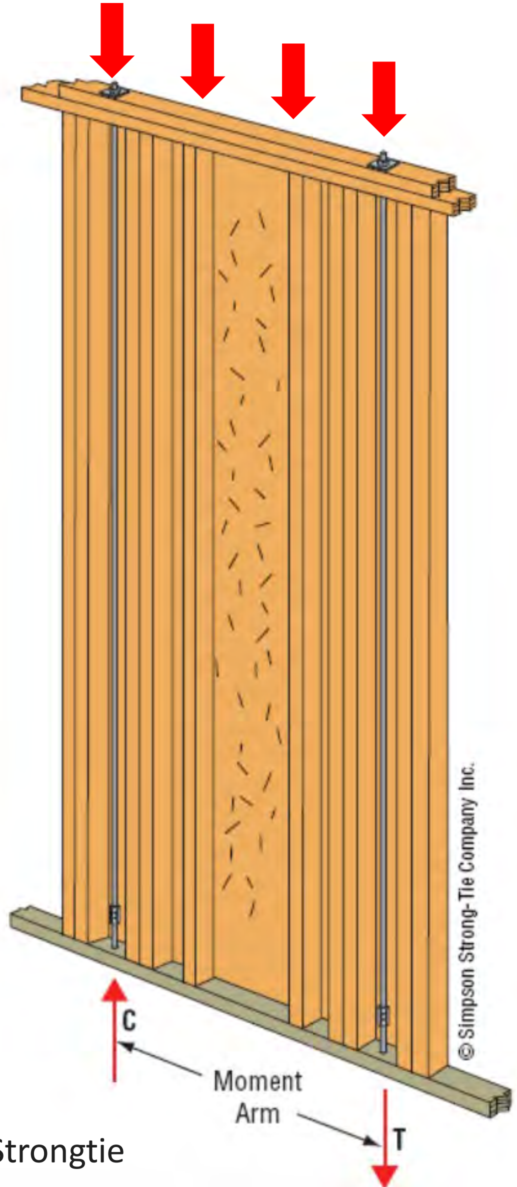


Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

Overturning Tension



Using Dead Load to Resist Overturning



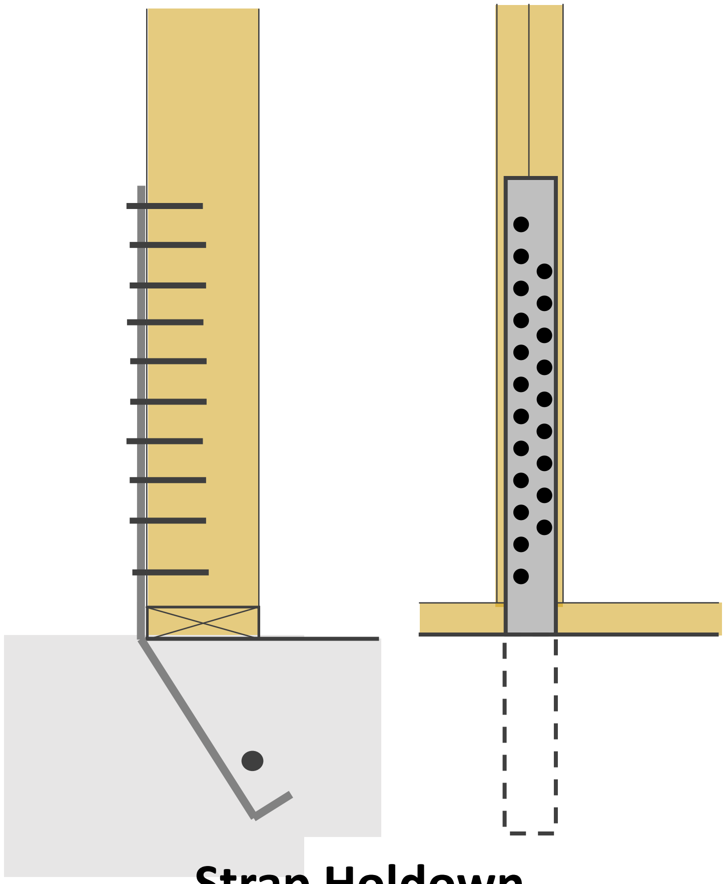
Source: Strongtie

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

Load Combinations of ASCE 7-10:

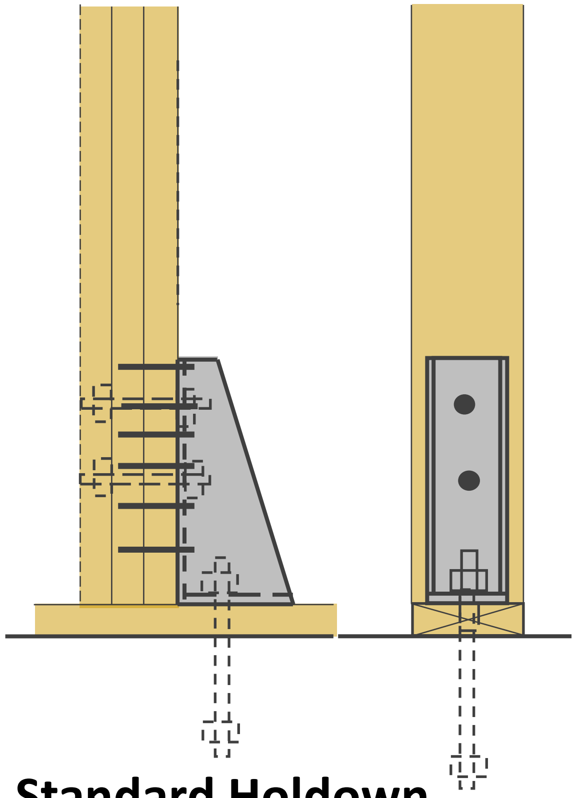
<u>0.6D</u> + 0.6W

Shear Wall Holdown Options



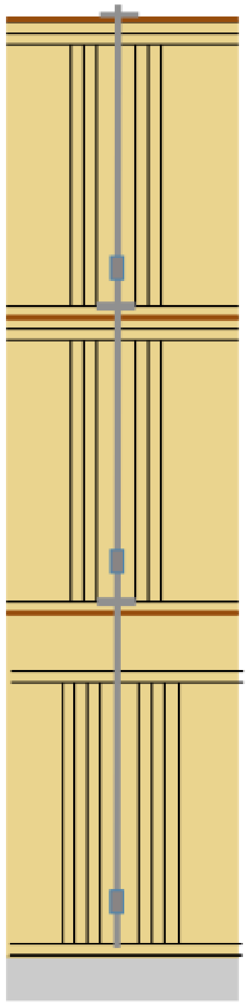
Strap Holdown Installation

6+ kip story to story capacities



Standard Holdown Installation

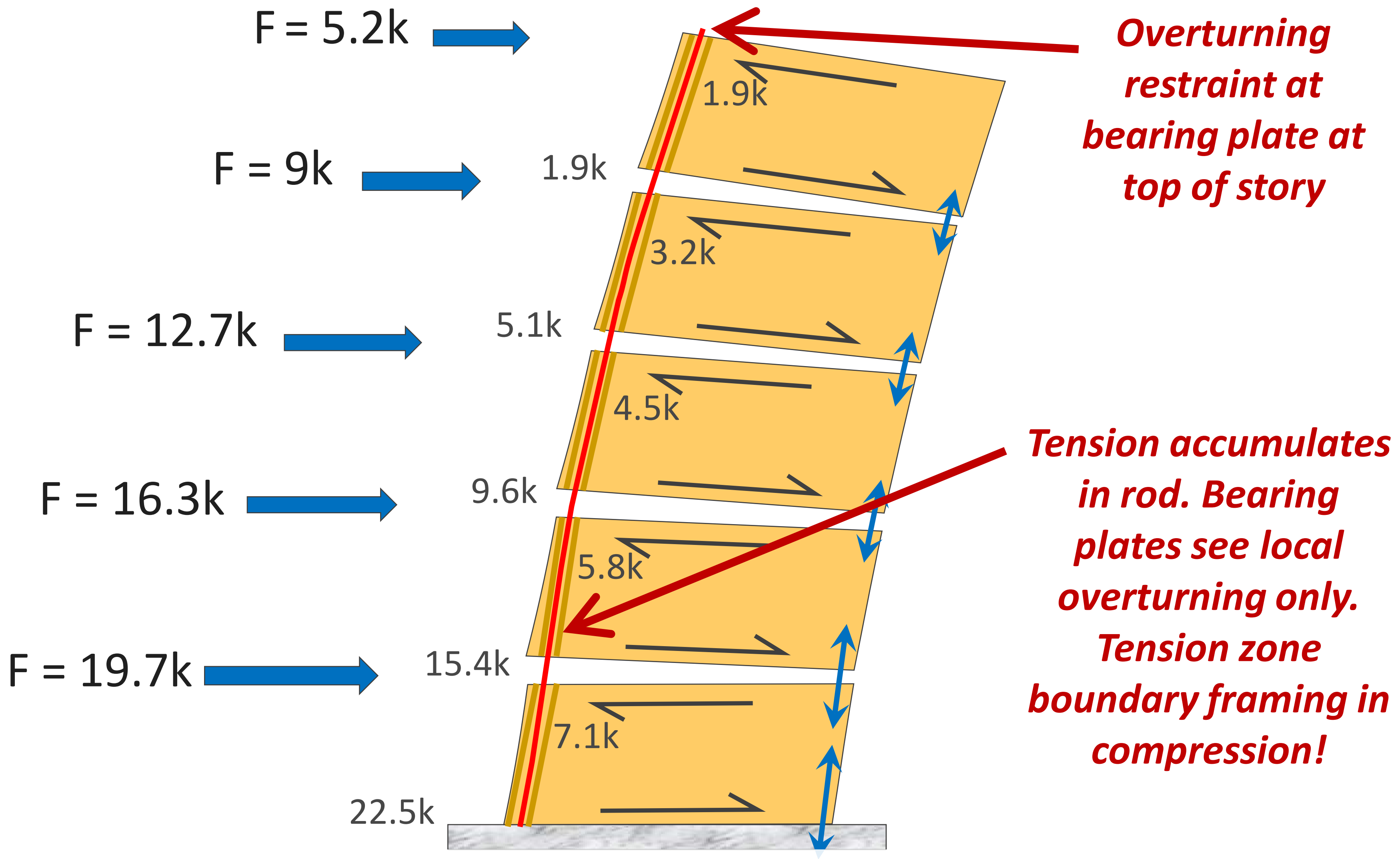
13+ kip capacities



Continuous Rod Tiedown Systems

100+ kip capacities
20+ kips/level

Components of Shear Wall Design



Continuous Rod Holdown System

Threaded Rod Tie Down w/Take Up Device

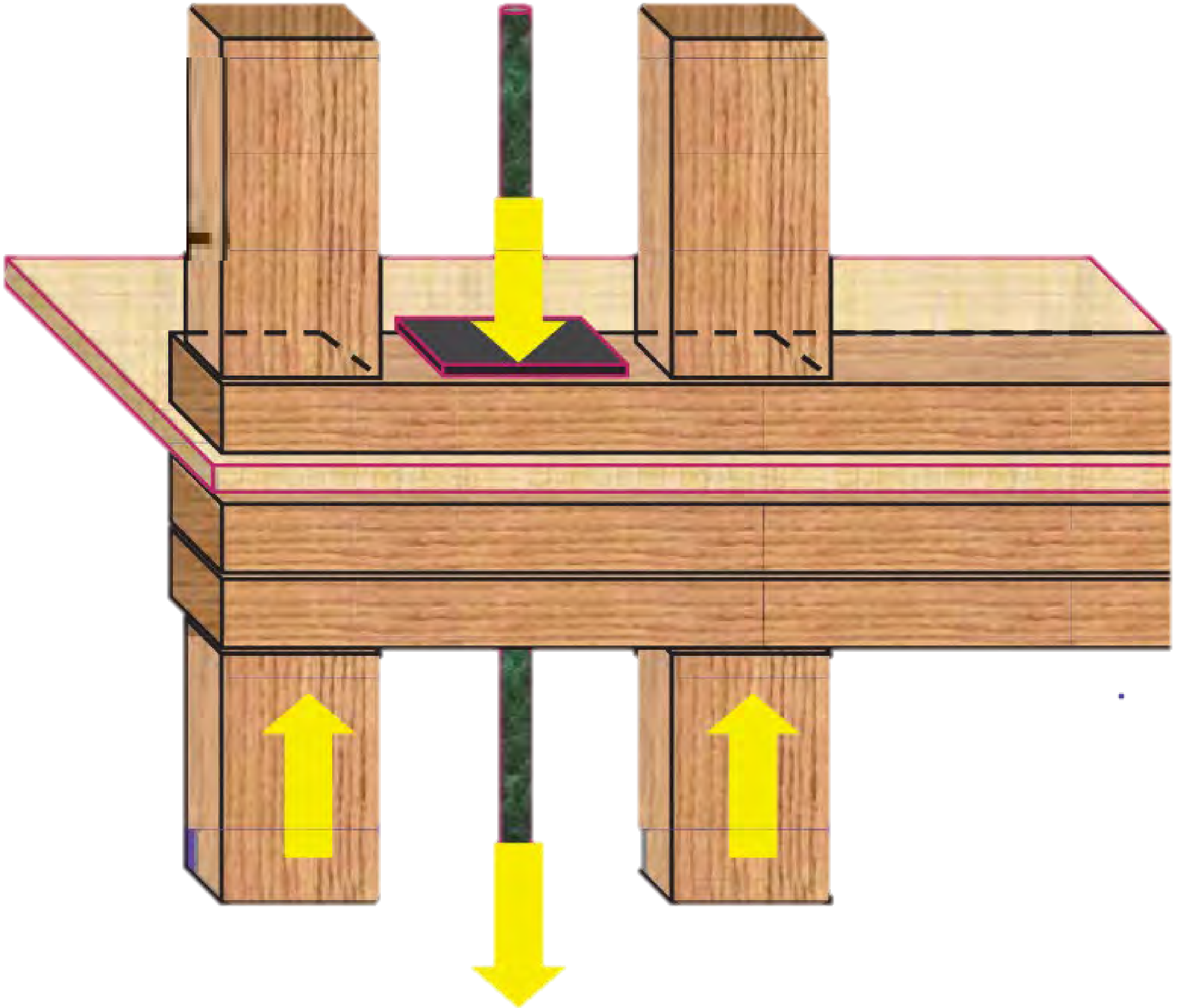
Source: Strongtie

Source: hardyframe.com

Tie Down Rod Size & Elongation

Level	Plate Hght	Tension	Rod Dia.	Steel	Rod Capacity	Rod Elong.
5th Floor	10 ft	1.9 k	3/8”	A36	2.4 k	0.10”
4th Floor	10 ft	5.1 k	5/8”	A36	6.7 k	0.09”
3rd Floor	10 ft	9.6 k	5/8”	A193	14.4 k	0.18”
2nd Floor	10 ft	15.4 k	3/4”	A193	20.7 k	0.19”
1st Floor	10 ft	22.5 k	7/8”	A193	28.2 k	0.2”

Bearing Plate Crushing



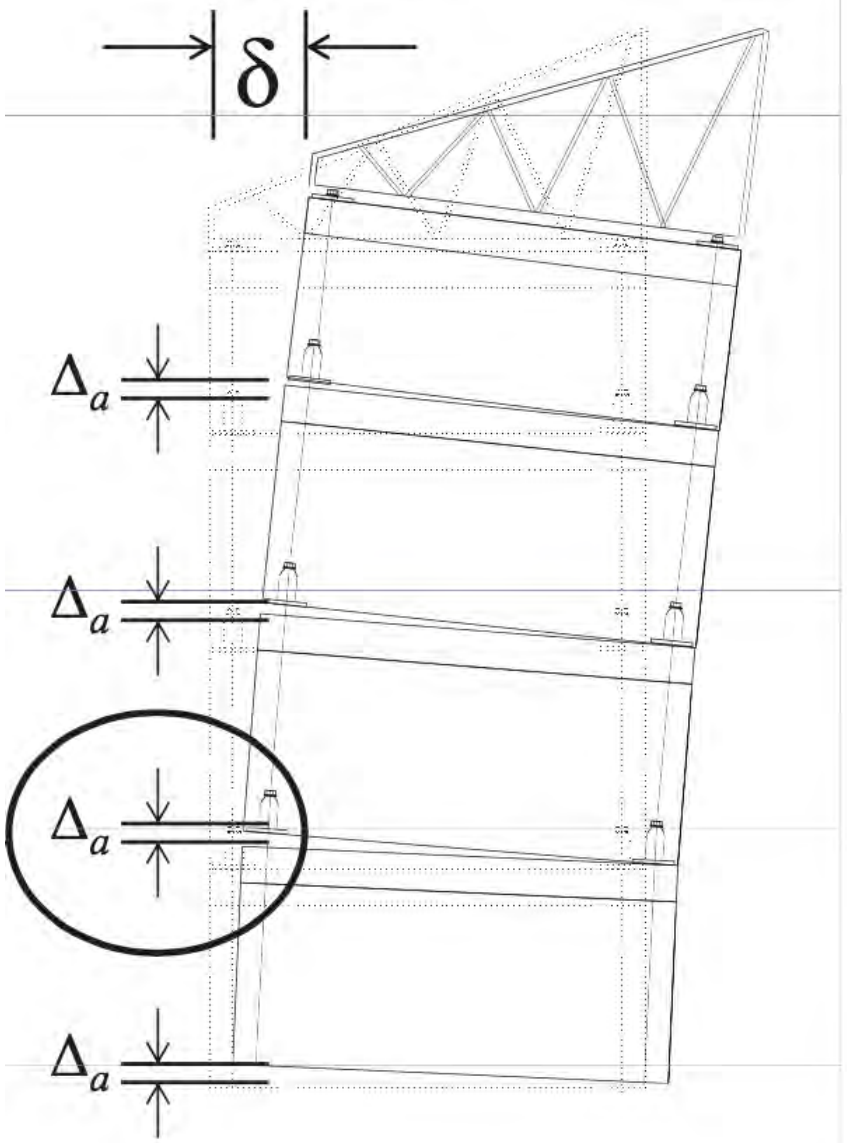
Bearing Plate Size & Thickness

Level	Bearing Plate					Bearing Load	Allow. Bearing Capacity	Bearing Plate Crush
	W	L	T	Hole Area	A_{brng}			
5th Floor	3 in	3.5 in	3/8”	0.25 in^2	10.25 in^2	1.9 k	4.4 k	0.012”
4th Floor	3 in	3.5 in	3/8”	0.518 in^2	9.98 in^2	3.2 k	4.2 k	0.022”
3rd Floor	3 in	5.5 in	1/2”	0.518 in^2	15.98 in^2	4.5 k	6.8 k	0.018”
2nd Floor	3 in	5.5 in	1/2”	0.69 in^2	15.8 in^2	5.8 k	6.7 k	0.03”
1st Floor	3 in	8.5 in	7/8”	0.89 in^2	24.6 in^2	7.0 k	10.4 k	0.014”

