Structural Engineering of Mid-Rise Wood-Frame Construction

Anthony Harvey, PE
WoodWorks

Funding Partners

WoodWorks Wood Products Council

2022 Board Partners

Boise Cascade

Georgia-Pacific

LP Building Solutions

West Fraser

Weyerhaeuser

2022 Market Development Partners

binderholz

DR Johnson Wood Innovations

Freres Engineered Wood

Global IFS

Hasslacher Norica Timber — From wood to wonders.

Mercer

Rosboro — Building Better

Sansin

SmartLam North America

Sterling Structural

StructureCraft

Vaagen Timbers

2022 Industry Advantage Partners

Eurotec

Kalesnikoff Timber Inspired

RedBuilt

Simpson Strong-Tie

Structurlam Mass Timber Corporation

NOW HIRING

REGIONAL DIRECTOR – CHICAGO, IL OR MINNEAPOLIS, MN METRO AREA

TECHNICAL DIRECTOR – REMOTE, US

REGIONAL DIRECTOR – SEATTLE, WA METRO AREA

WOODWORKS

WOOD PRODUCTS COUNCIL

Design Professionals:
One-on-One Support & Assistance

PROJECT SUPPORT FIELD DIVISION

Find the Regional Director for your location:

Outline

Section 1: Mid-Rise Fire and Life Safety

Section 2: Structural Impacts of Mid-Rise

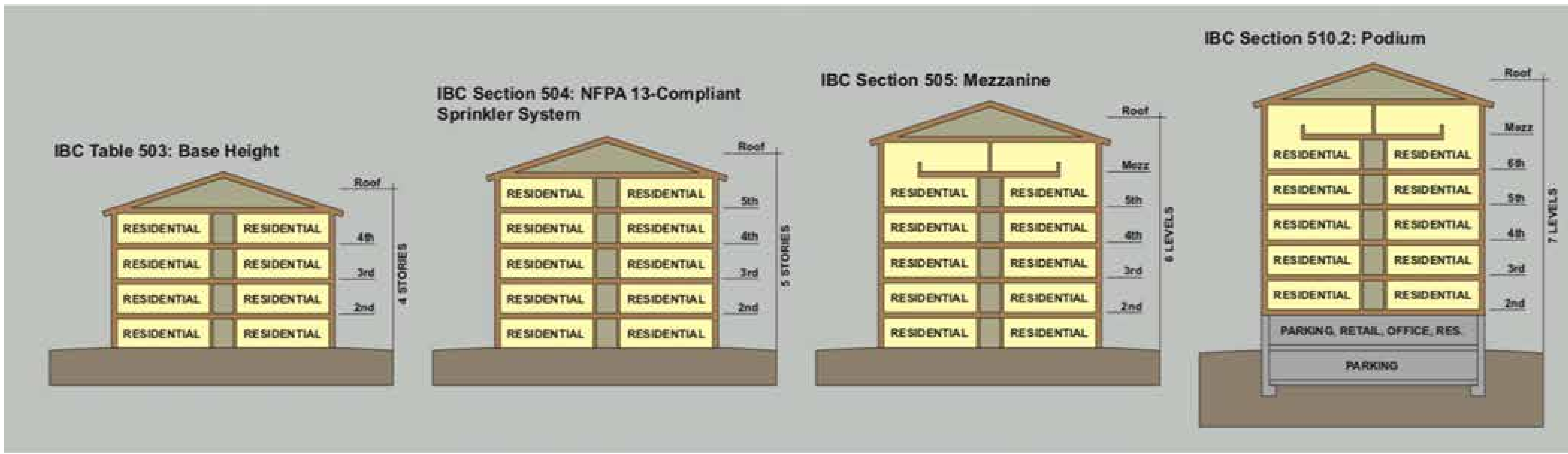
Section 3: Lateral Design of Wood-over-Podium

Wood Mid-Rise Construction

Marselle Condos, Seattle, WA

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

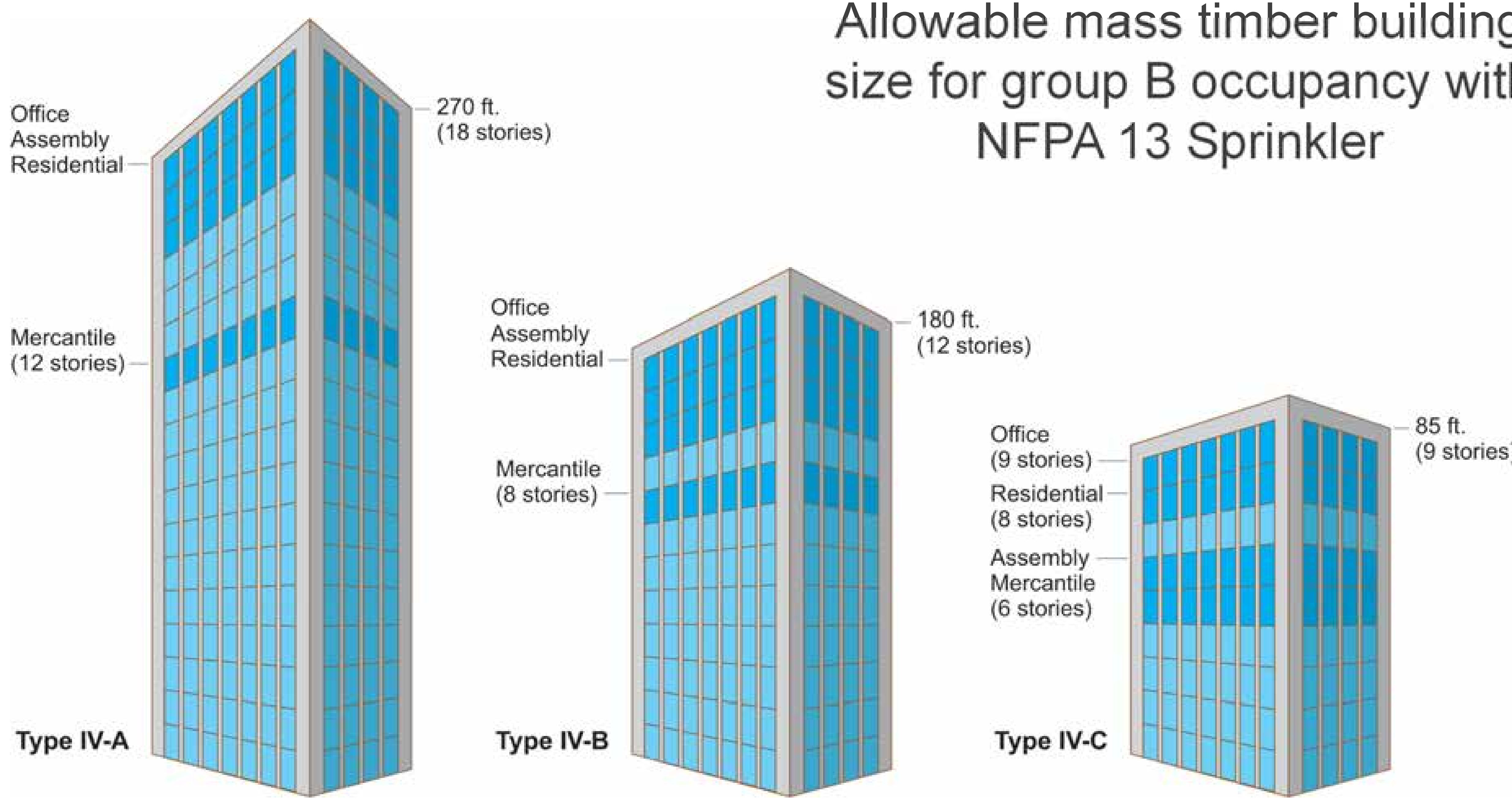
Evolution of Mid-Rise



Credit: WoodWorks

Construction Types

New Options in 2021 IBC
Allowable mass timber building size for group B occupancy with NFPA 13 Sprinkler



Mid-Rise Construction Types

Type III

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

Type V

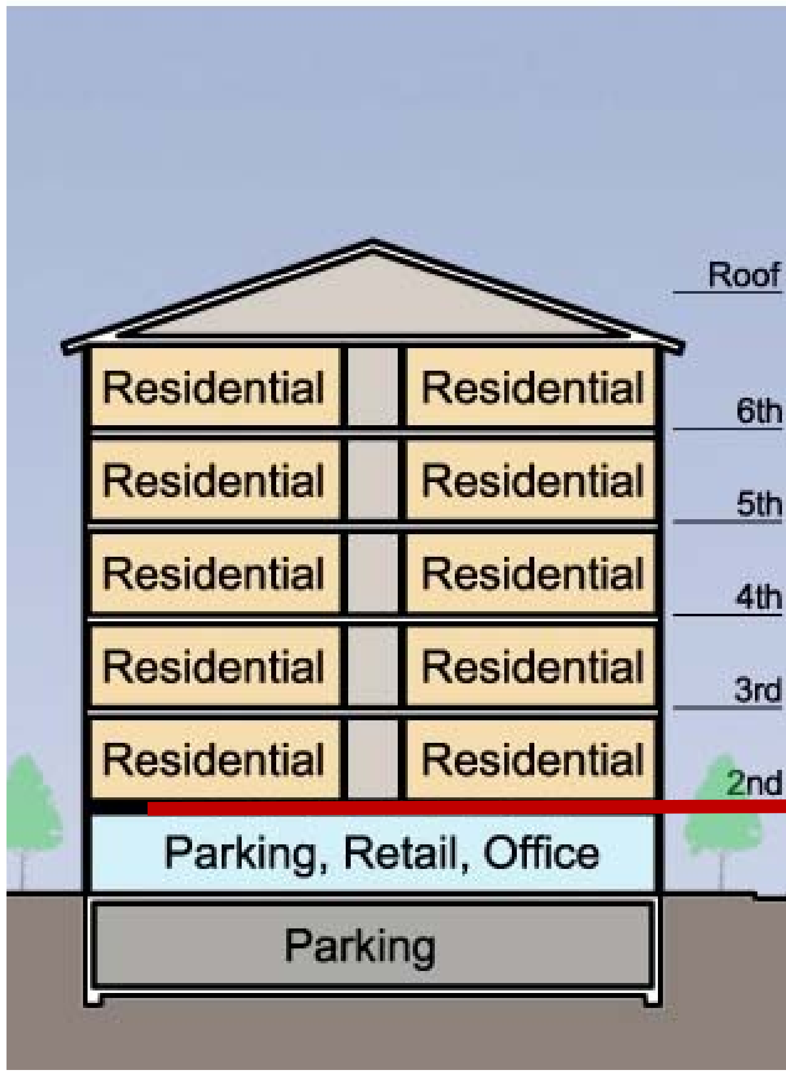
- All building elements are any allowed by code

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

Type IV (Heavy Timber)

- Exterior walls non-combustible (may be FRTW)
- Interior elements qualify as Heavy Timber

Evolution of IBC Mixed-Use Podium



3Hr

Type IA

IBC	2006	2009	2012	2015
Section	509.2	509.2	510.2	510.2
Upper Occupancy	A, B, M, R or S			
Lower Occupancy	S-2 Parking	A, B, M, R or S-2 Parking		Any Except H
Podium Height	1 Story			No Restriction

IBC Provisions for Mixed-Use podium have been evolving.