Shearwall Deformation – System Stretch

Total system stretch includes:

- Rod Elongation
- Take-up device displacement
- Bearing Plate Crushing
- Sole Plate Crushing



Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

Accumulative Movement

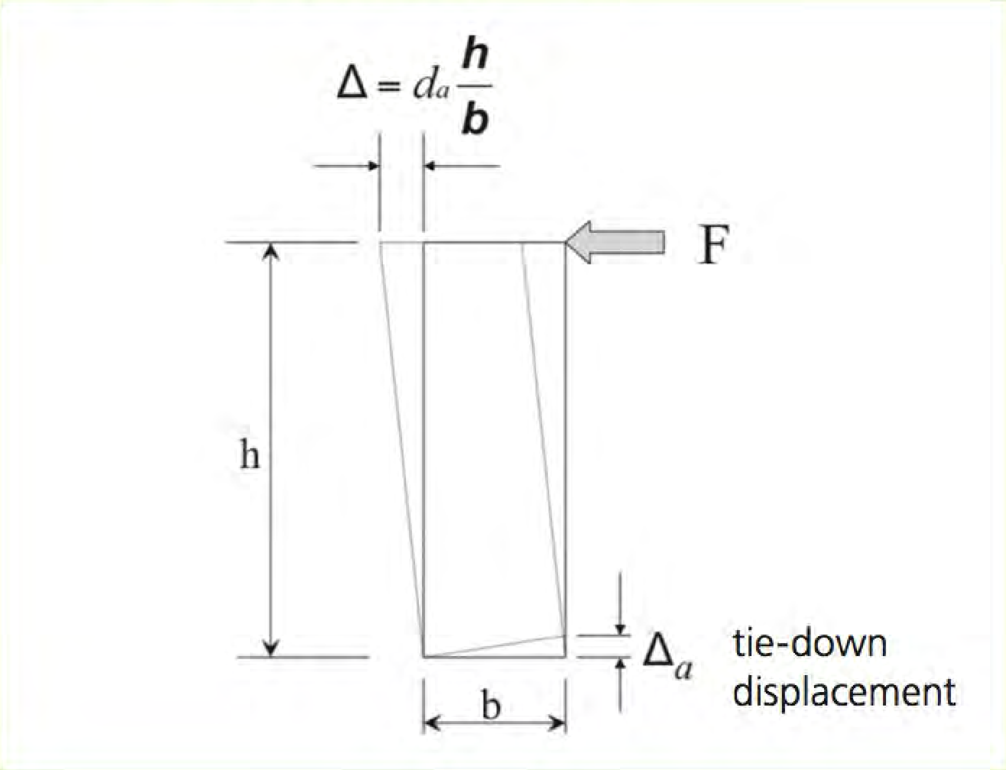
With Shrinkage Compensating Devices

Level	Rod Elong.	Shrinkage	Sole Plate Crush	Bearing Plate Crush	Take Up Deflect. Elong.	Total Displac.
5th Floor	0.1”	0.03”	0.057”	0.012”	0.03”	0.23”
4th Floor	0.09”	0.03”	0.067”	0.022”	0.03”	0.24”
3rd Floor	0.18”	0.03”	0.171”	0.018”	0.03”	0.43”
2nd Floor	0.19”	0.03”	0.195”	0.03”	0.03”	0.48”
1st Floor	0.2”	0.03”	0.13”	0.014”	0.03”	0.4”

Shearwall Tie Down Elongation

SDPWS Definition of Δ_a: *"Total vertical elongation of wall anchorage system (including fastener slip, device elongation, rod elongation, etc.) at the induced unit shear in the wall.*

Figure 11. *Effect of Δ_a on Drift*



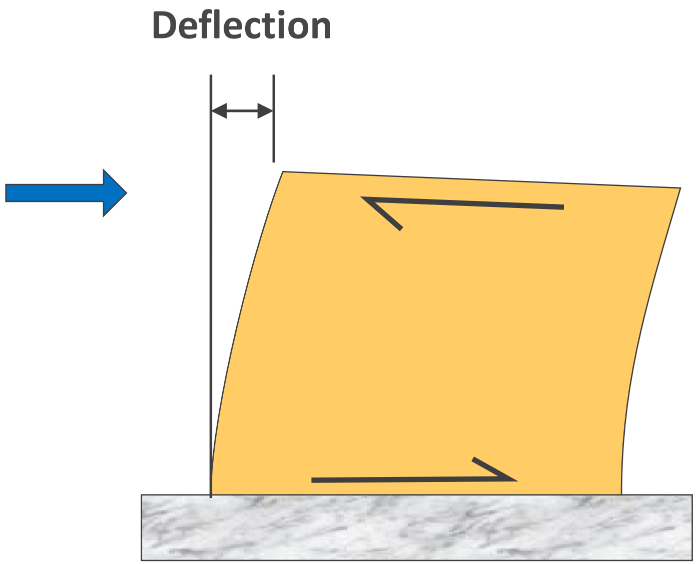
Notes for Figure 11:

Where: h = floor-to-floor height

b = the out-to-out dimension of the shear wall

Source: WoodWorks Five-Story Wood-Frame Structure over Podium Slab Design Example

Shear Wall Deflection



SDPWS 2008 Eq 4.3-1

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

SDPWS 2008 Eq. C4.3.2-1

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

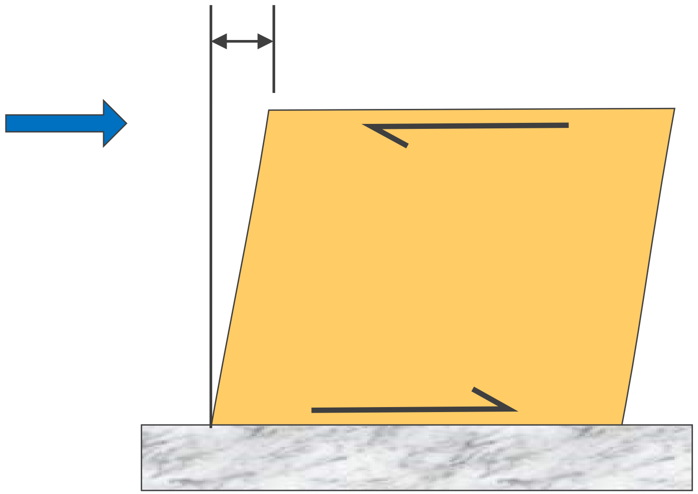
IBC 2000 to 2015 Eq. 23-2

$$\Delta = \boxed{\frac{8vh^3}{EAb}} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$

Bending of boundary elements

Shear Wall Deflection

Deflection



SDPWS 2008 Eq 4.3-1

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

SDPWS 2008 Eq. C4.3.2-1

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

IBC 2000 to 2015 Eq. 23-2

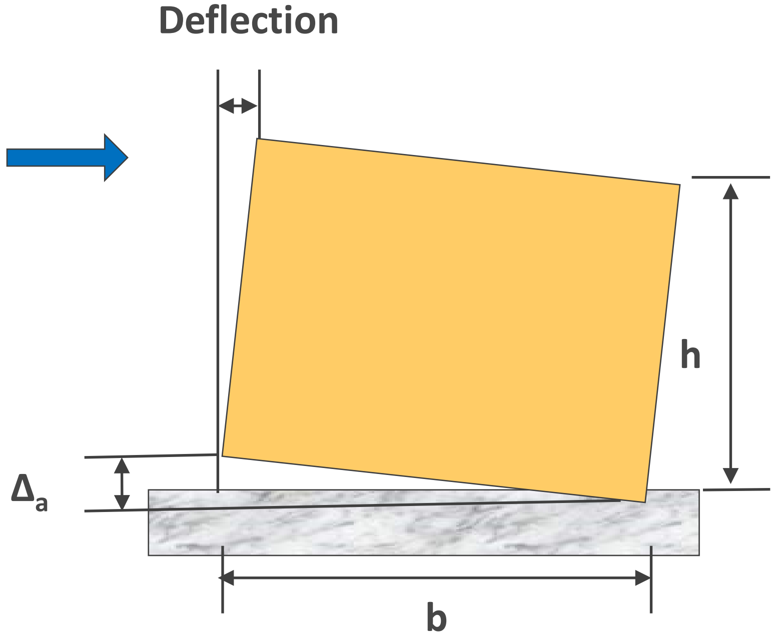
$$\Delta = \frac{8vh^3}{EAb} + \boxed{\frac{vh}{Gt} + 0.75he_n} + d_a\frac{h}{b}$$

Shear Deformation of Sheathing Panels

&

Slip of nails @ panel to panel connections

Shear Wall Deflection



SDPWS 2008 Eq 4.3-1

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

SDPWS 2008 Eq. C4.3.2-1

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

IBC 2000 to 2015 Eq. 23-2

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \boxed{d_a\frac{h}{b}}$$

Rigid Body Rotation

Shearwall Deflection

Level	Unit Shear	End Post A	End Post E	Ga	Total Displace.	Deflection
5th Floor	179 plf	10.5 in^2	1400 ksi	10 k/in	0.23”	0.26”
4th Floor	310 plf	24.5 in^2	1400 ksi	10 k/in	0.24”	0.4”
3rd Floor	438 plf	24.5 in^2	1400 ksi	10 k/in	0.43”	0.59”
2nd Floor	562 plf	36.8 in^2	1400 ksi	13 k/in	0.48”	0.6”
1st Floor	679 plf	49 in^2	1400 ksi	13 k/in	0.4”	0.67”

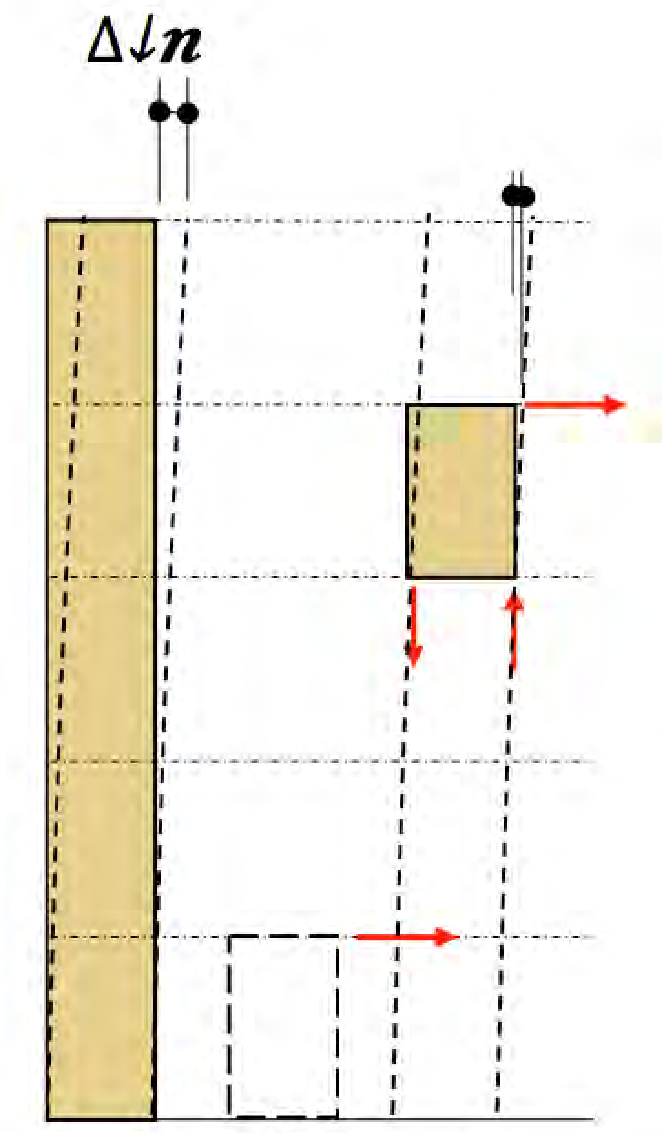
Shearwall Deflection Methods

Multiple methods for calculating accumulative shearwall deflection exist

Mechanics Based Approach:

- Uses single story deflection equation at each floor
- Includes rotational & crushing effects
- Uses SDPWS 3 part equation

Other methods exist which use alternate deflection equations, FEM



Shearwall Deflection Criteria for Wind

Unlike seismic, no code information exists on deflection/drift criteria of structures due to wind loads

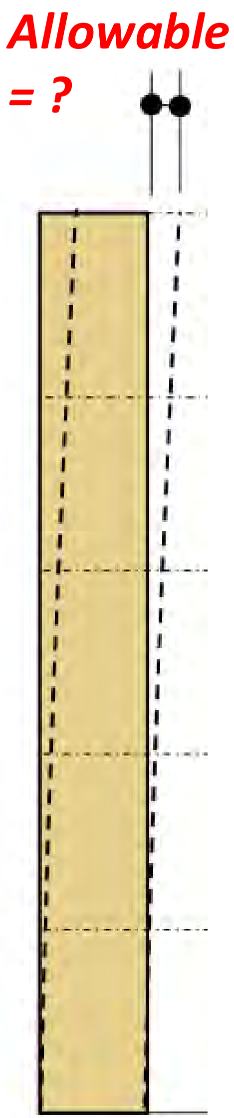
Serviceability check to minimize damage to cladding and nonstructural walls

ASCE 7-10:

C.2.2 Drift of Walls and Frames. Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems due to wind effects shall not impair the serviceability of the structure.

What wind force should be used?

What drift criteria should be applied?



Shearwall Deflection Criteria for Wind

Wind Forces

Consensus is that ASD design level forces are too conservative for building/frame drift check due to wind

- Commentary to ASCE 7-10 Appendix C suggests that some recommend using 10 year return period wind forces:
    - ~ 70% of 700 return period wind (ultimate wind speed for risk category II buildings)
- Others (AISC Design Guide 3) recommend using 75% of 50 year return period forces

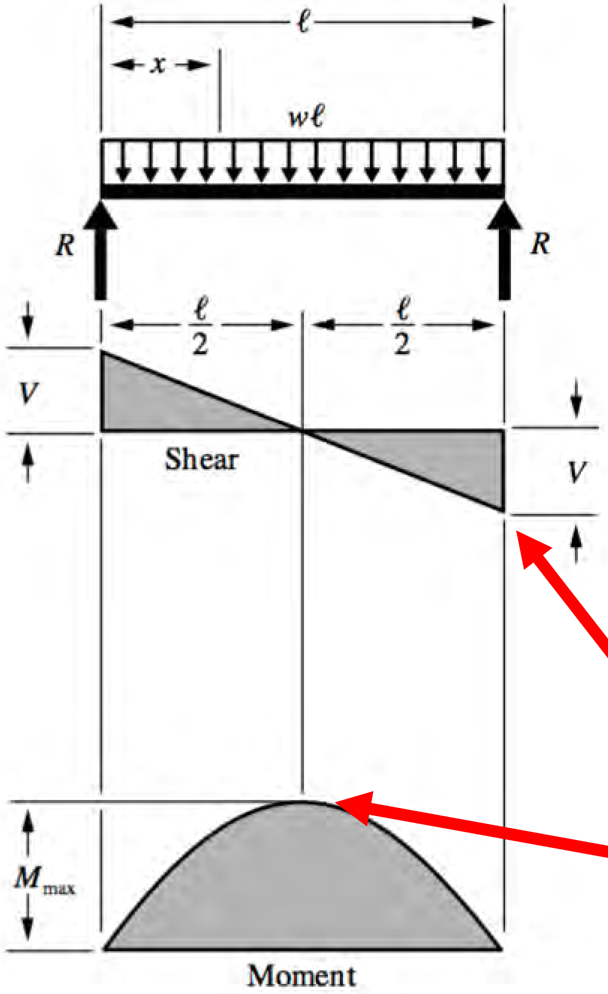
Drift Criteria

Can vary widely with brittleness of finishes but generally recommendations are in the range of H/240 to H/600

Diaphragm Design

Calculating Diaphragm Forces

Figure 1 Simple Beam – Uniformly Distributed Load



$$R = V \quad \ldots\ldots \quad = \frac{w\ell}{2}$$

$$V_x \quad \ldots\ldots \quad = w\left(\frac{\ell}{2} - x\right)$$

$$M_{max} \text{ (at center)} \quad \ldots\ldots \quad = \frac{w\ell^2}{8}$$

$$M_x \quad \ldots\ldots \quad = \frac{wx}{2}(\ell - x)$$

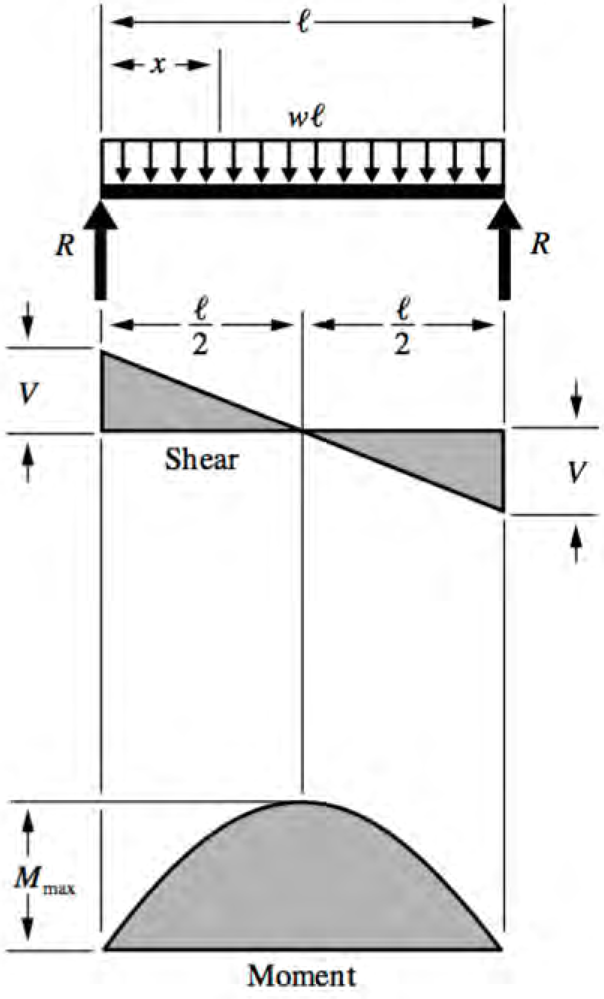
Max Shear at Ends

Max Moment at Mid-Span

AWC Design Aid 6

Calculating Diaphragm Forces

Figure 1 Simple Beam – Uniformly Distributed Load

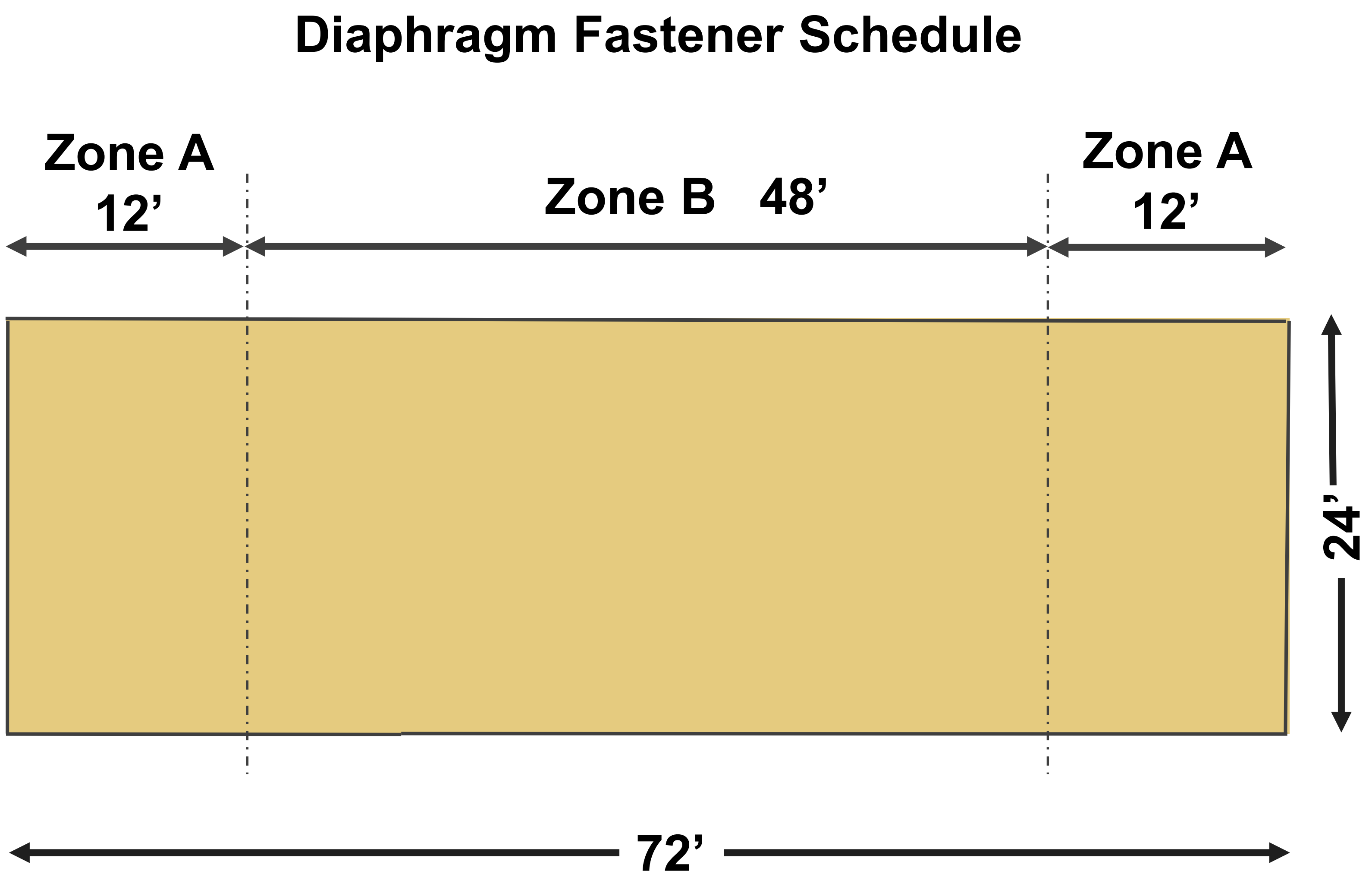


Diaphragm Shear:

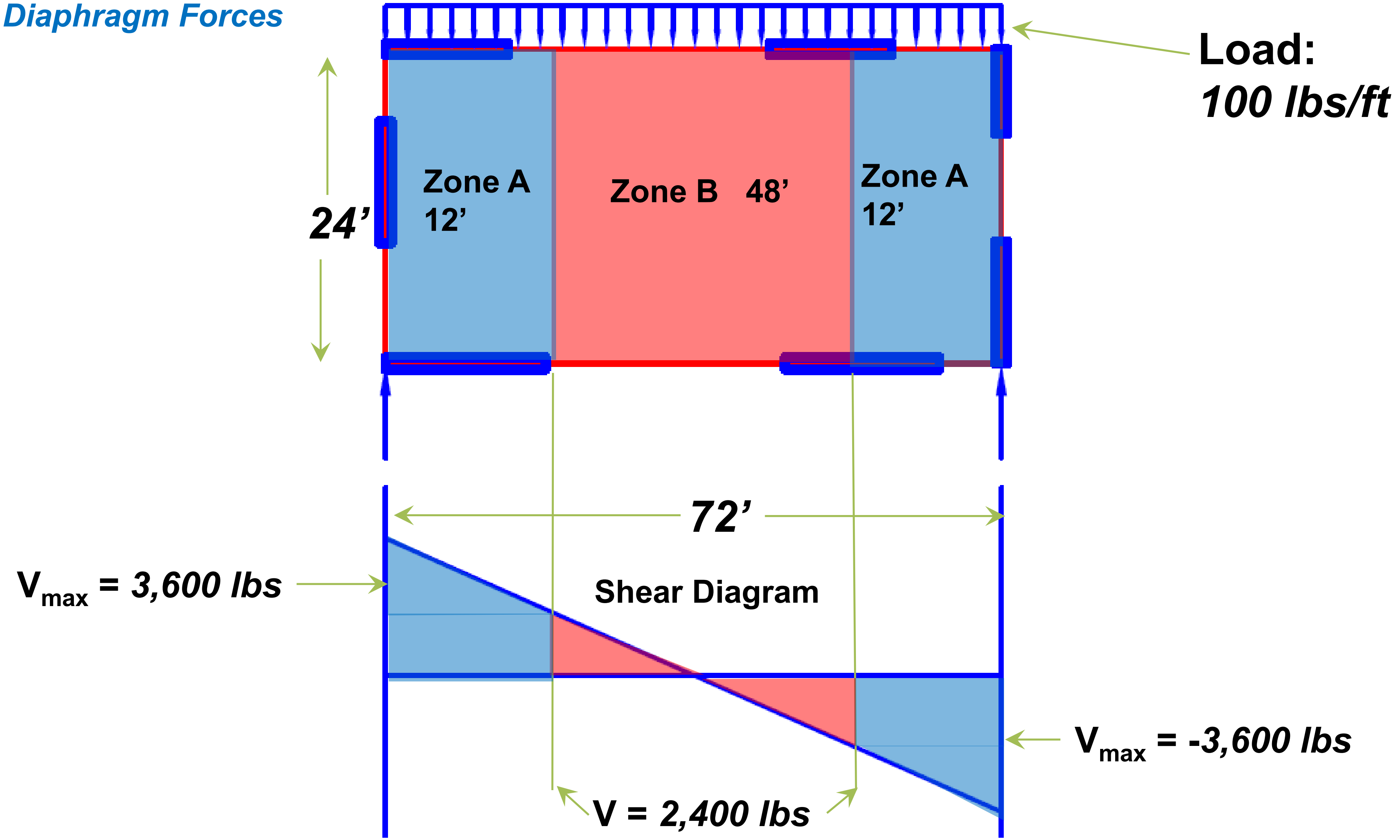
- **Max Shear = Diaphragm Reaction at Shearwall**
- **Diaphragm Unit Shear = Reaction / Length of Diaphragm = plf**

AWC Design Aid 6

Calculating Diaphragm Forces



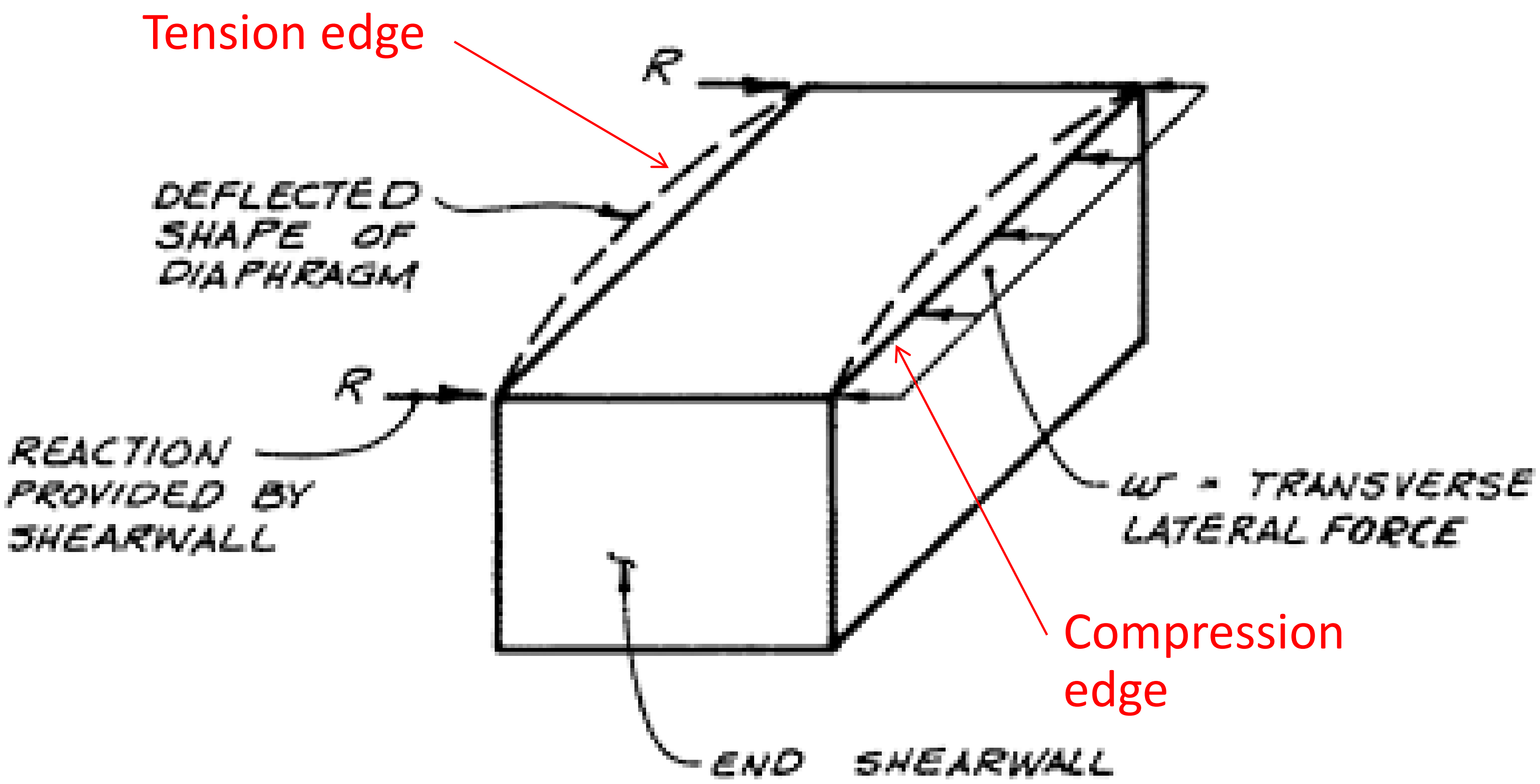
Diaphragm Forces



Diaphragm Fastener Schedule

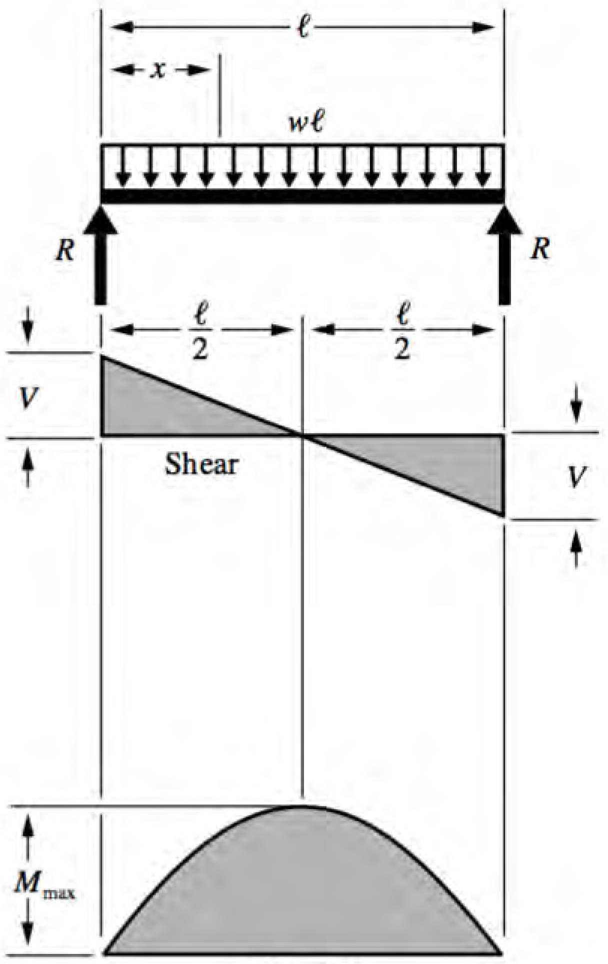
- *Zone A: Nailing Pattern 1*
- *Zone B: Nailing Pattern 2*

Diaphragm – Bending Member



Diaphragm Chord Forces

Figure 1 Simple Beam – Uniformly Distributed Load



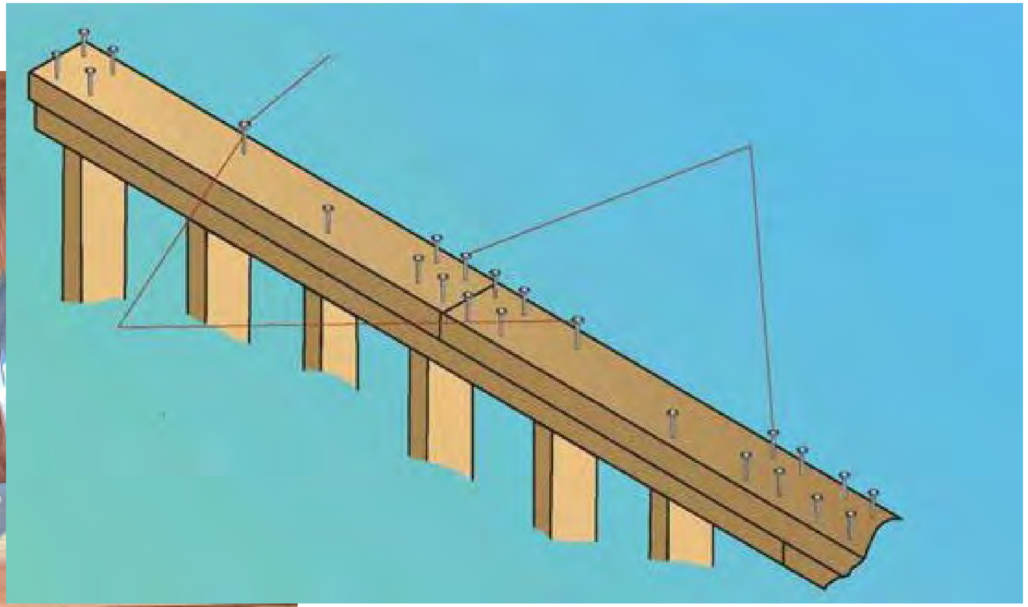
<u>**Diaphragm Chord Forces:**</u>

- **Max Chord Force Occurs at Location of Max Moment**
- **Chord Force = T or C**
- **Chord Force = M_{MAX} / Diaphragm Depth**
- **Chord Unit Shear = Chord Force / Length of Diaphragm = plf**