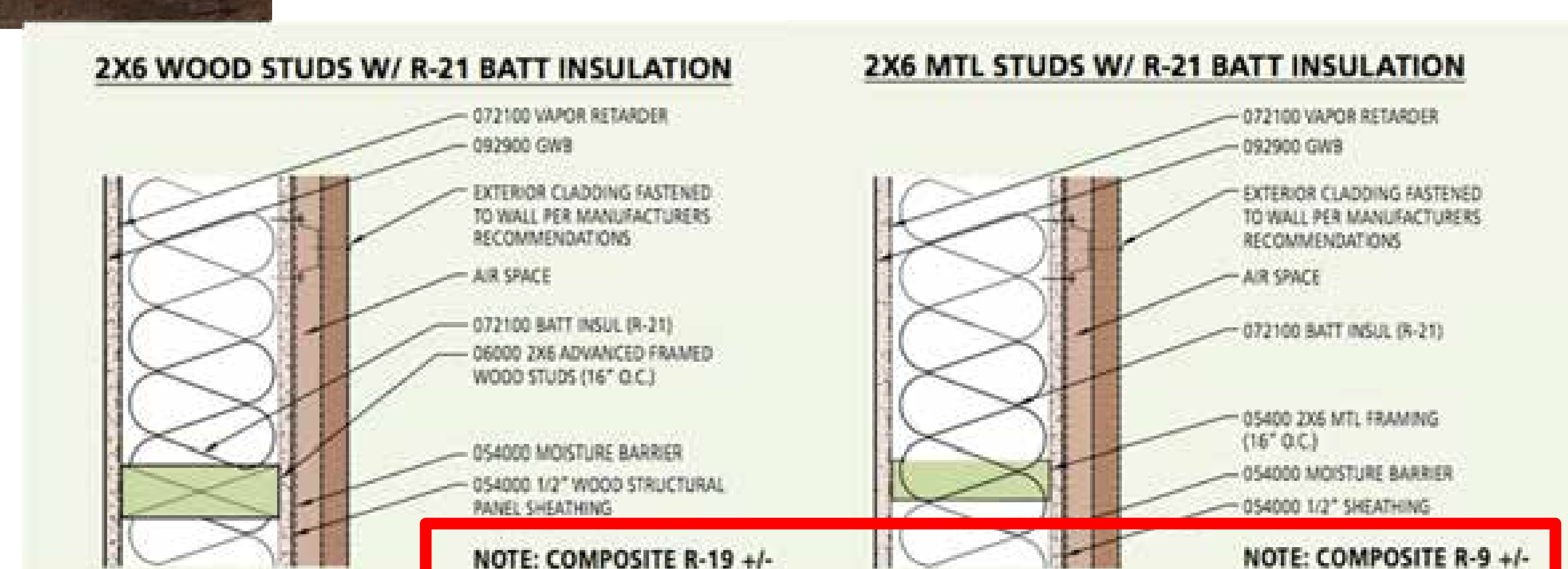
2015 IBC allows multiple podium stories above grade.

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

Let’s Talk Structure

Credit: WoodWorks

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

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
• $10 \text{ ft} \leq X < 30$ feet	1	0	1	0	1	0
• $X \geq 30$ feet	0	0	0	0	HT	0

Credit: WoodWorks

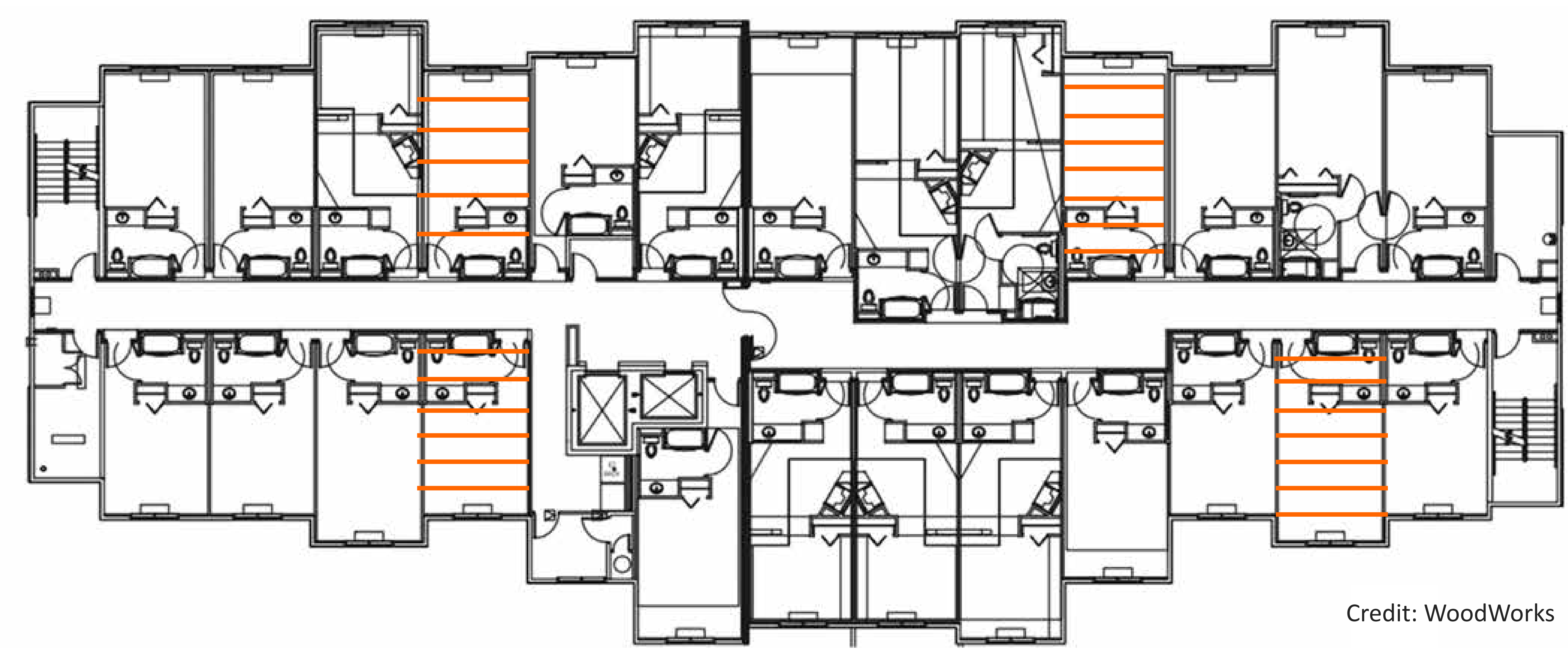
Type III:

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

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?

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

Design for Code Acceptance 3

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

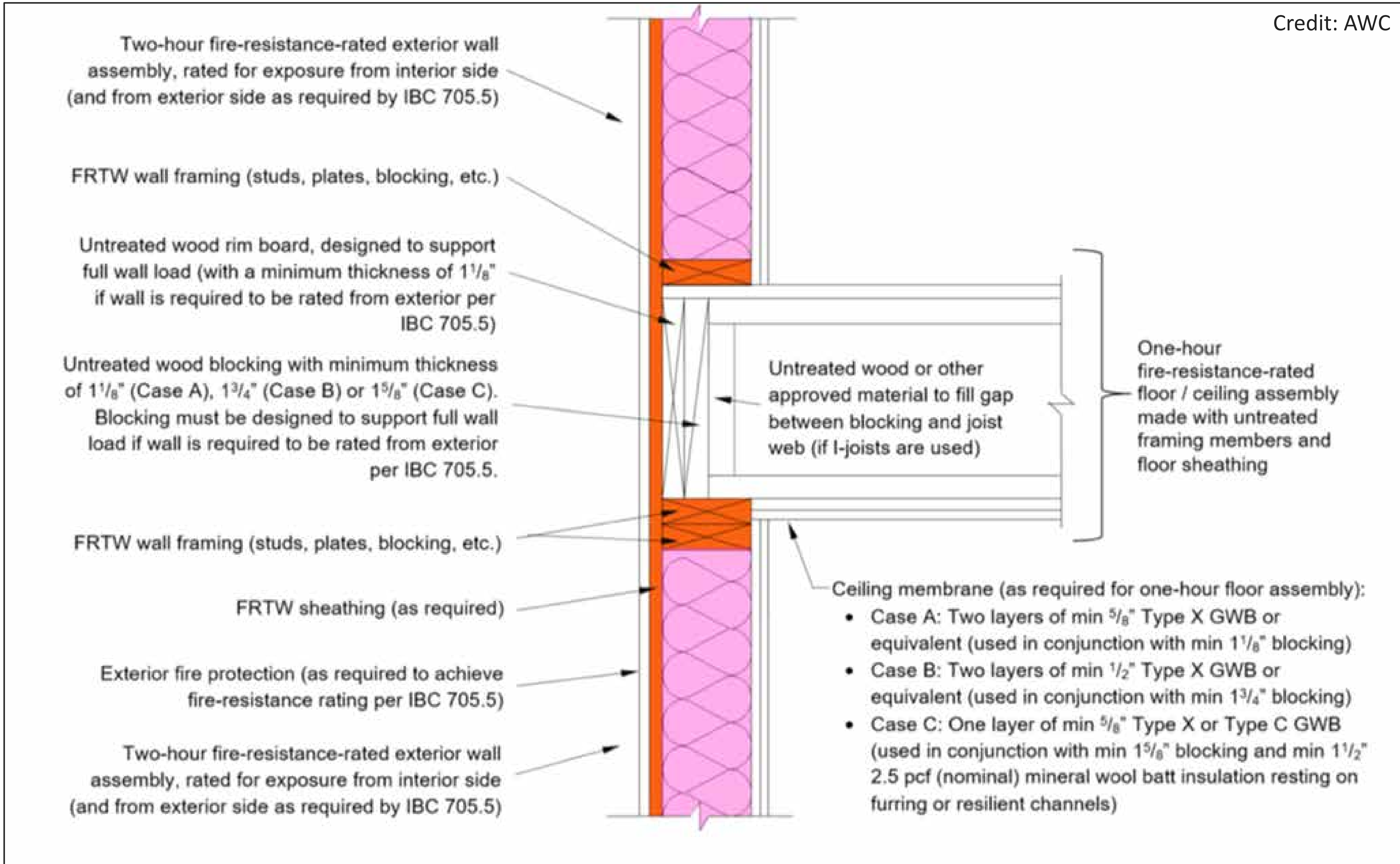


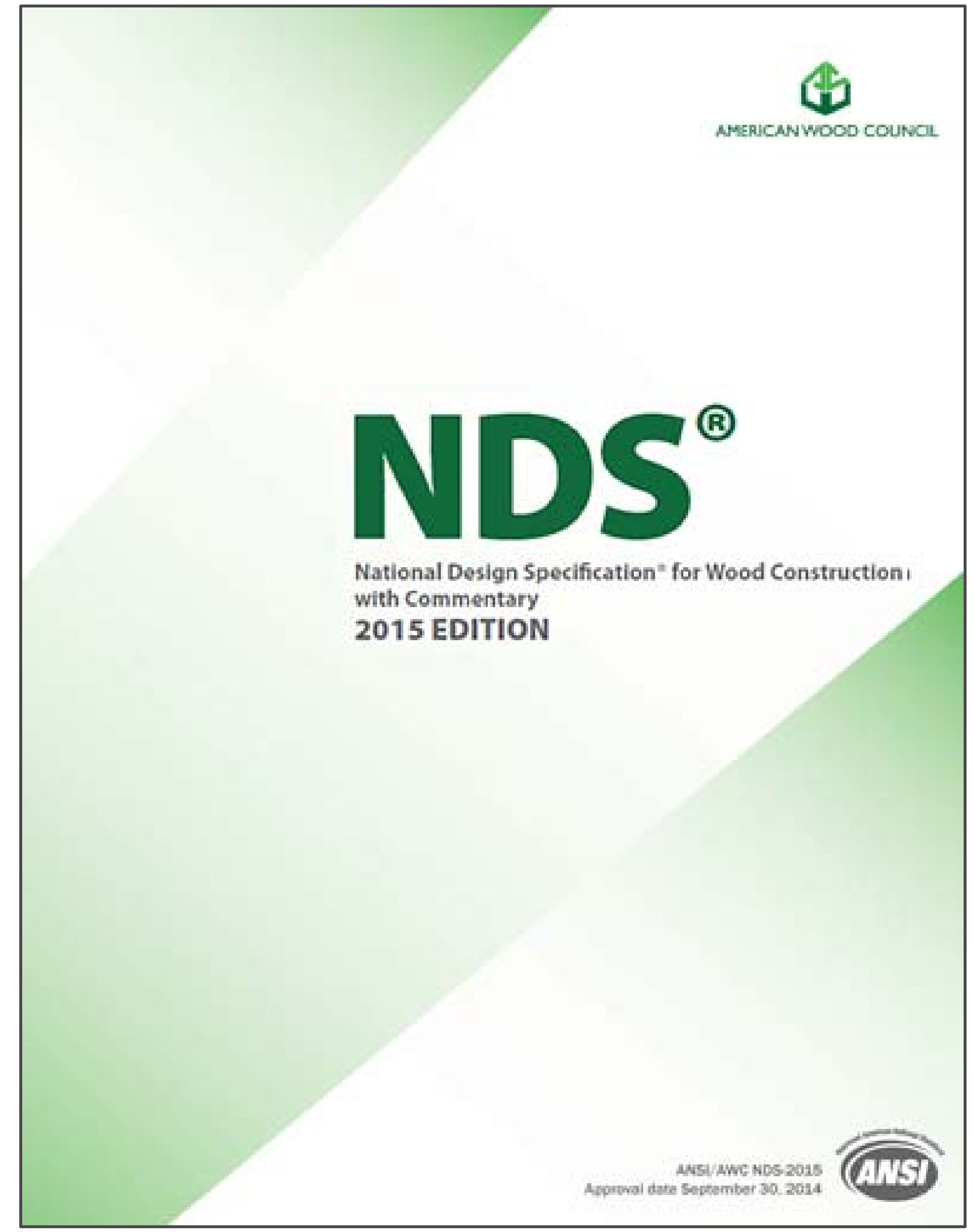
Figure 1A: Example detail for Type III-A exterior wall–floor intersection with rim board and blocking