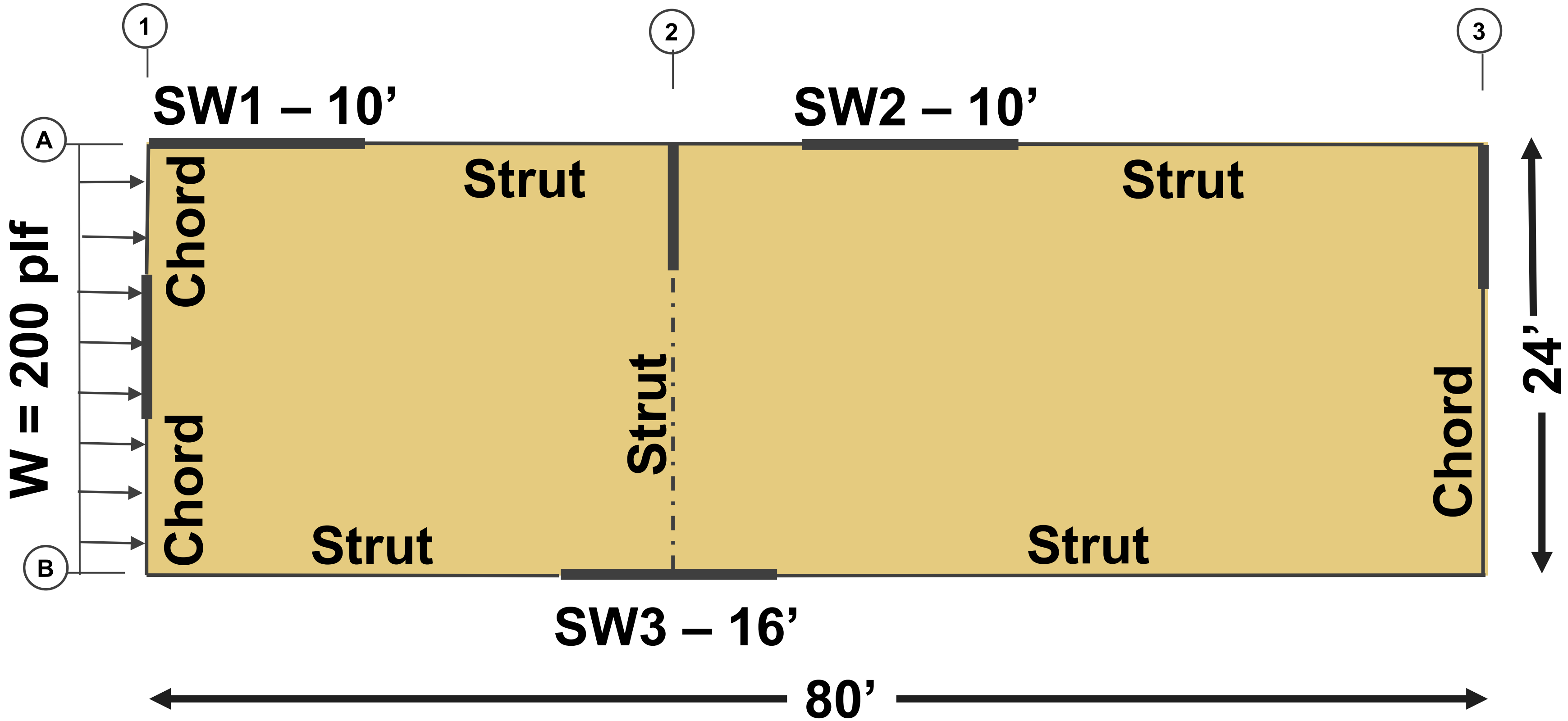
AWC Design Aid 6

Diaphragm Chords

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

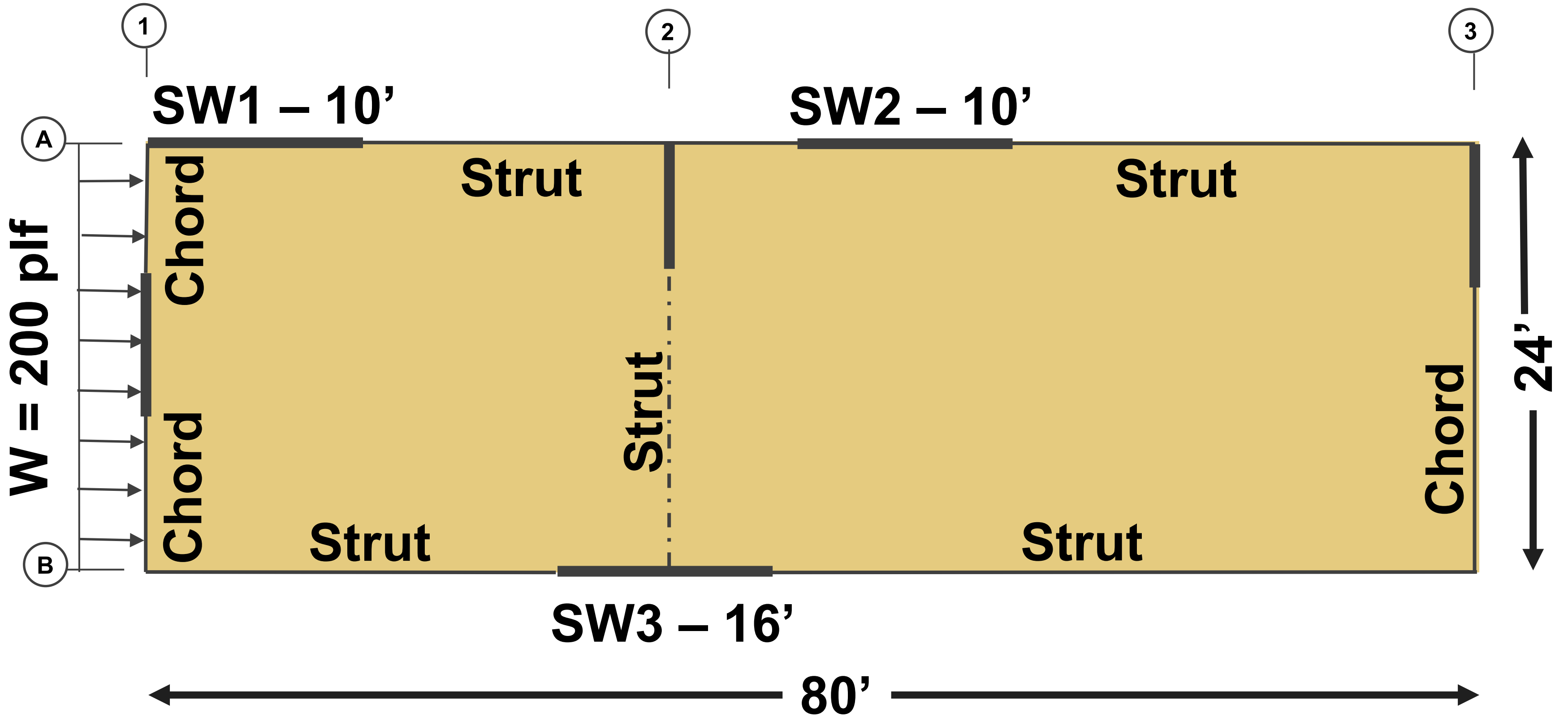


Diaphragm Boundary



Reaction = 200 plf * 24'/2 = 2400 lbs
Diaphragm Only at Shearwall = 2400 lbs / 16' = 150 plf

Diaphragm Boundary



Does this mean that no drag struts are required?

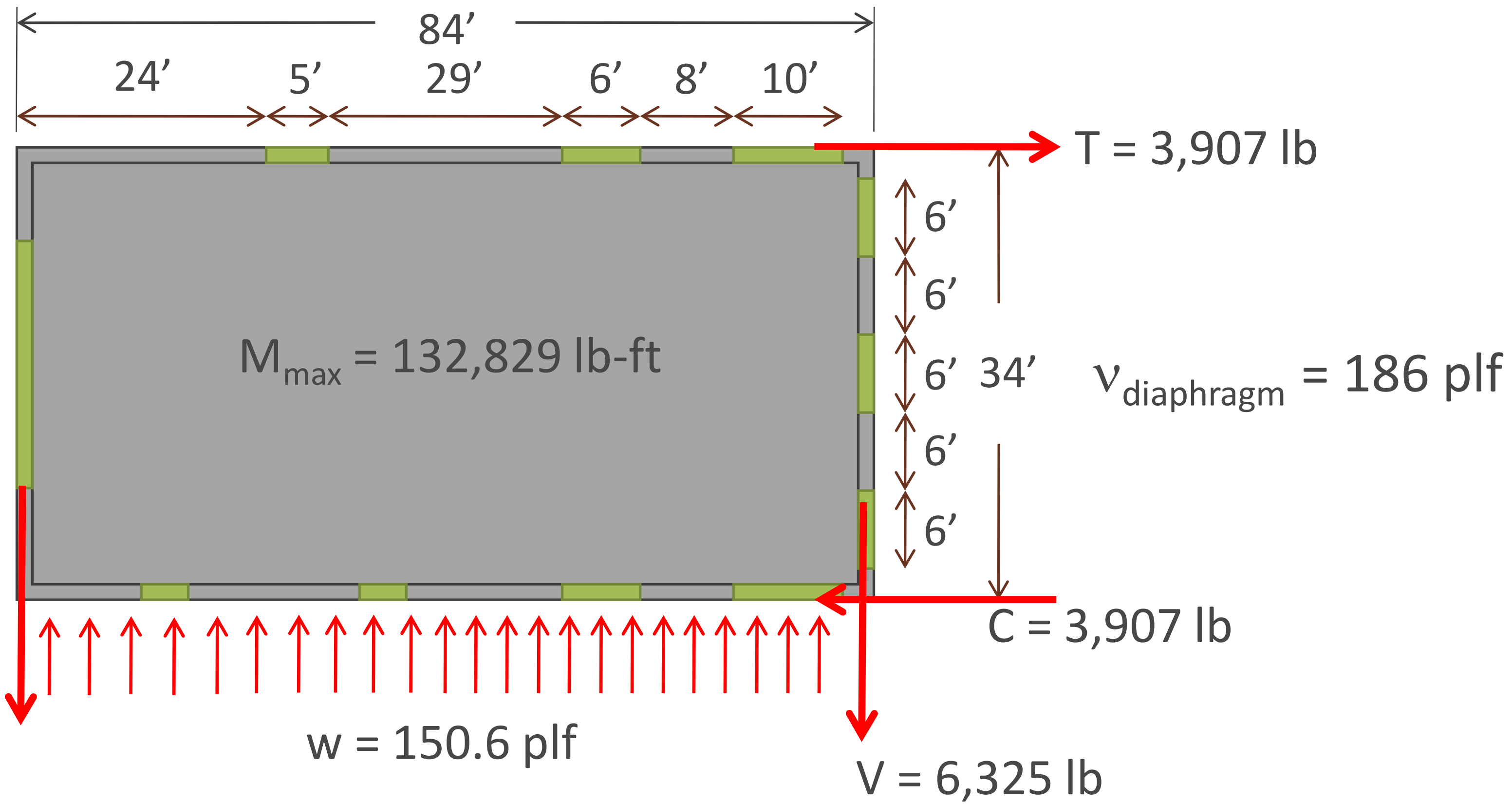
Diaphragm Boundary

All edges of a diaphragm shall be supported by a boundary element. (ASCE 7-10 Section 11.2)

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

Diaphragm Design - Deflection

Assume 7/16" OSB Sheathing with 24/16 Span Rating. Unblocked diaphragm with 8d common nails at 6" o.c. at all panel edges. Spruce Pine Fir trusses spaced 24" o.c.



See SDPWS example C4.2.2-3 & APA L350 for design examples

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 Design – Deflection

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

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

ν = max unit shear in diaphragm - plf

L = diaphragm length (perpendicular to force) - ft

E = modulus of elasticity of diaphragm chords - psi

A = area of chord (cross section) - in^2

W = Width of diaphragm in direction of applied force - ft

$G_v t_v$ = shear stiffness, lb/in of panel depth

x = distance from chord splice to nearest support - ft

Δ_c = diaphragm chord splice slip - in

e_n = nail slip - in

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

Alternate Equation

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \qquad \text{(C4.2.2-2)}$$

Diaphragm Design – Deflection

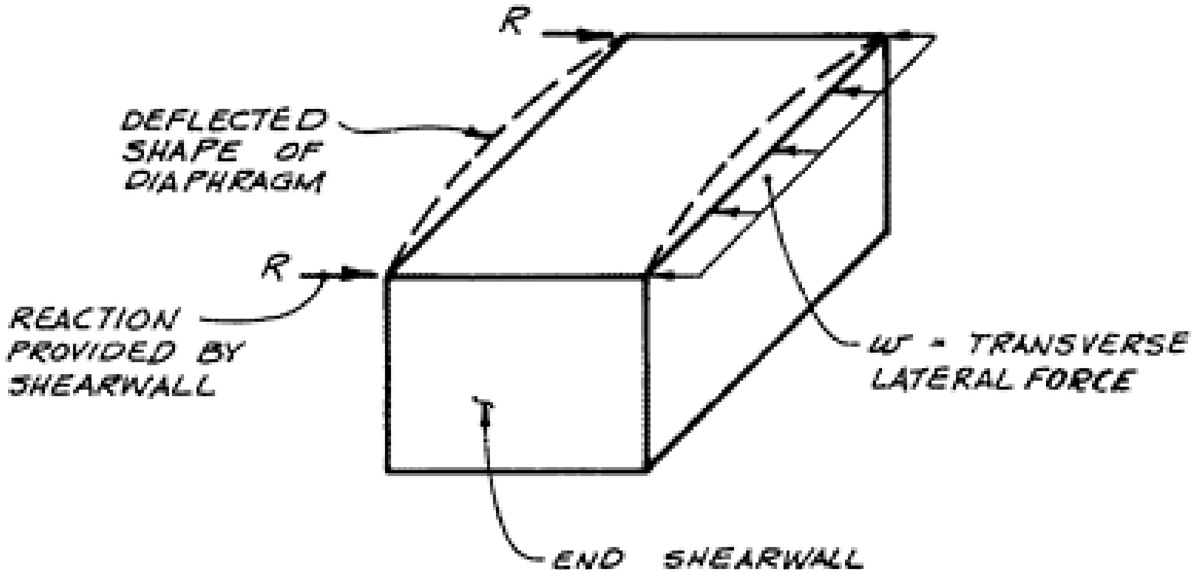
(bending, chord deformation excluding slip)	(shear, panel deformation)	(shear, panel nail slip)	(bending, chord splice slip)

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

$\Delta_{bending} = 5\nu L^3 / 8EAW$

$= 5*(186 \text{ plf})*(84')^3 / [8*(1{,}400{,}000 \text{ psi})*(2*1.5''*5.5'')*(34')]$

$= 0.088''$



Diaphragm Design – Deflection

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

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

$\Delta_{shear} = \nu L / 4G_v t_v$

SDPWS Table C4.2.2A:

$G_v t_v$ = 83,500 lb/in of depth for 7/16" OSB, 24/16 span rating

= (186 plf)*(84') / [4*83,500]

= 0.047"

Table C4.2.2A Shear Stiffness, $G_v t_v$ (lb/in. of depth), for Wood Structural Panels

Span Rating[4]	Minimum Nominal Panel Thickness (in.)	Structural Sheathing				Structural I			
		Plywood			OSB	Plywood			OSB
		3-ply	4-ply	5-ply[3]		3-ply	4-ply	5-ply[3]	
Sheathing Grades[1]									
24/0	3/8[2]	25,000	32,500	37,500	77,500	32,500	42,500	41,500	77,500
24/16	7/16	27,000	35,000	40,500	$\boxed{83,500}$	35,000	45,500	44,500	83,500
32/16	15/32	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
40/20	19/32	28,500	37,000	43,000	88,500	37,000	48,000	47,500	88,500
48/24	23/32	31,000	40,500	46,500	96,000	40,500	52,500	51,000	96,000

Diaphragm Design – Deflection

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

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

$\Delta_{panel\ nail\ slip} = 0.188Le_n$

V_n = load per nail = 186 plf / (12/6") = 93 lbs per nail (*panel nails spaced 6" o.c.*)

$e_n = (V_n / 616)^{3.018}$ ***per footnote in SDPWS Table C4.2.2D, slip needs to be increased by 20% when OSB is not Structural I grade*

$e_n = 1.2*(V_n / 616)^{3.018} = 1.2*(93 / 616)^{3.018} = 0.004"$

$\Delta_{panel\ nail\ slip} = 0.188*84'*0.004" = 0.063"$

Table C4.2.2D Fastener Slip, e_n (in.)

Sheathing	Fastener Size	Maximum Fastener Load (V_n) (lb/fastener)	Fastener Slip, e_n (in.) Fabricated w/green (>19% m.c.) lumber	Fabricated w/dry (≤ 19% m.c.) lumber
Wood Structural Panel (WSP) or Particleboard[1]	6d common	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
	8d common	220	$(V_n/857)^{1.869}$	$\boxed{(V_n/616)^{3.018}}$
	10d common	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$

Diaphragm Design – Deflection

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

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

$\Delta_{chord\ splice} = \Sigma(x\Delta_c) / 2W$

$\Delta_c = 2(T \text{ or } C) / \gamma n$ *(the 2 in the numerator is to account for splice slip on each side of the joint)*

γ = load/slip modulus for connection = 180,000($D^{1.5}$) for dowel-type fasteners (wood-to-wood) *NDS 11.3.6* D = diameter of dowel-type fastener (*16d common*)

$\gamma = 180{,}000(0.162^{1.5}) = 11{,}737$ lb/in/nail

Δ_{c3} = 2(3,827 lb) / (11,737 * 21) = 0.031” *T_3 = 130,829 lb-ft / 34 = 3,827 lb*

Δ_{c2} = 2(3,189 lb) / (11,737 * 21) = 0.026” *T_2 = 108,432 lb-ft / 34 = 3,189 lb*

Δ_{c1} = 2(1,914 lb) / (11,737 * 21) = 0.016” *T_1 = 65,058 lb-ft / 34 = 1,914 lb*

Diaphragm Design – Deflection

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

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

$x^*\Delta_{\text{tension chord}}$ = 12’*0.016” + 24’*0.026” + 36’*0.031” + 36’*0.031” + 24’*0.026” + 12’*0.016”

$x^*\Delta_{\text{tension chord}}$ = 3.86 in-ft

Diaphragm Design – Deflection

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

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

From SDPWS: Assuming butt joints in the compression chord are not tight and have a gap that exceeds the splice slip, the tension chord slip calculation is also applicable to the compression chord.

$x*\Delta_{\text{tension chord}} = x*\Delta_{\text{tension chord}} = 3.86$ in-ft

$$\Delta_{\text{chord splice}} = \frac{3.86 + 3.86}{2\,(34')}$$

$\Delta_{\text{chord splice}} = 0.114''$

$\delta_{dia} = (0.088 + 0.047 + 0.063 + 0.114)\ 2.5$ *(2.5 to account for unblocked) diaphragm)*

$\delta_{dia} = 0.78''$

Flexibility and Redundancy Design Challenges

16 Powerhouse, Sacramento, CA
D&S Development
LPA Sacramento

Codes and Standards

A variety of challenges often occur on projects due to:

- Fewer opportunities for shear walls at exterior wall lines which cause Open-front diaphragm conditions
- Increased building heights, and
- Potential multi-story shear wall effects.

In mid-rise, multi-family buildings, corridor only shear walls are becoming very popular way to address the lack of capable exterior shear walls.

For guidance on how to analyze a double open-front, or corridor only shear wall diaphragm, and help engineers better understand flexibility issues associated with these types of structures, see Malone webinar series Feb 2020.

The analysis techniques provided in those presentations are intended to demonstrate one method of analysis, but not the only means of analysis.

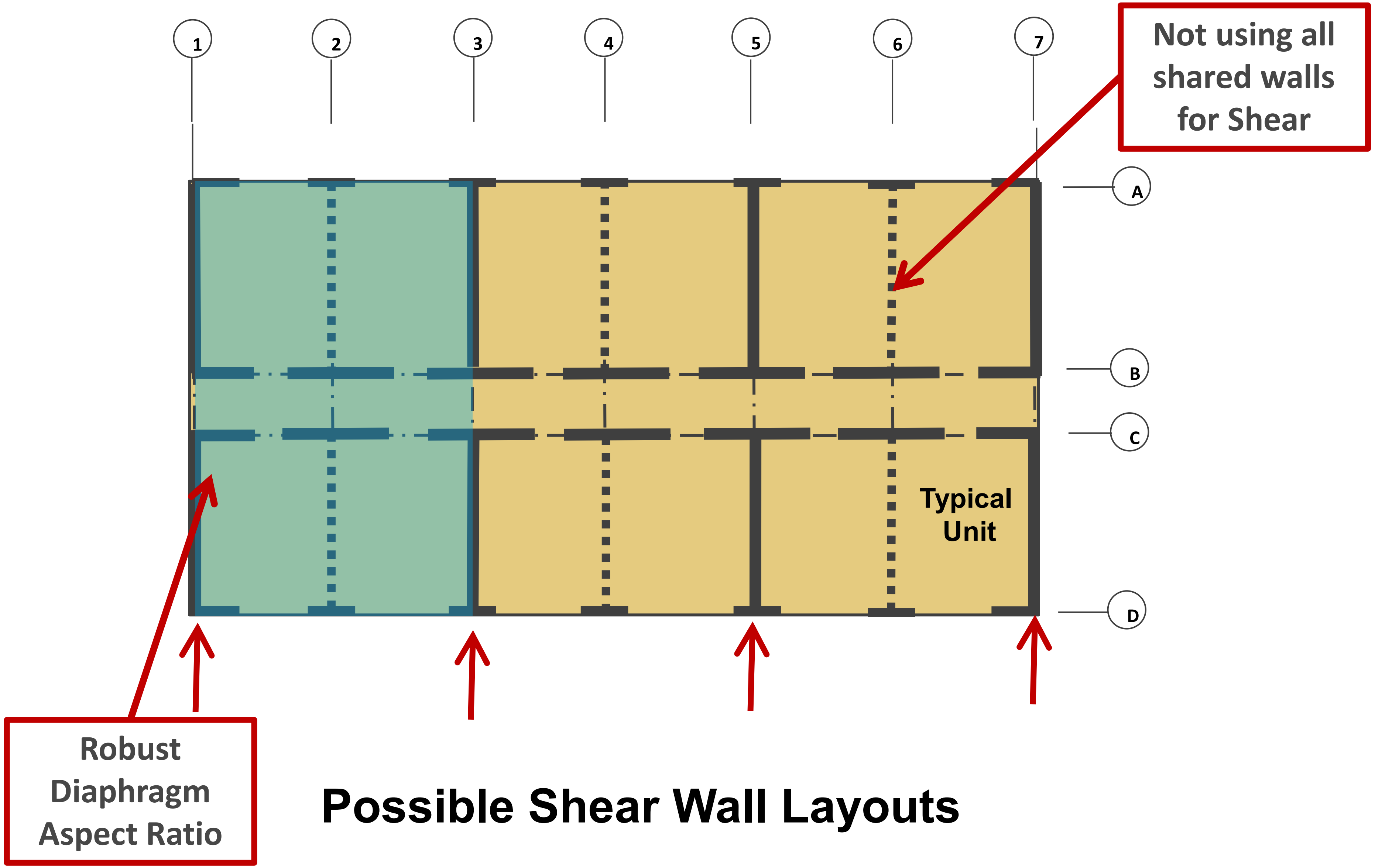
Rigid or Flexible Diaphragm?

Light Frame Wood Diaphragms often default to Flexible Diaphragms

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

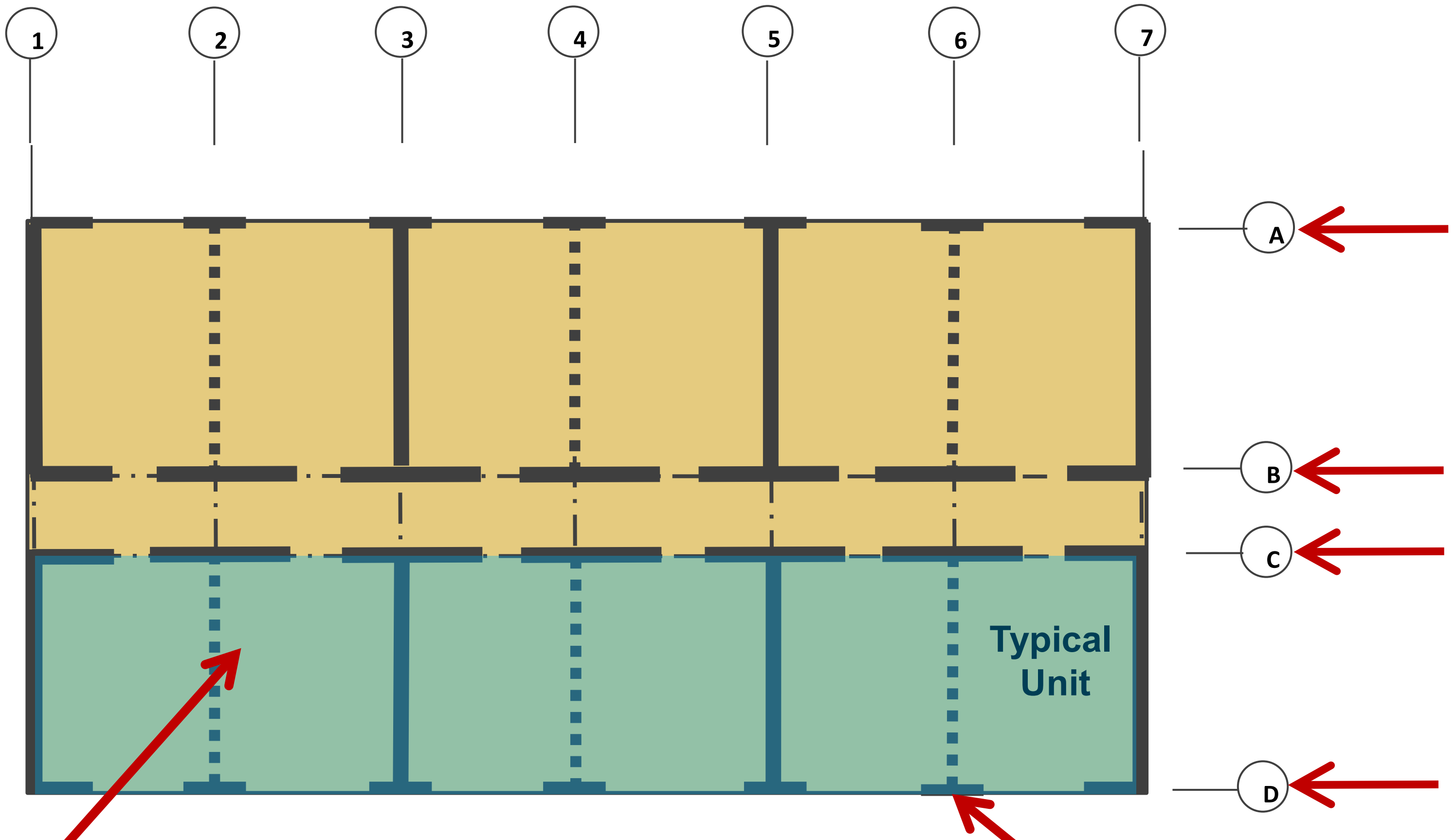
Diaphragms constructed of wood structural panels are permitted to be idealized as flexible

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)

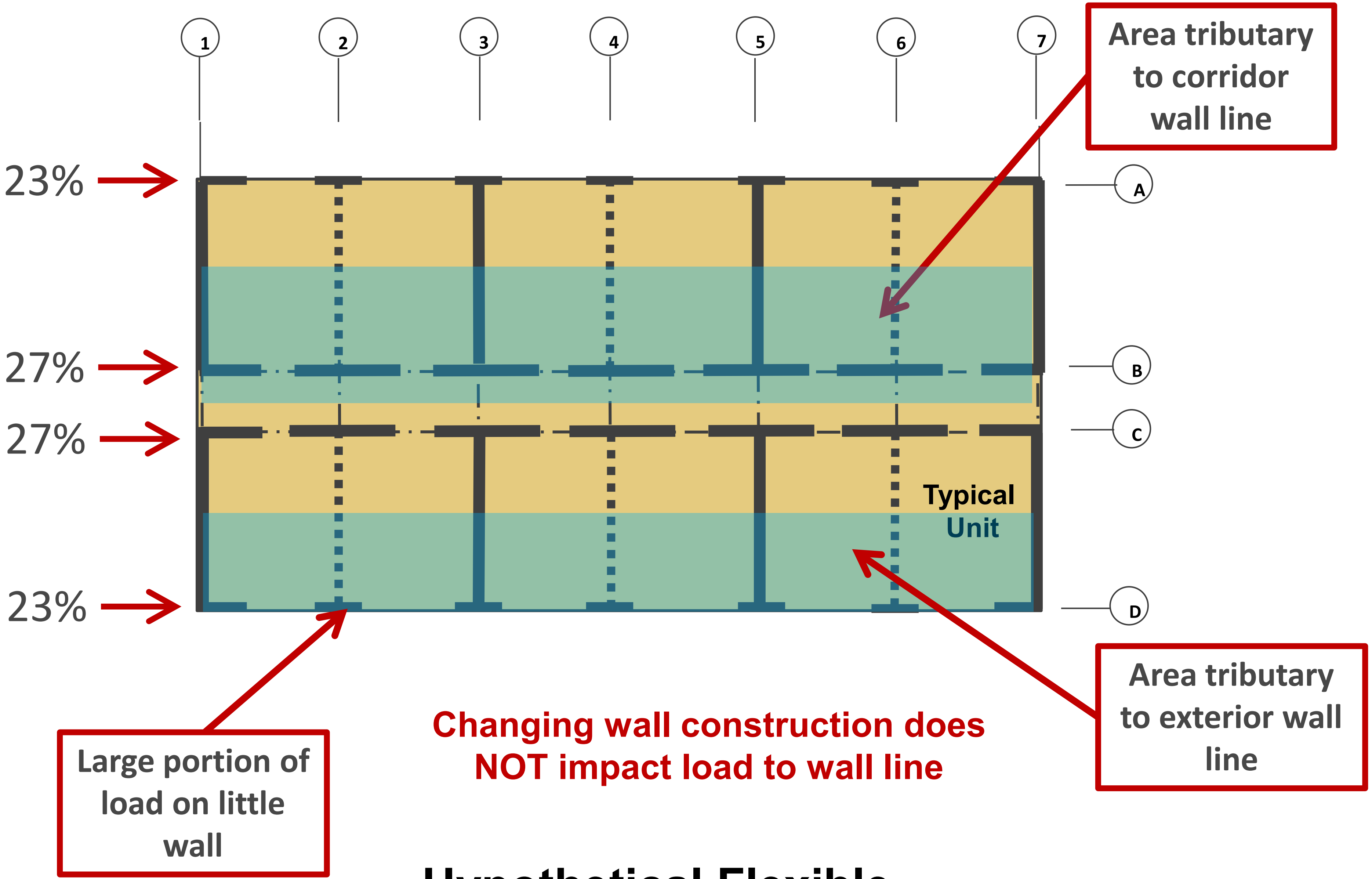
Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

[…]

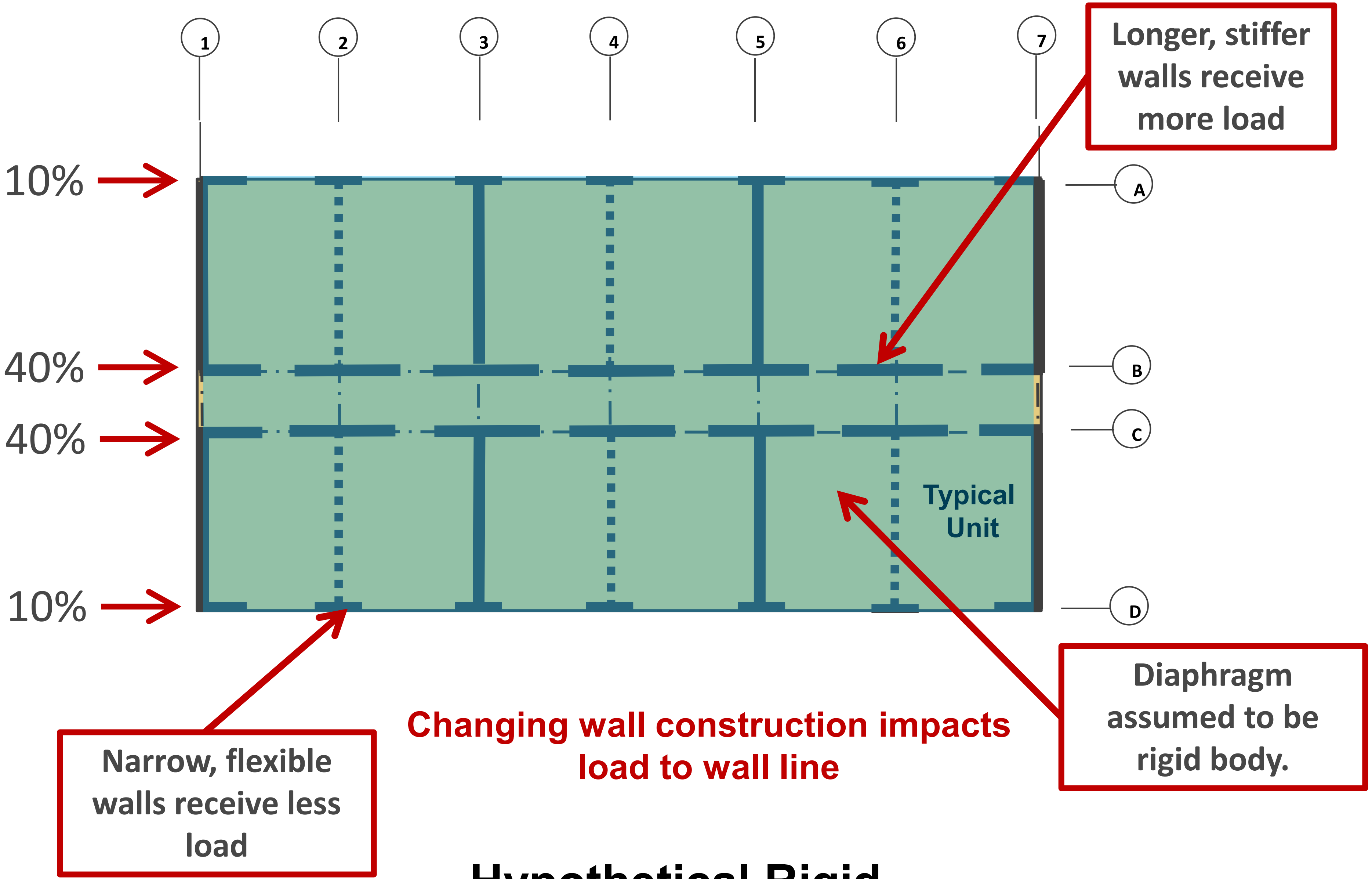
c. In structures of light-frame construction where all of the following conditions are met:

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

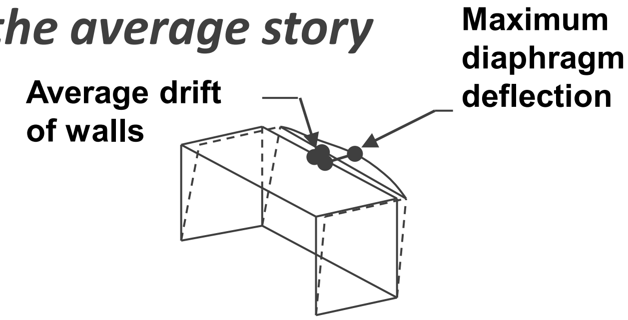
Can a Rigid Diaphragm be Justified?