Type III Exterior Walls – FRT

Structural Impacts of using FRTW

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 Climate Zone			Southern pine 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:

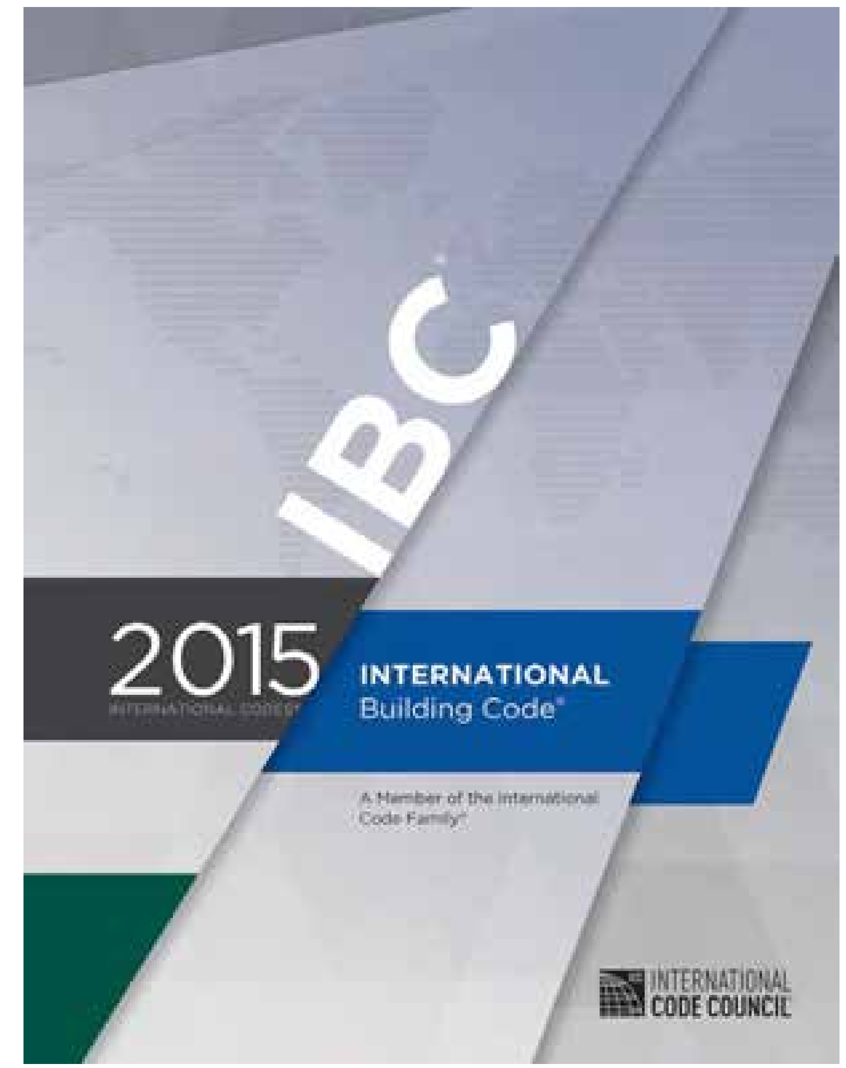
FRT Plywood Thickness (inches)	Untreated Plywood Thickness (inches)
$^{3}/_{8}$	$^{5}/_{16}$
$^{7}/_{16}$	$^{3}/_{8}$
$^{15}/_{32}$	$^{7}/_{16}$
$^{1}/_{2}$	$^{15}/_{32}$

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 **shrinkage of the wood framing will not have adverse effects on the structure or any plumbing, electrical or mechanical systems, or other equipment installed** 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.



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

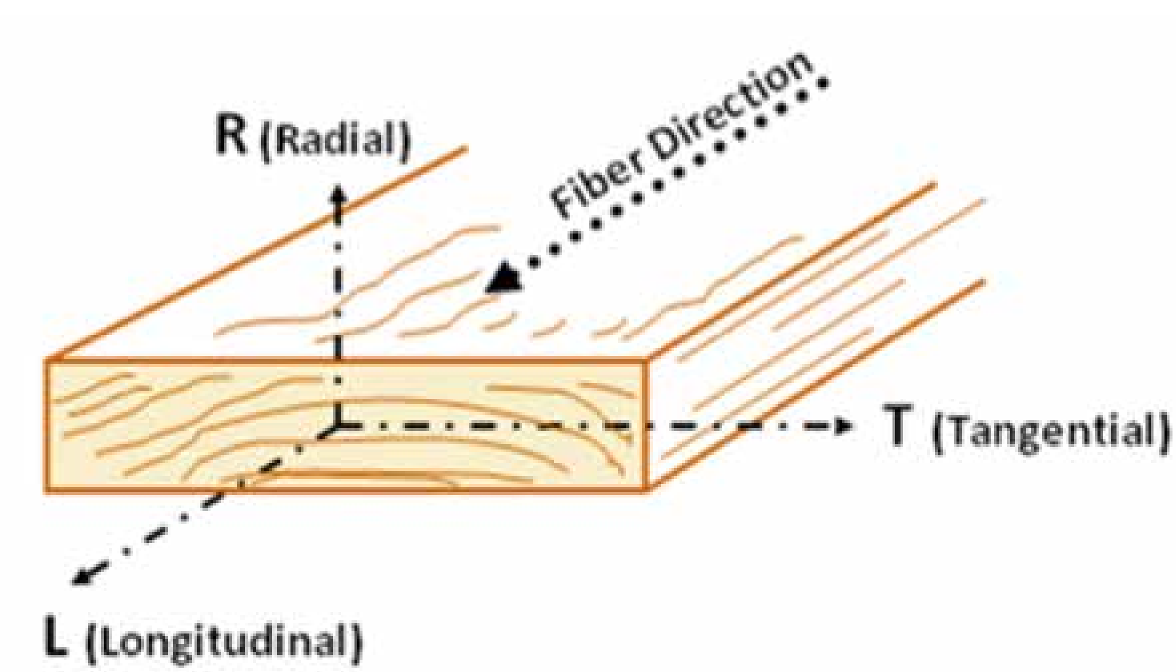
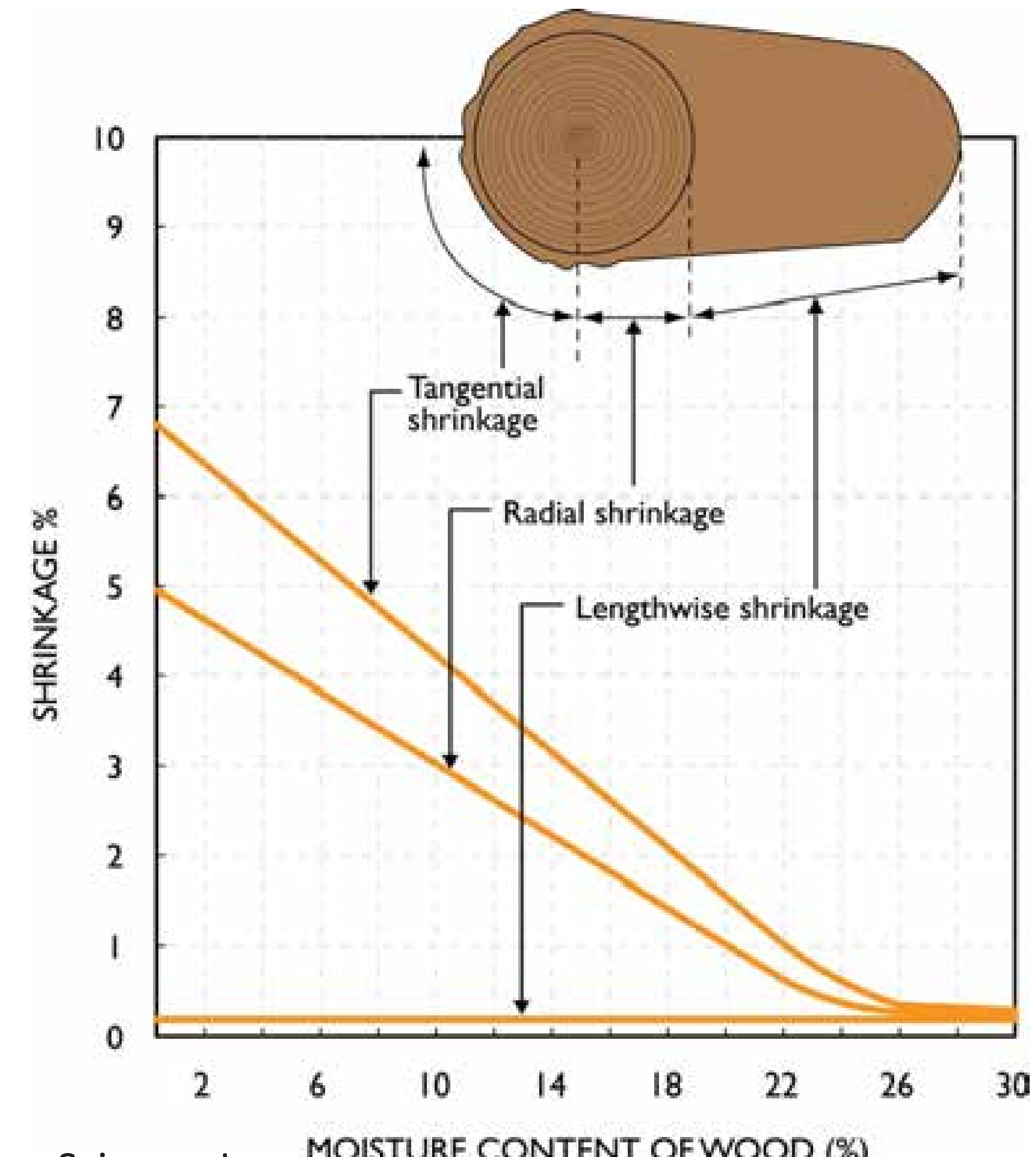


Image: RDH Building Science, Inc.