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.*

IBC 2012 Chapter 2 Definition (Wind & Seismic)

*A diaphragm is **rigid** for the purpose of distribution of story shear and torsional moment when the **lateral deformation of the diaphragm is less than or equal to two times the average story drift**.*



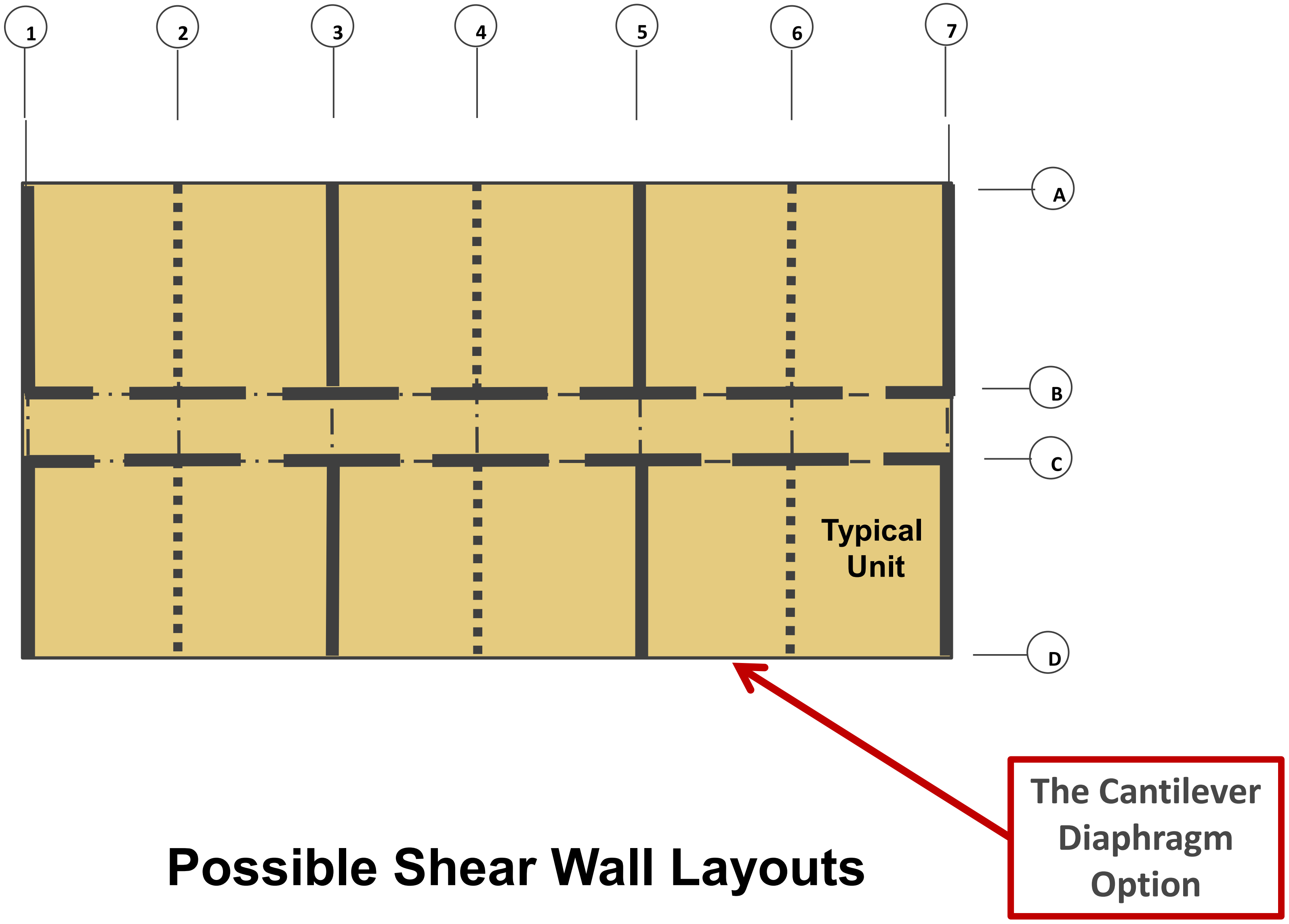
Two More Diaphragm Approaches

Semi-Rigid Diaphragm Analysis

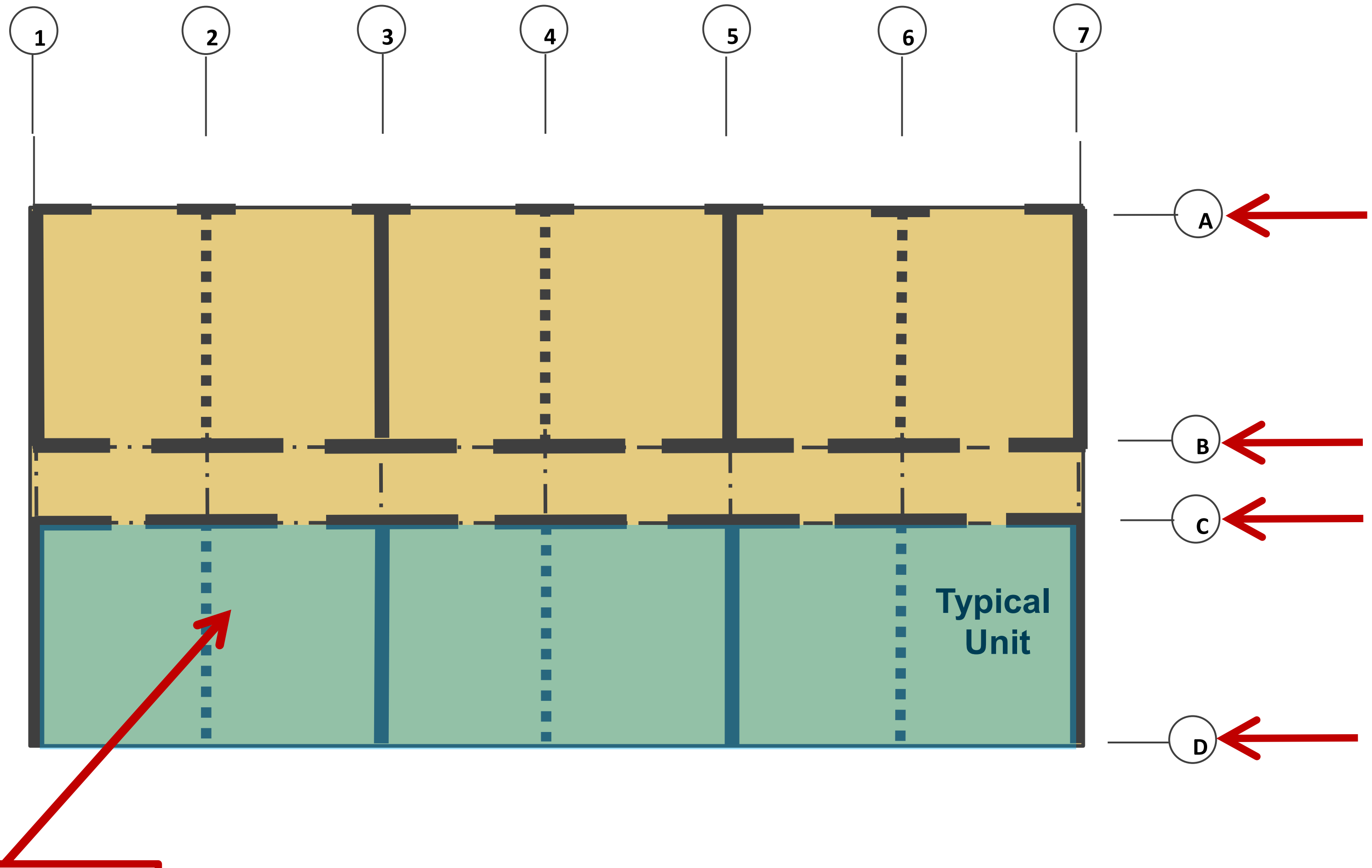
- Neither idealized flexible nor idealized rigid
- Explicit modeling of diaphragm deformations with shear wall deformations to distribute lateral loads
- Not easy

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



Possible Shear Wall Layouts

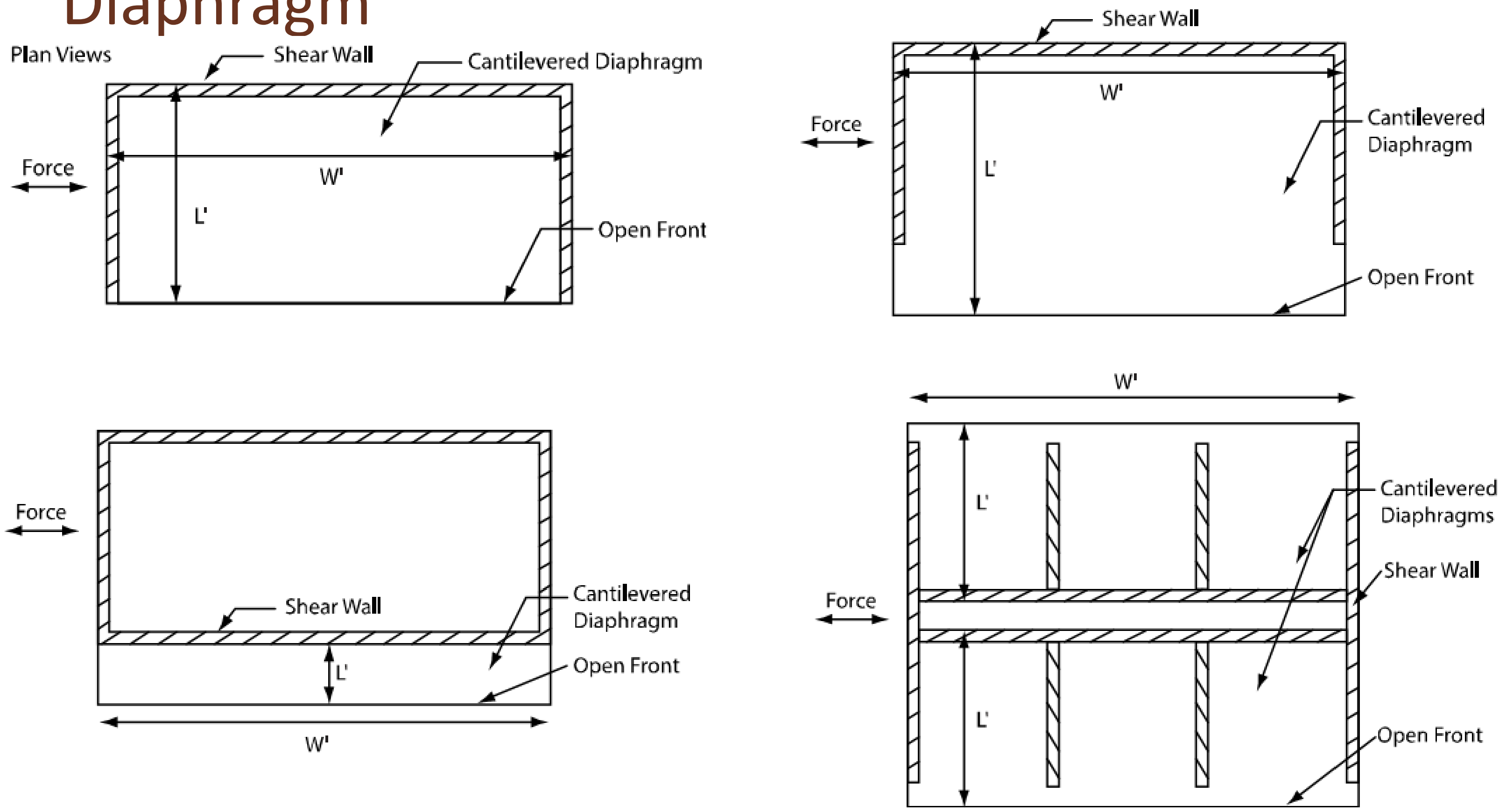


Robust Aspect Ratio but only supported on 3 sides...

Possible Shear Wall Layouts

Cantilevered Diaphragms in SDPWS 2015

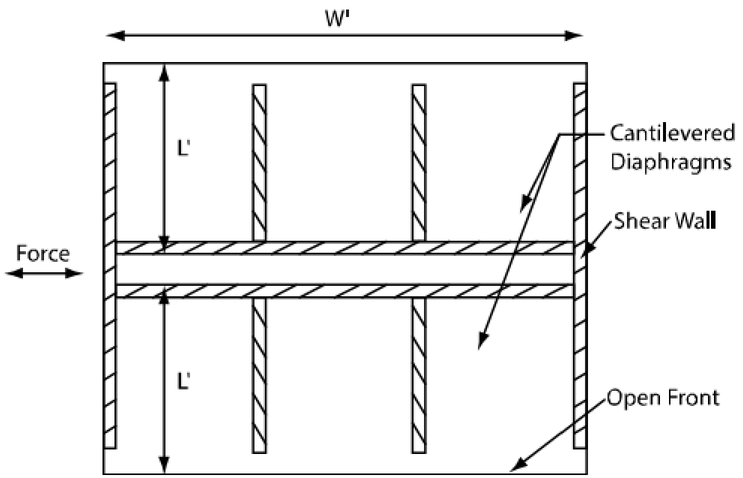
- Open Front Structure with a Cantilevered Diaphragm



AWC SDPWS 2015 Figure 4A

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.

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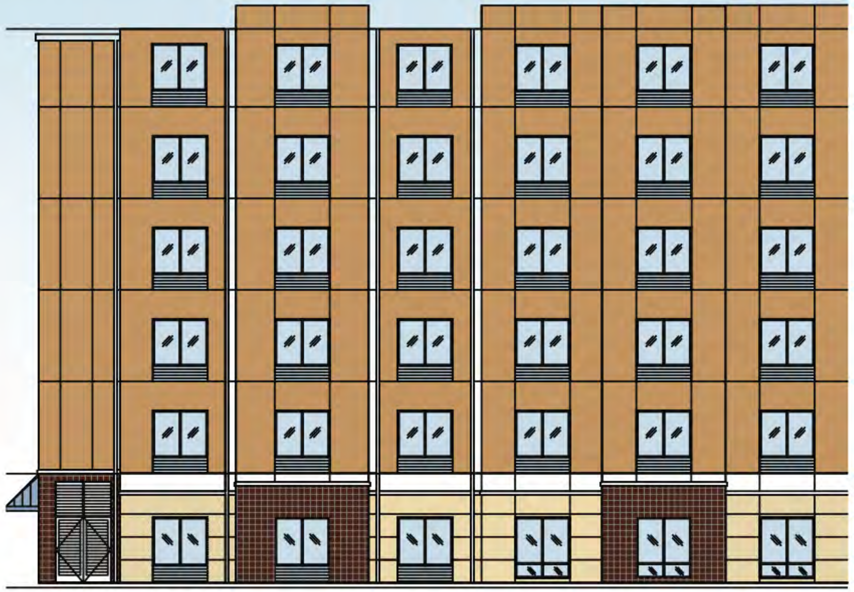
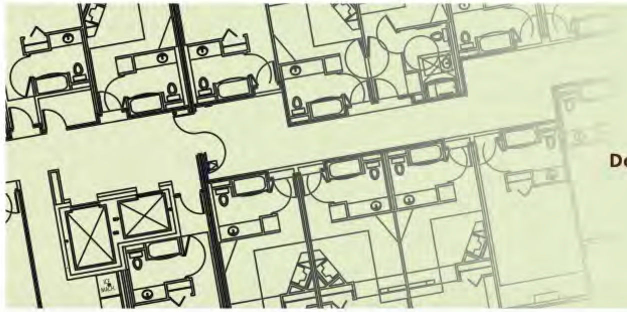
DESIGN EXAMPLE

Five-Story Wood-Frame Structure over Podium Slab





Developed for WoodWorks by
Douglas S. Thompson, PE, SE, SECB
STB Structural Engineers, Inc.
Lake Forest, CA

WoodWorks™
WOOD PRODUCTS COUNCIL

DESIGN EXAMPLE

A Design Example of a Cantilever Wood Diaphragm

Photo: Richard Lubrant

Photo: Andrew Pogue

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

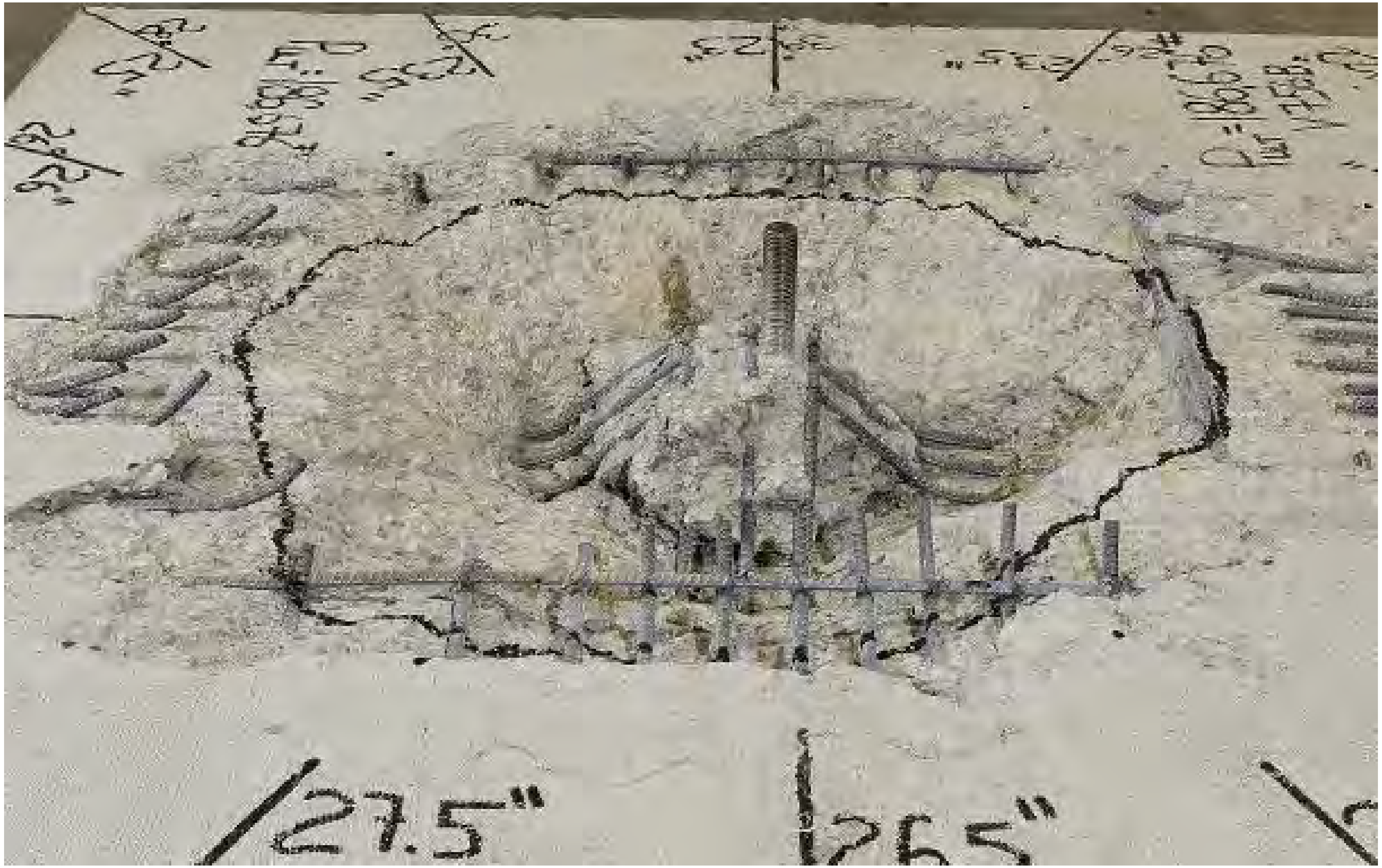
Photos: TOP: Crescent Terminus, Architect: Lord Aeck Sargent, Engineer: SCA Consulting Engineers, Location: Atlanta, GA
INSET: Carbon 12, Architect: Path Architecture, Engineer: Munzig Structural Engineers, Location: Portland, OR

Wi

Shear Wall to Podium Slab Interface

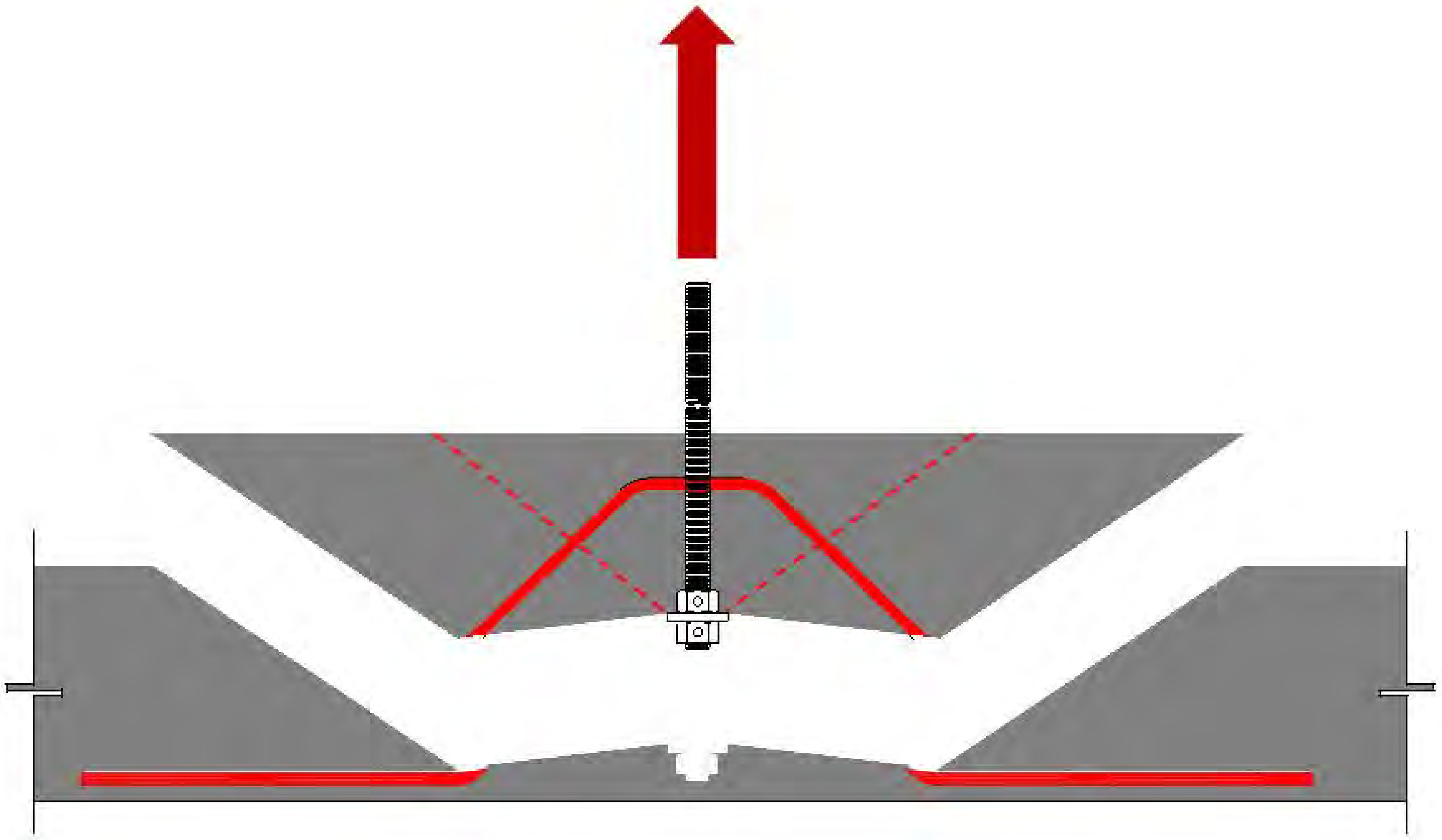
- Amplification of seismic forces is required for elements supporting discontinuous walls per ASCE 7-10 12.3.3.3
- Overstrength factor of 3 (may be reduced to 2.5 per footnote g of Table 12.2-1) is required
- Attachment to concrete slab must also conform to ACI 318 Appendix D
- Typically will be transitioning from ASD for wood design to LRFD for concrete design
- Hold down attachments to concrete options: embedded nuts or plates, sleeves through slab, welded studs & reinforcing

Tie Down Attachment to Concrete



Source: Strongtie

Tie Down Bolt with Washer



Source: Strongtie

Tie Down Anchor Chair in Cast Slab

Source: Earthbound Anchors

Embedded Steel Plates – Weld on Rods

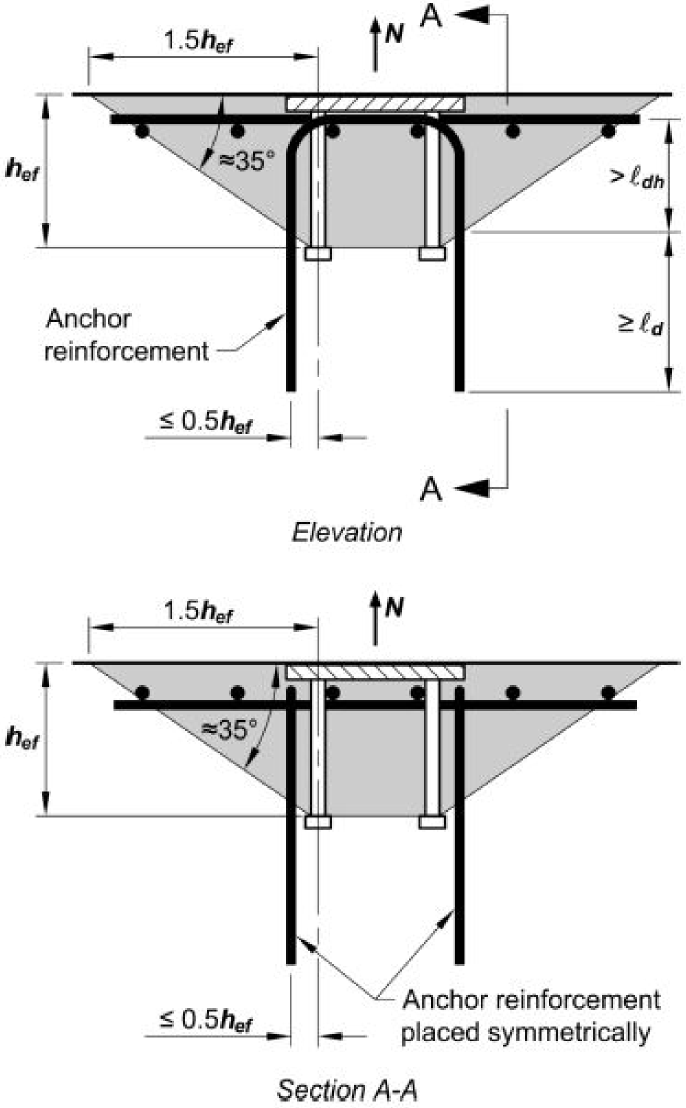
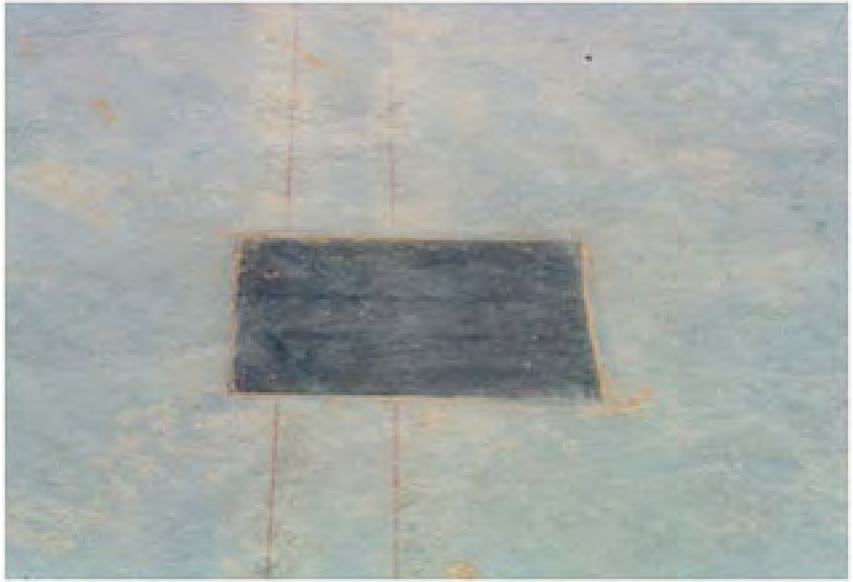


Fig. RD.5.2.9—Anchor reinforcement for tension.