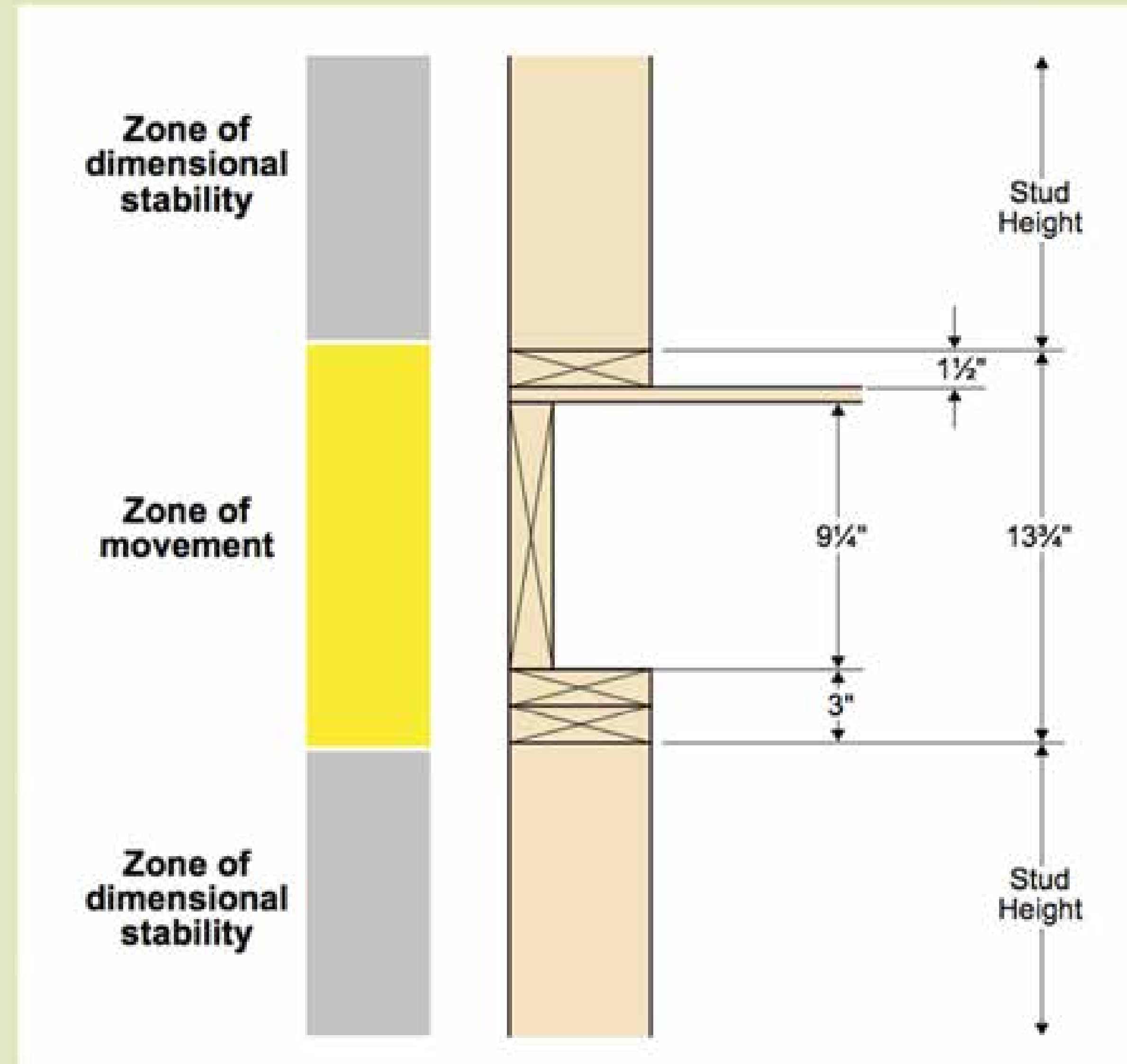


Shrinkage Calculations – Cross Grain Wood

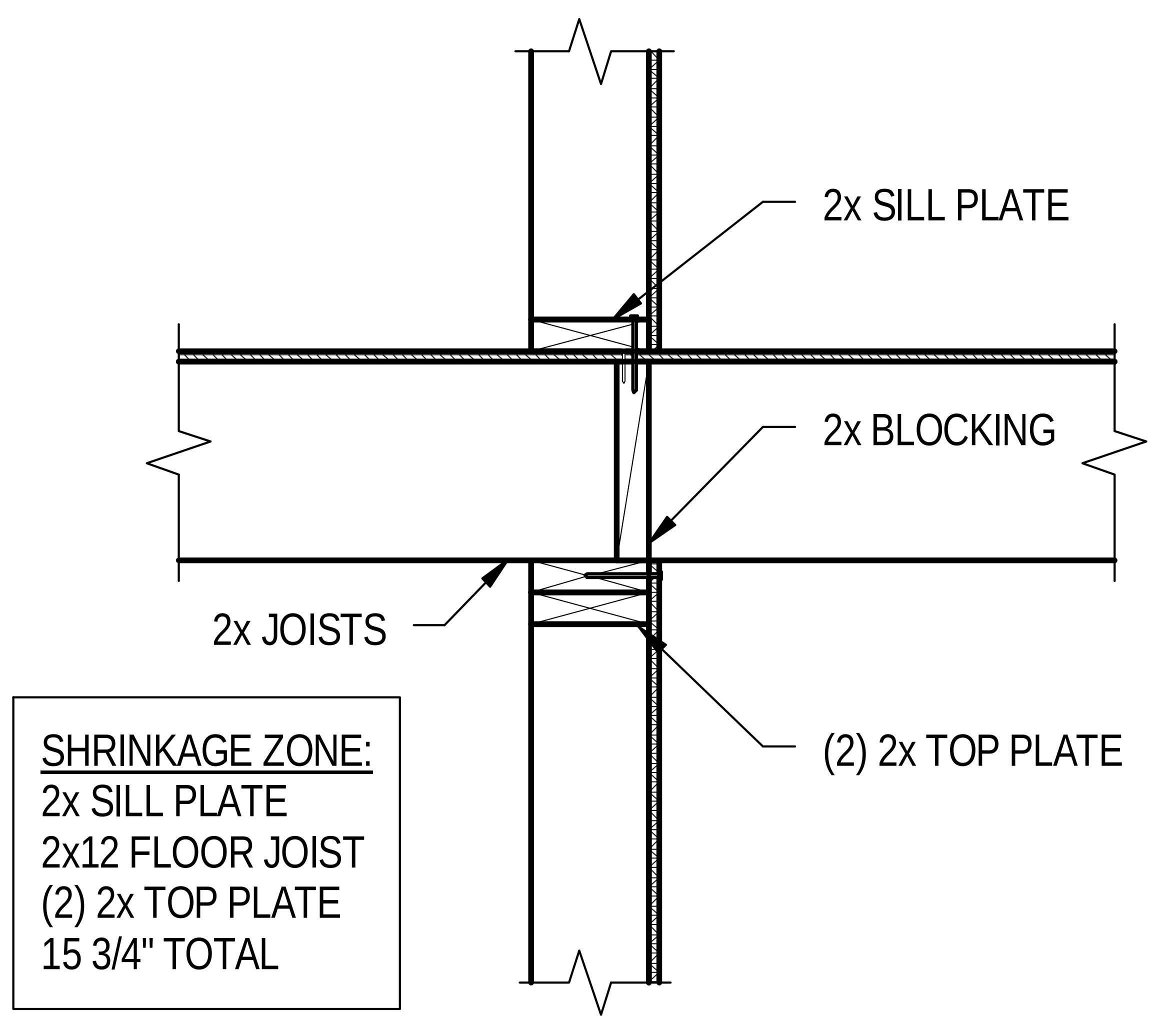
Shrinkage occurs in cross-grain, but not longitudinal, wood dimensions

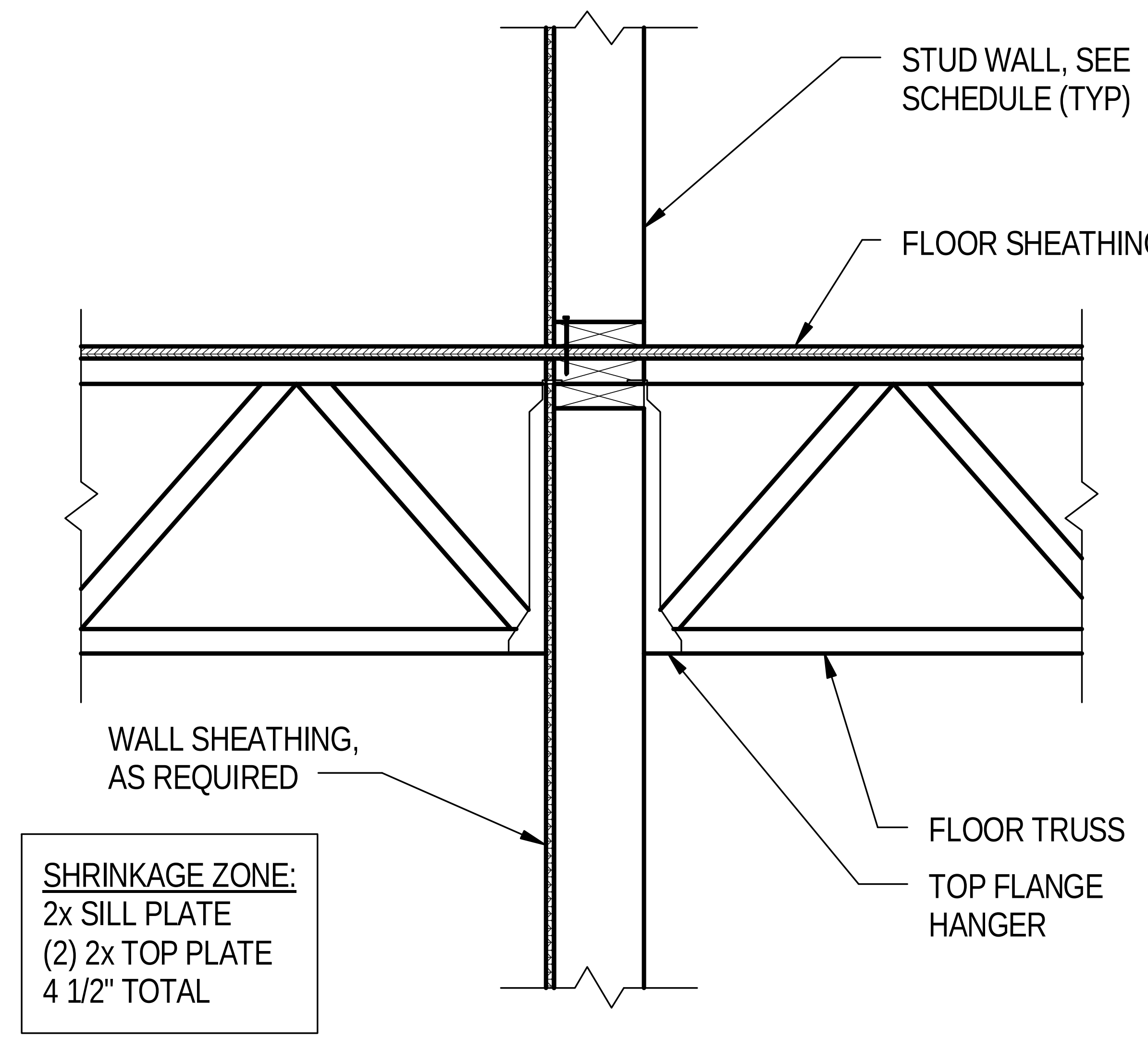
- Primarily in horizontal members
- Wall plates
- Floor/rim joists
- Engineering judgement required when determining what to include in shrinkage zone
- Should Sheathing, I-Joists, Trusses, other products manufactured with low MC be included?

FIGURE 5:
Shrinkage zone in platform-framed detail



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”**

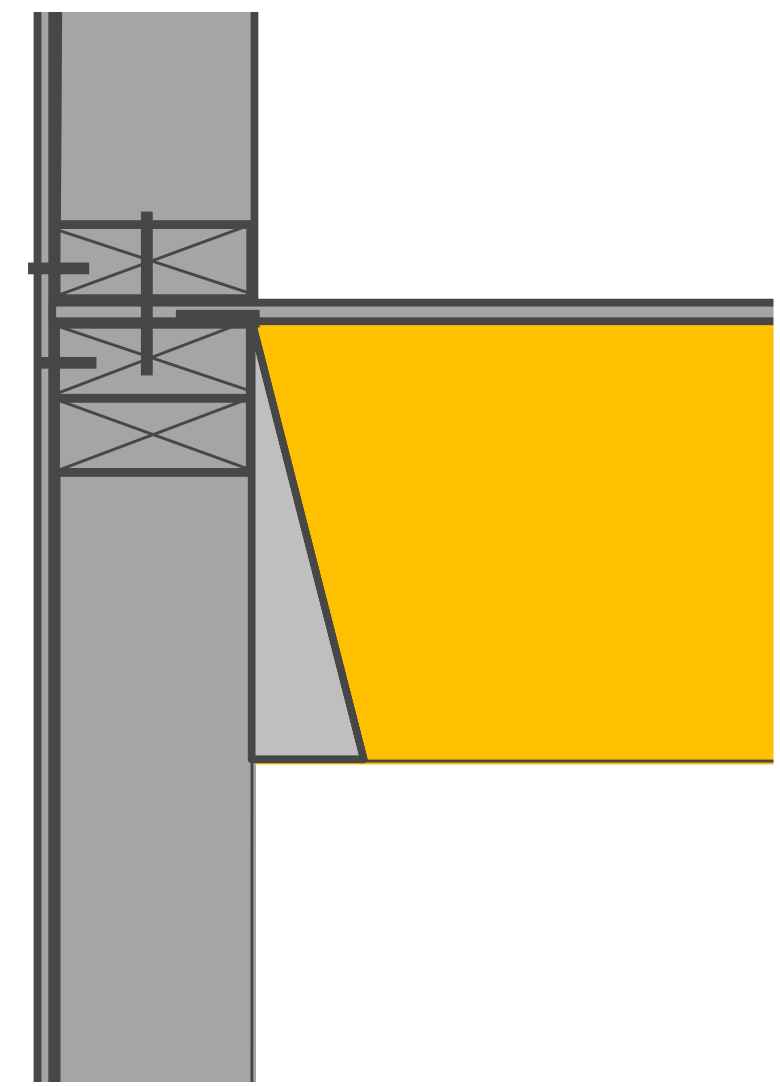
5-story building: **0.4” total**

Minimizing Shrinkage - Detailing

Semi-balloon framing:

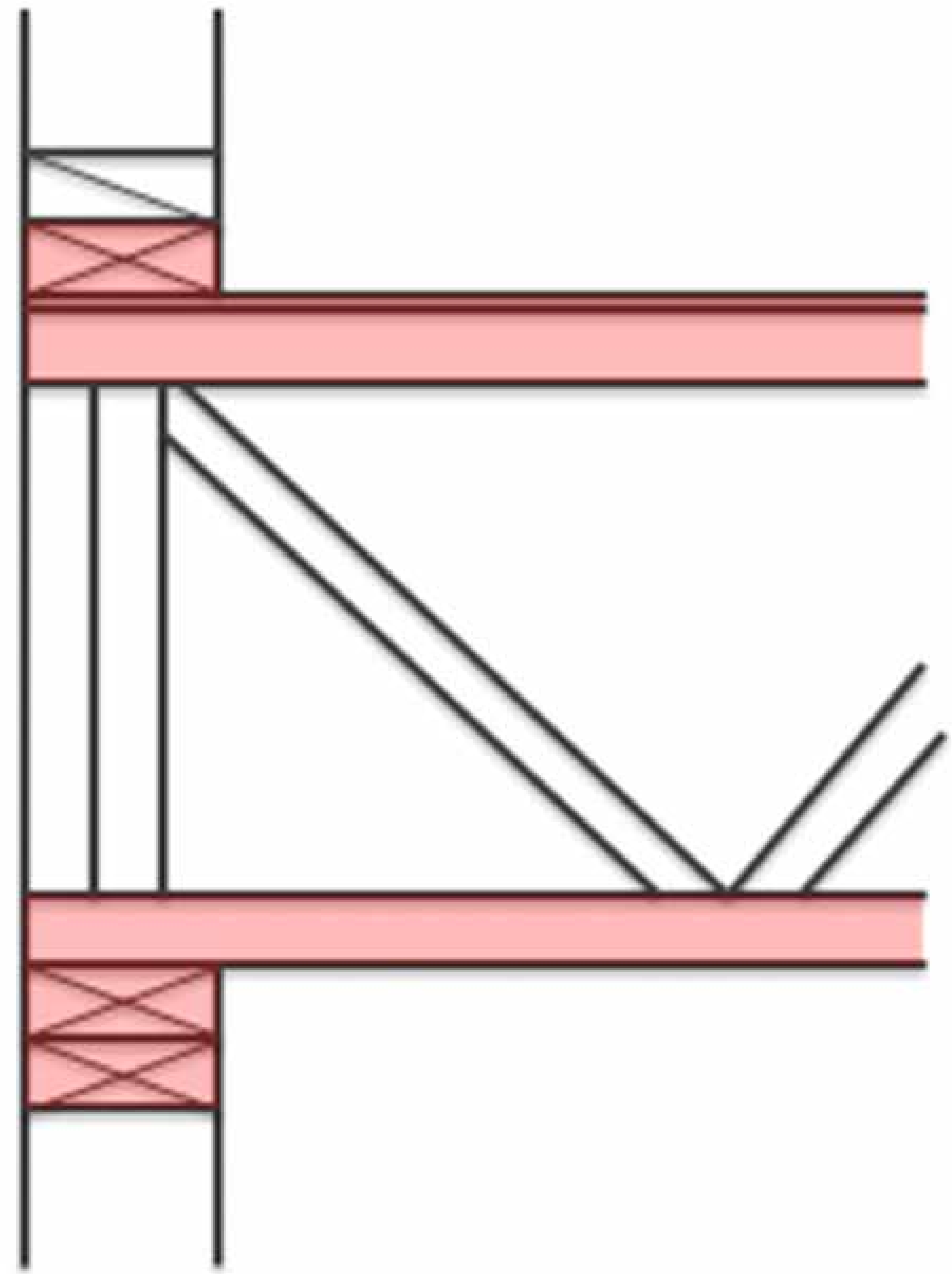
- Incorporates floor framing hanging from top plates
- Floor framing/rim joist doesn't contribute to shrinkage

Non-standard stud lengths and increased hardware requirements should be considered



Shrinkage Calculations – Cross Grain Wood

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



Differential Movement – Veneer Opening

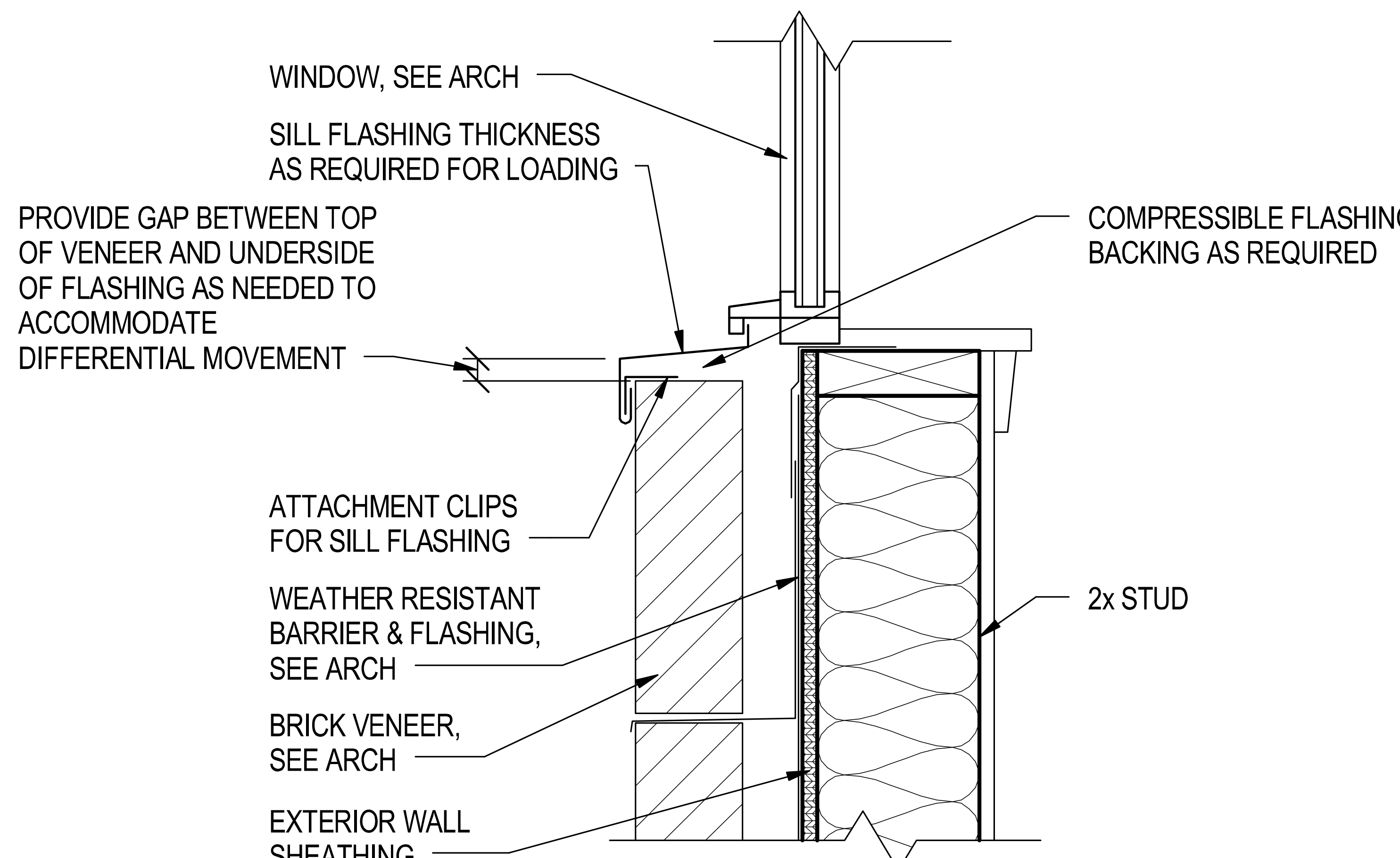


Image: Schaefer

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.

Stacked Bearing Wall Design

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

Bearing Wall Design

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: Fasten Master

Credit: WoodWorks

Wall Blocking Requirements

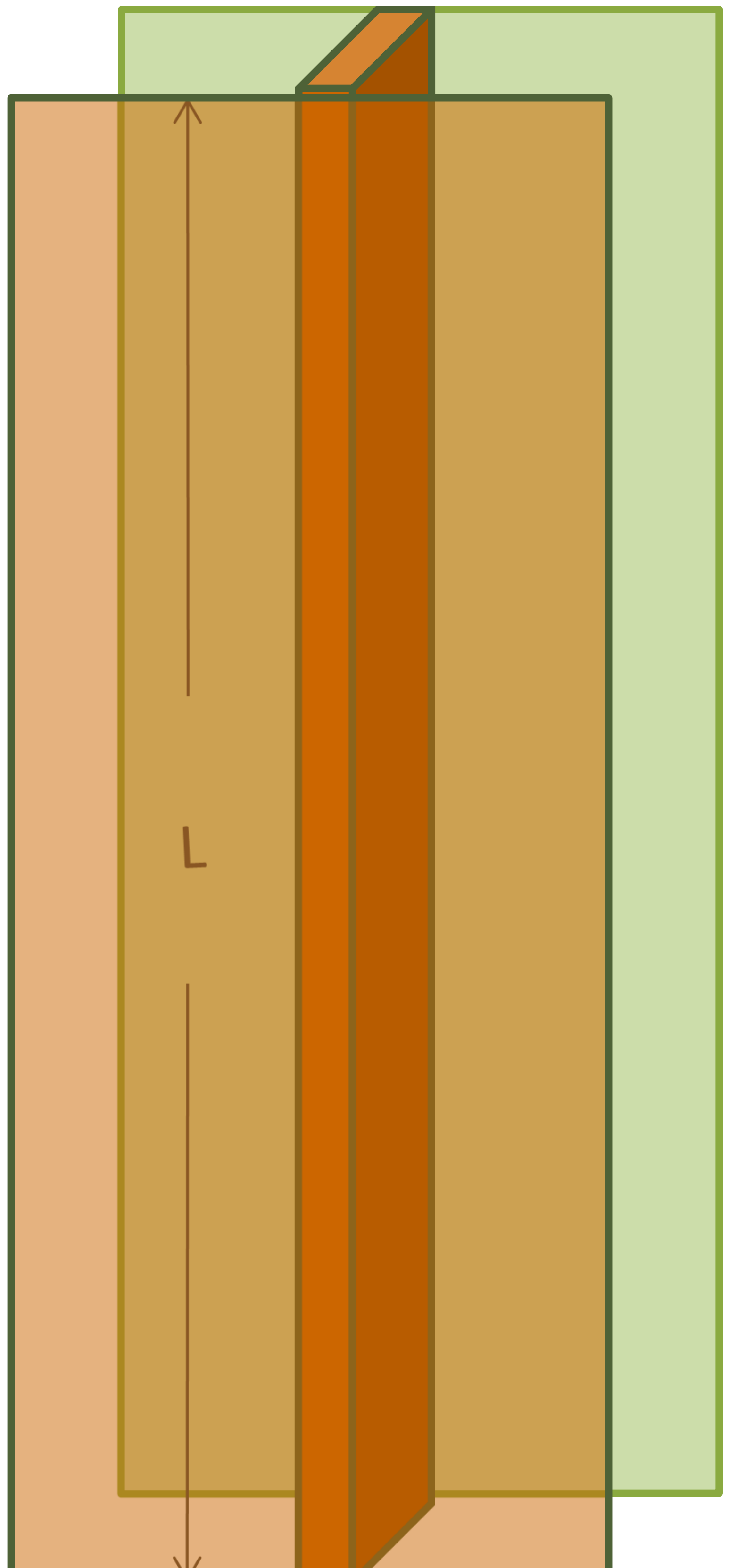
Wall Blocking Requirements

- 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.