Tie Down Anchors – Precast Through Bolt

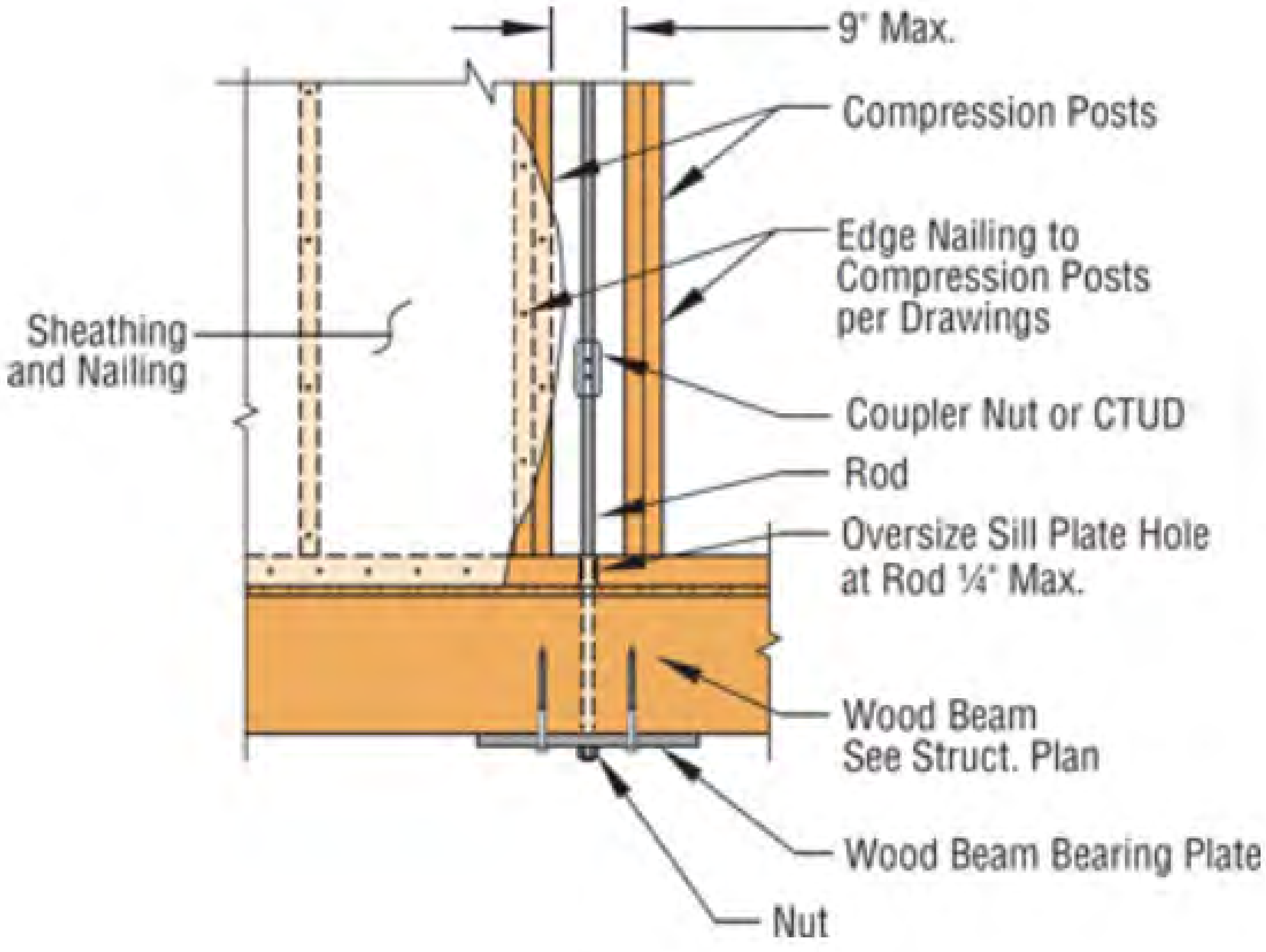
Tie Down Anchors – Through Podium

Discontinuous Shear Walls

Karuna I
Holst Architecture

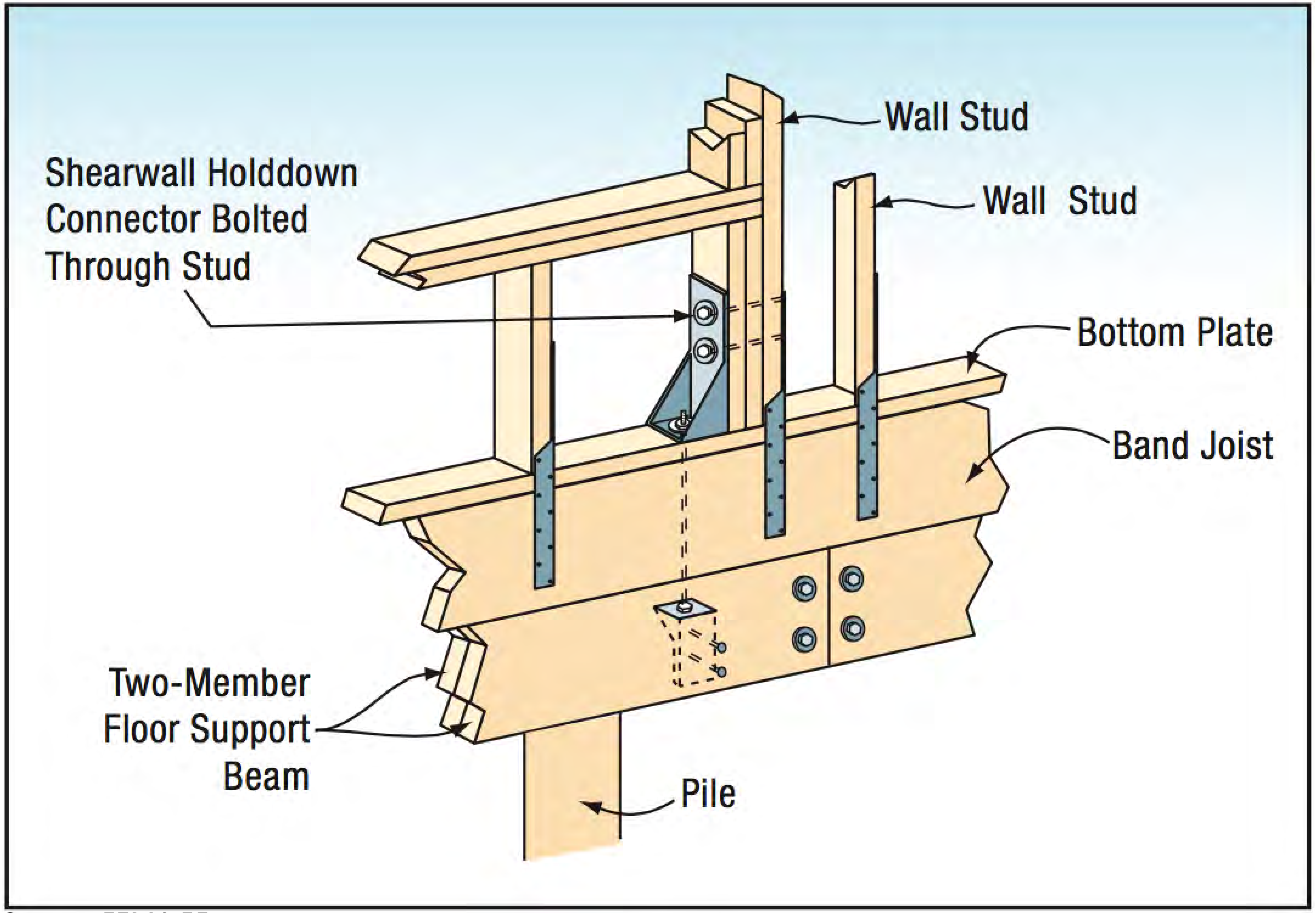
Photo: Terry Malone

Offset Shear Wall Overturning Resistance



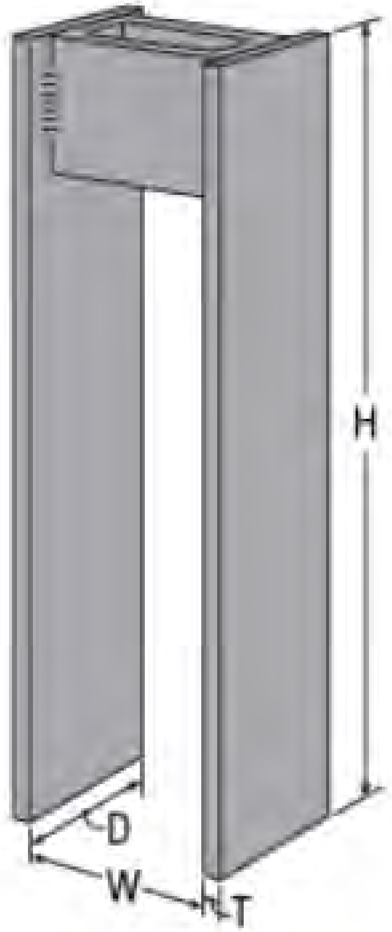
Source: Strongtie

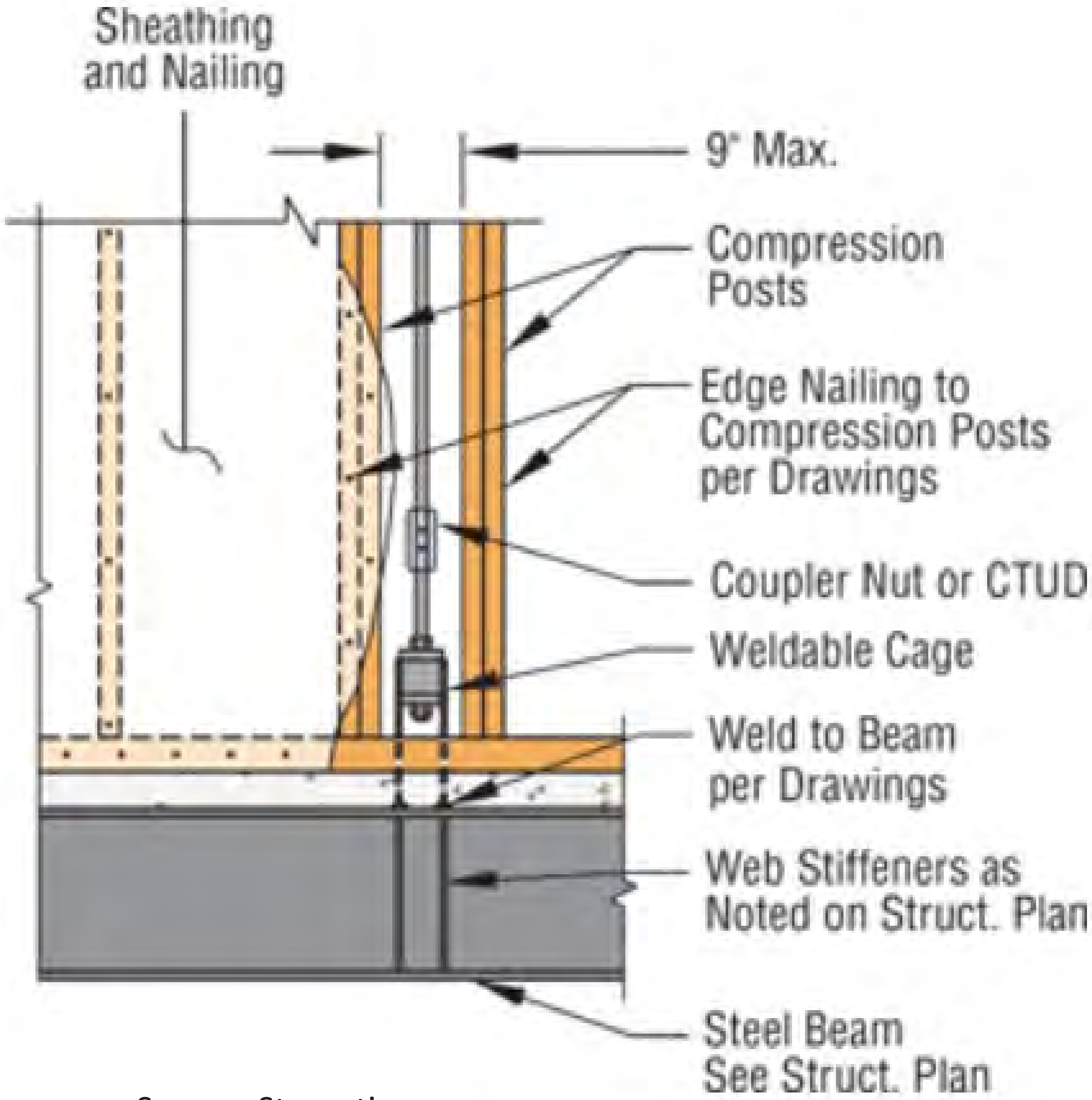
Offset Shear Wall Overturning Resistance



Source: FEMA 55

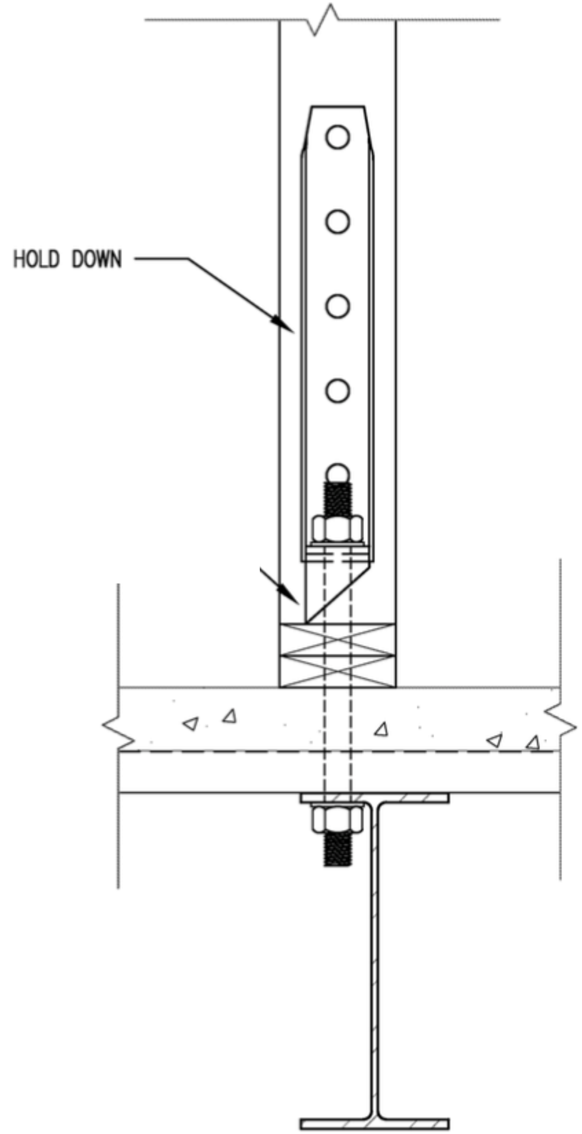
Tie Down to Steel Beam Attachment

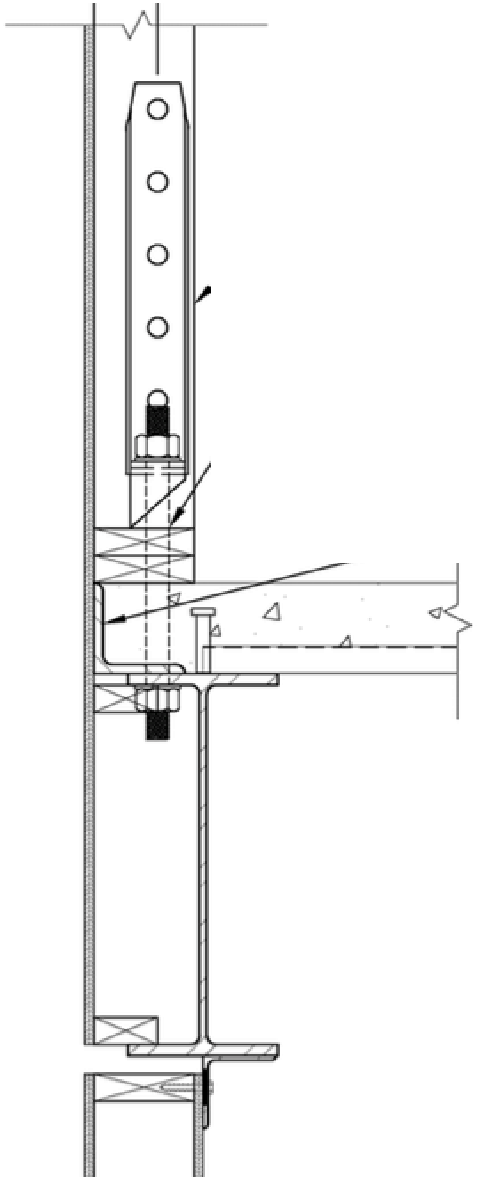




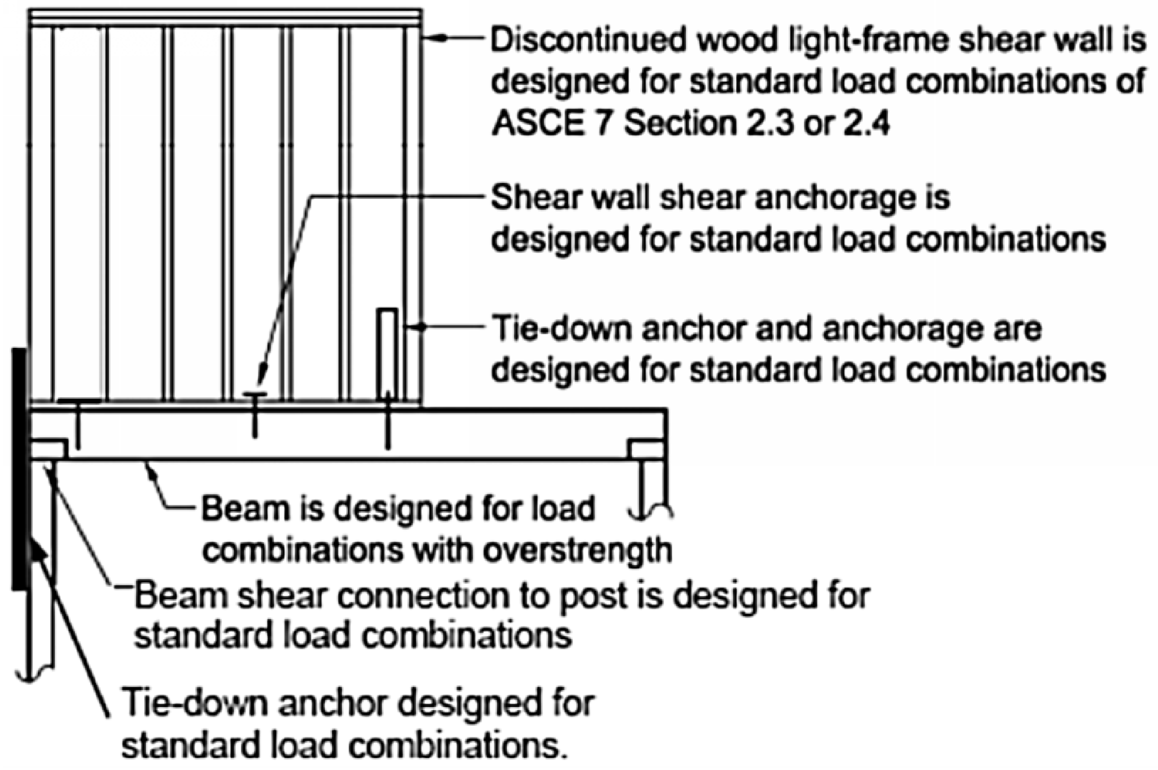
Source: Strongtie

Tie Down to Steel Beam Attachment





Shear Wall to Podium Slab Interface



ASCE 7-10 Section 12.3.3.3 and Commentary C12.3.3.3 provides guidance on seismic load requirements for various elements supporting discontinuous shear walls

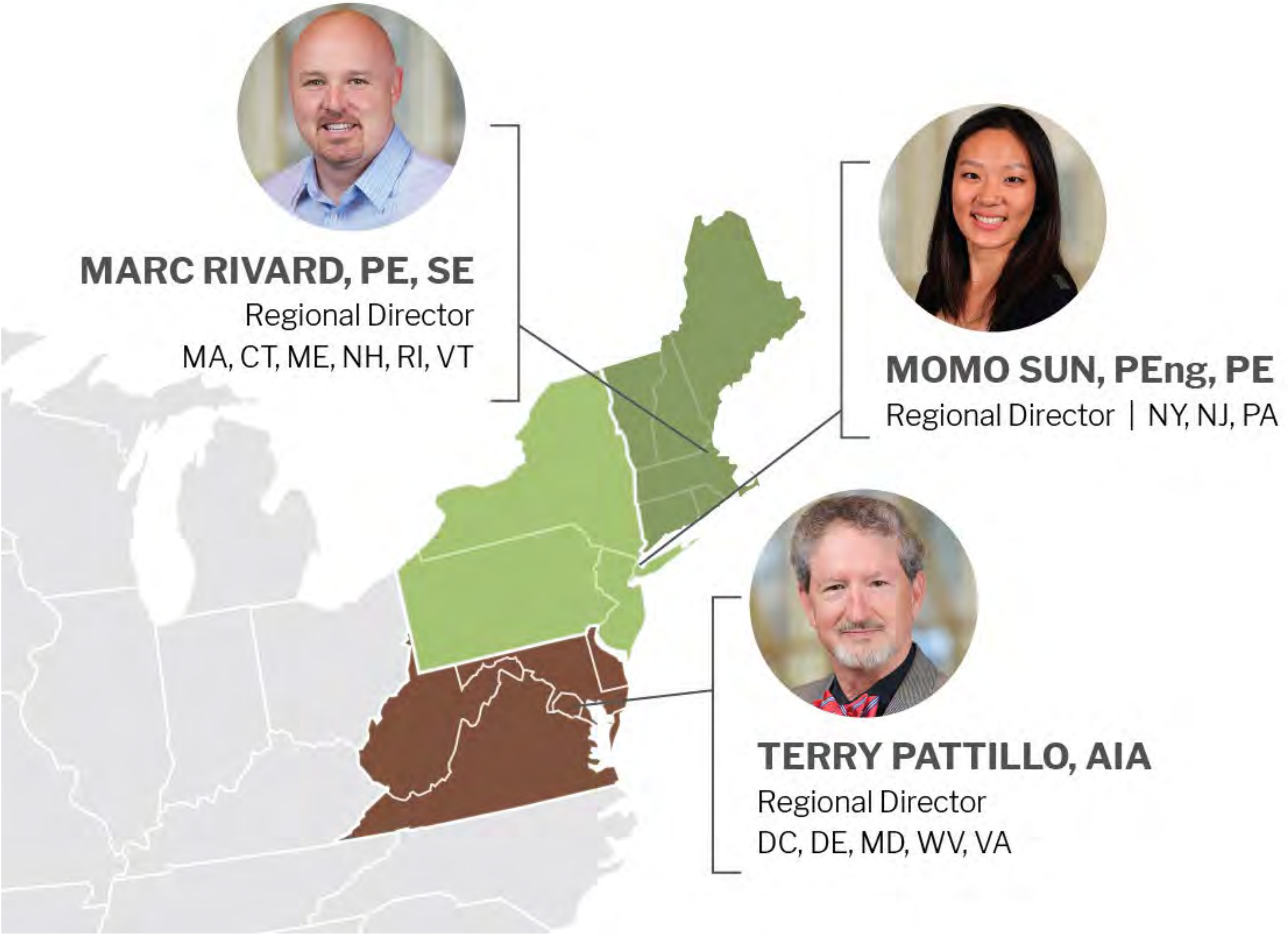
Photo: Brett Drury

Questions? Contact us anytime!

Marc Rivard, PE, SE
(617) 997-3890
marc.rivard@woodworks.org

Momo Sun, PE, PEng
857-242-8975
momo.sun@woodworks.org

Terry Pattillo, AIA
(919) 995-6672
terryp@woodworks.org



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