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

- 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?

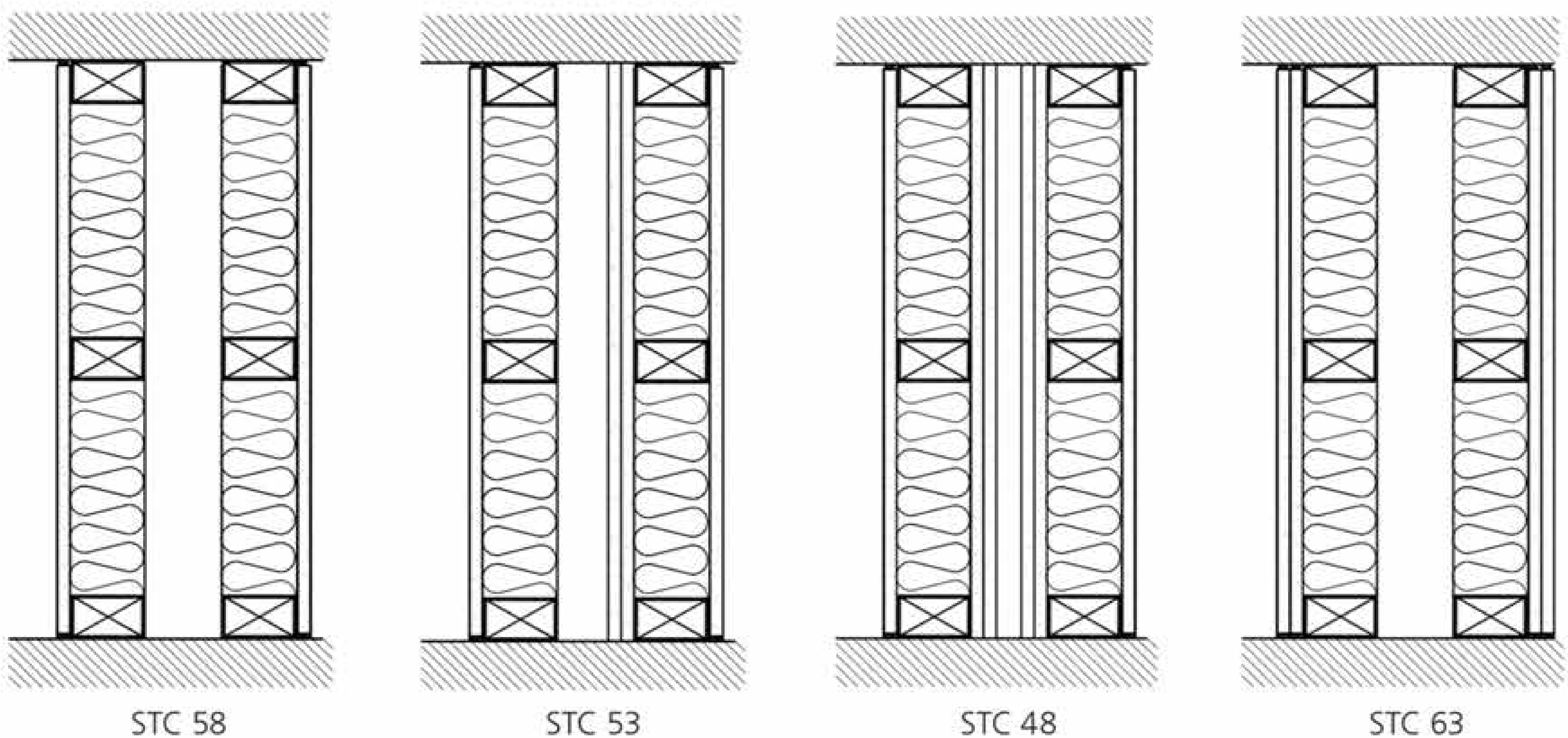
Yes, but placement is very important!


Credit: WoodWorks

Acoustical Design

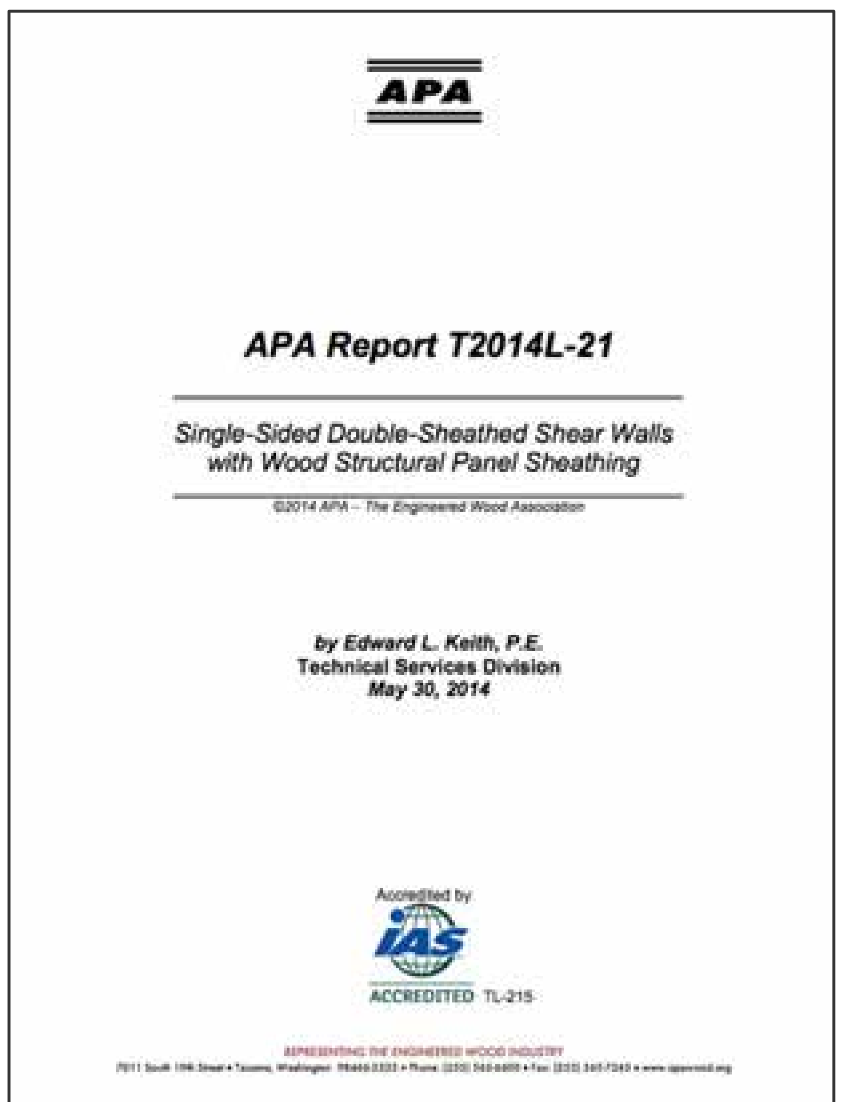
FIGURE 6

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

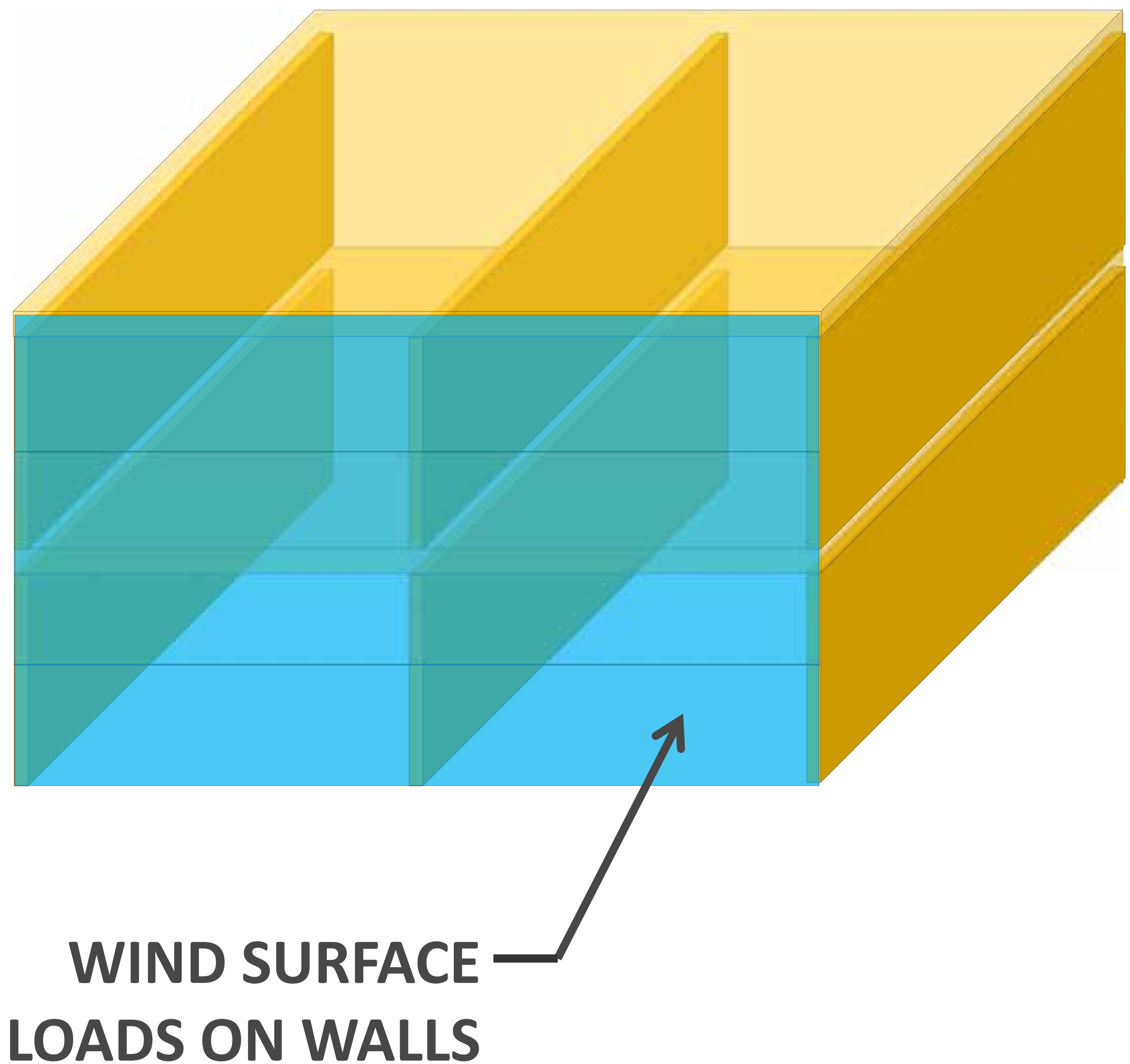
REPRESENTING THE ENGINEERED WOOD INDUSTRY

Acoustical Design

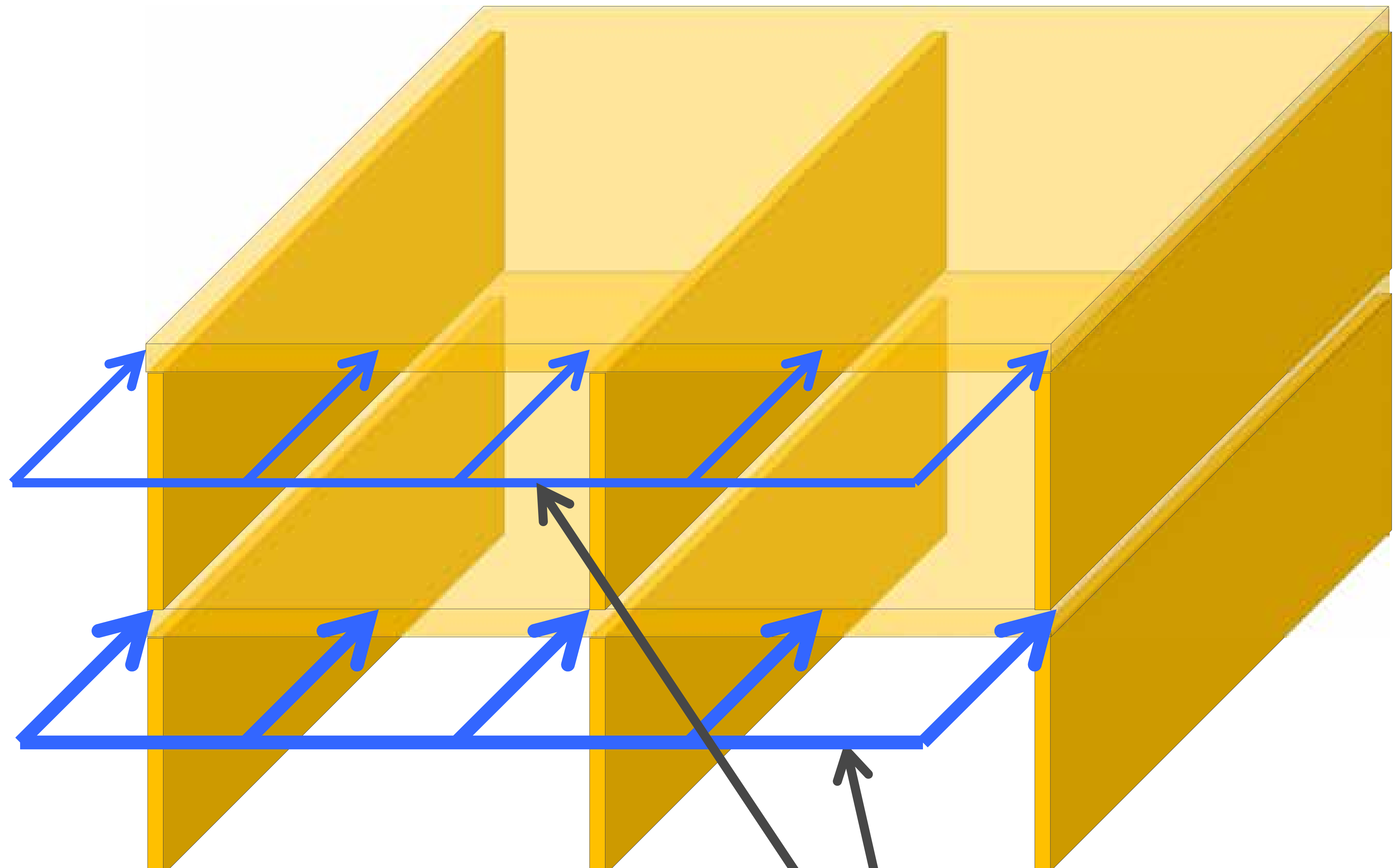
- Staggered stud wall condition:
- Blocking bridges finish on one side of wall to studs on opposite side, defeats purpose.
- Solution: use flat blocking in wall (wide face against WSP)

Lateral Design Topics

Multi-Story Wind Load Design

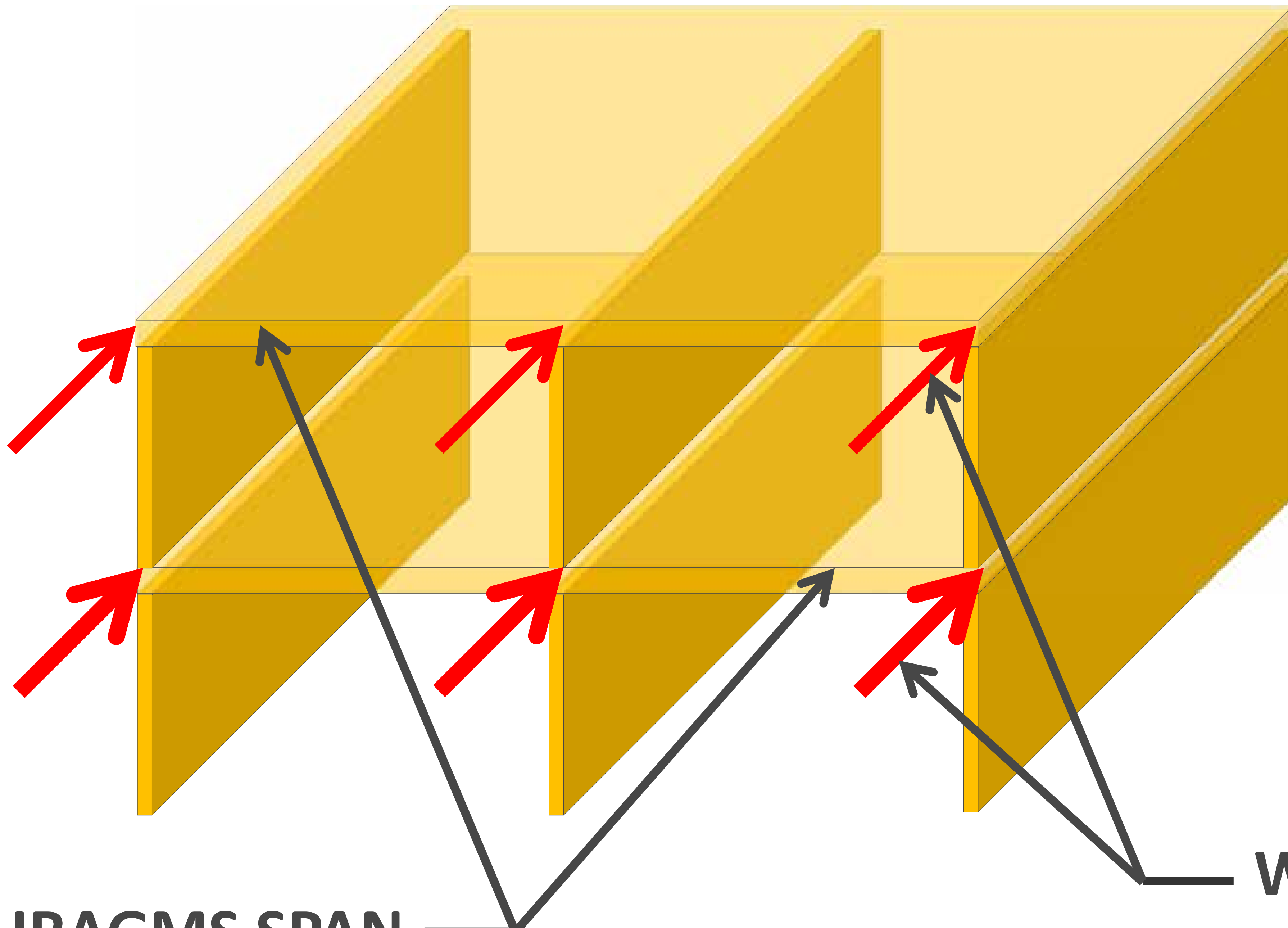


Multi-Story Wind Load Design

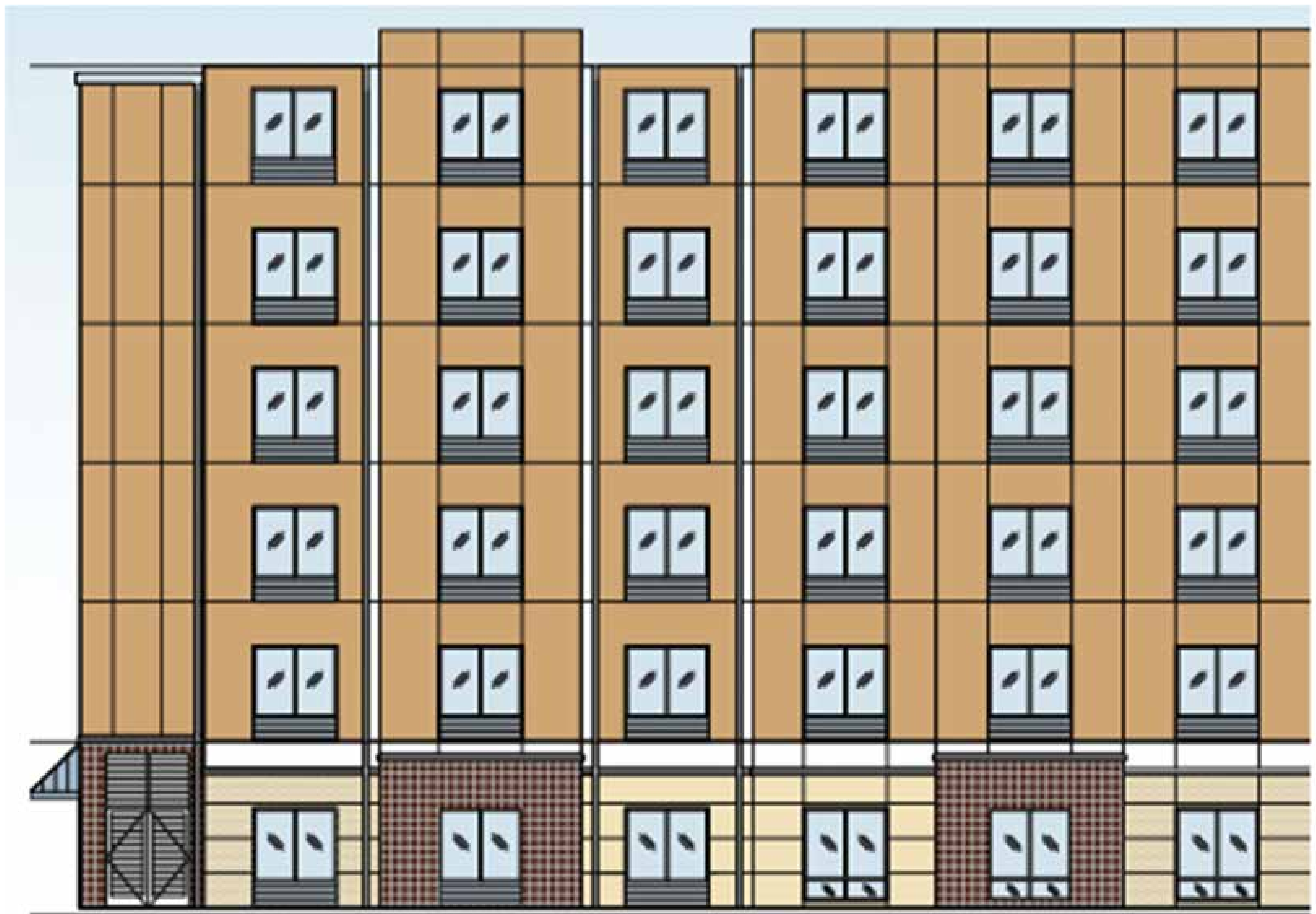


WIND INTO DIAPHRAGMS AS
UNIFORM LINEAR LOADS

Multi-Story Wind Load Design



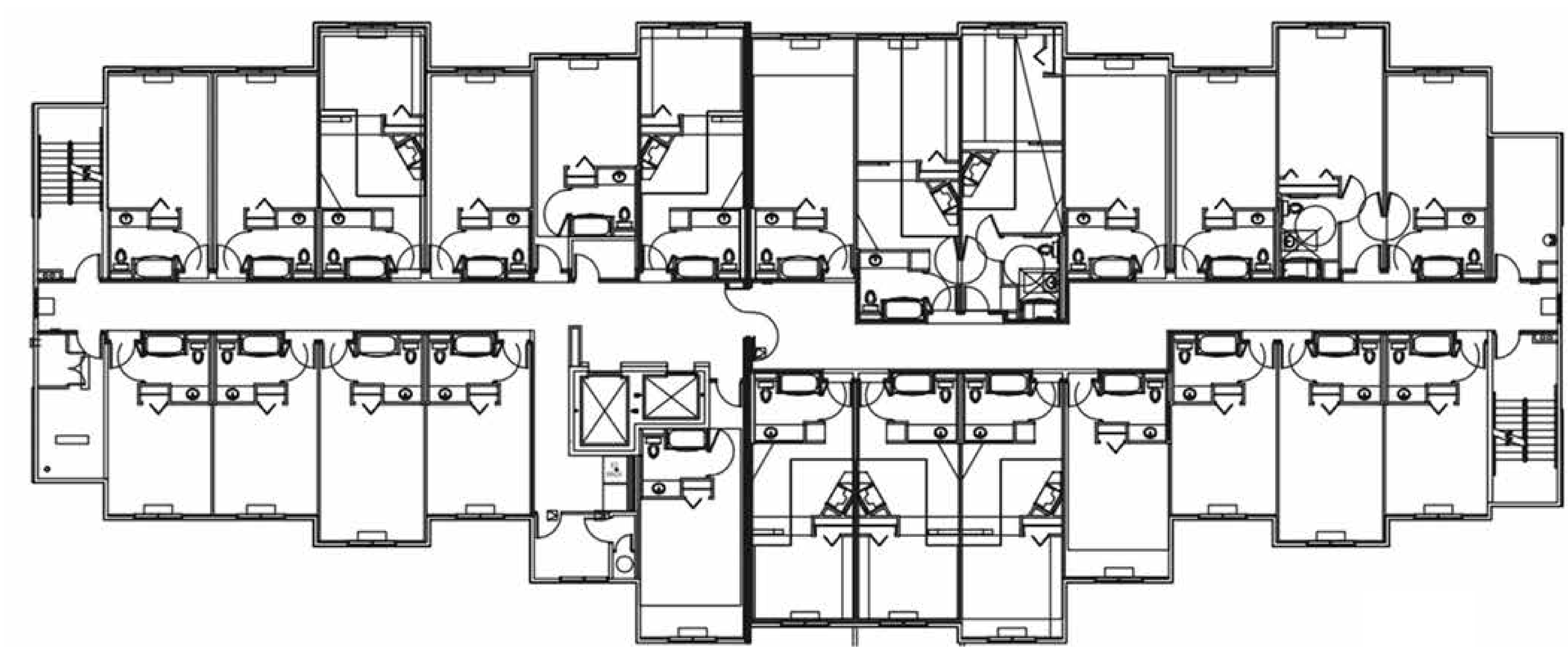
Multi-Story Wind Design



Elevation

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

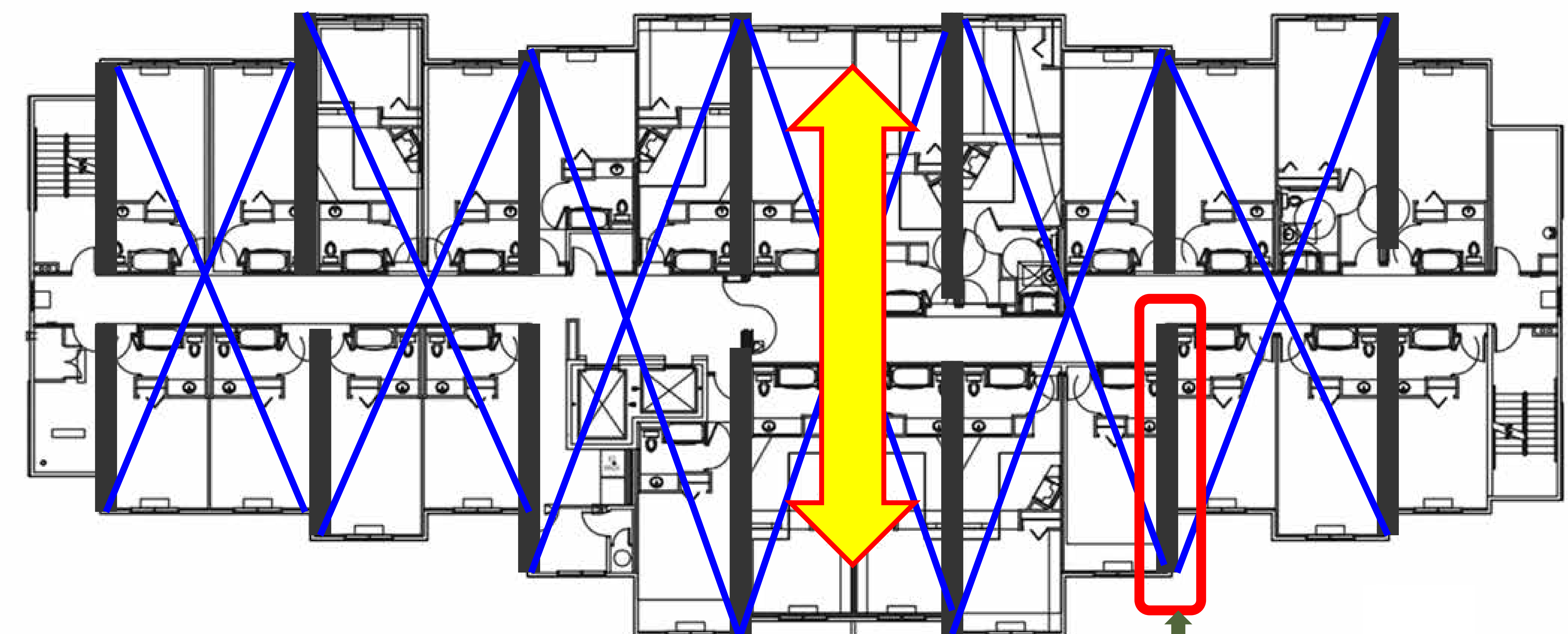
Multi-Story Wind Design



Floor Plan

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

Multi-Story Wind Design

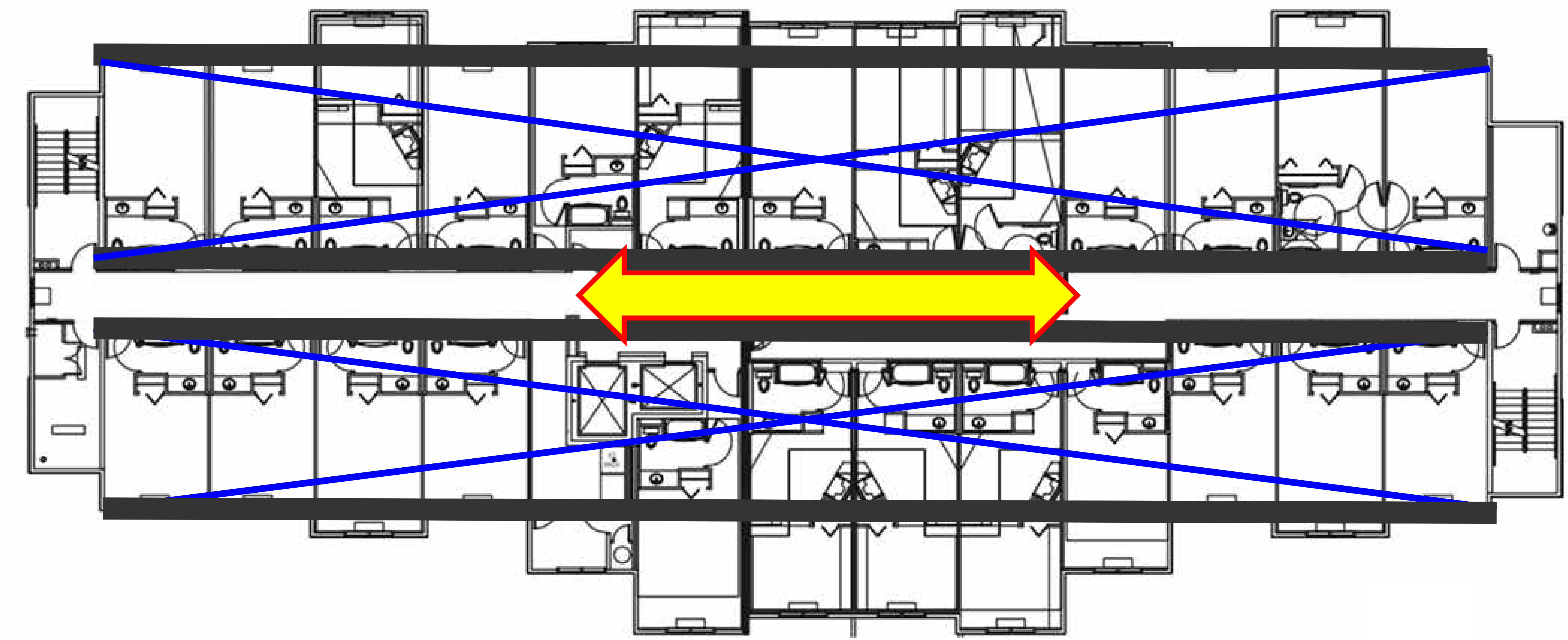


Shearwall design we'll look at

Shearwall Layout

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

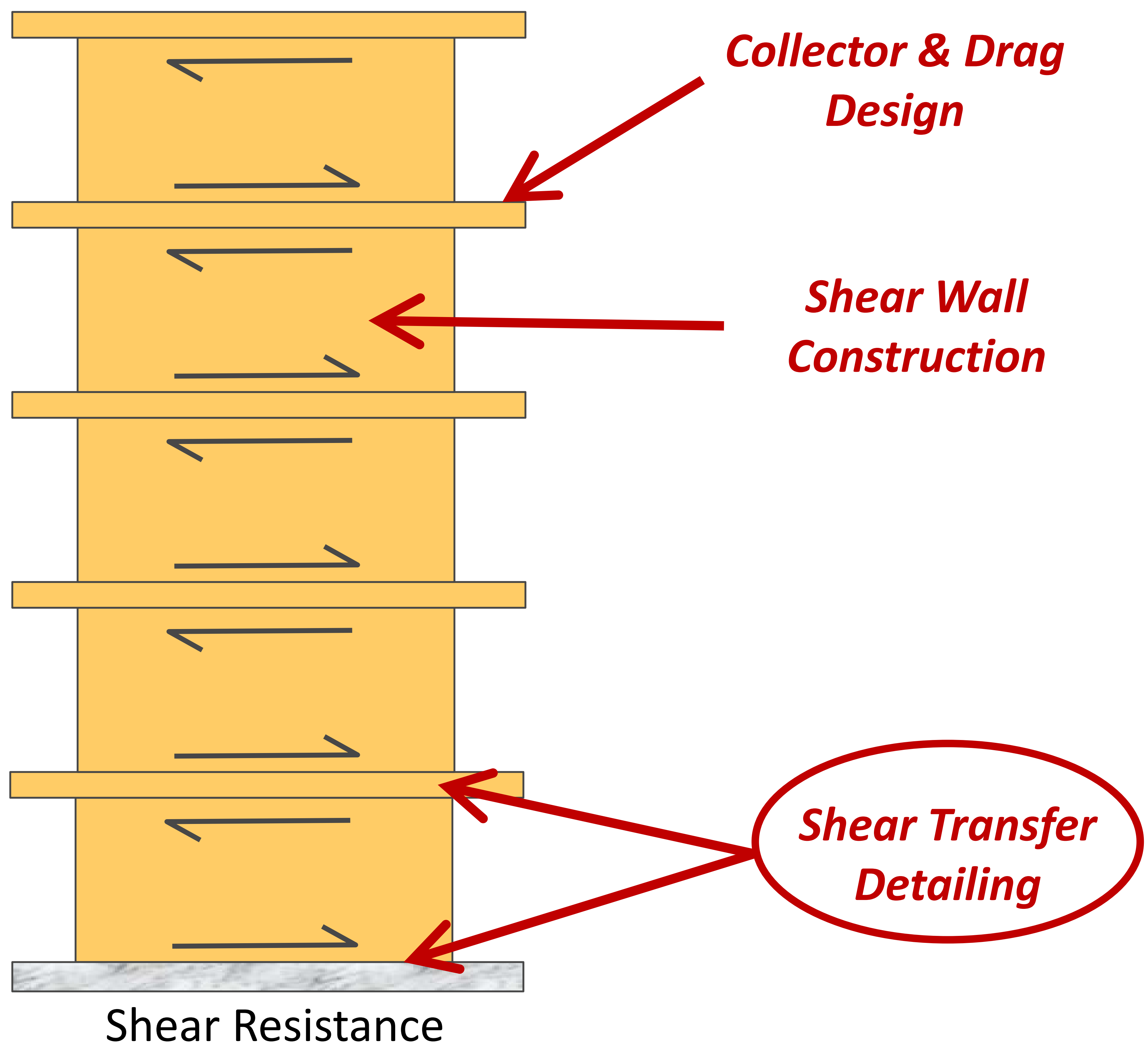
Multi-Story Wind Design



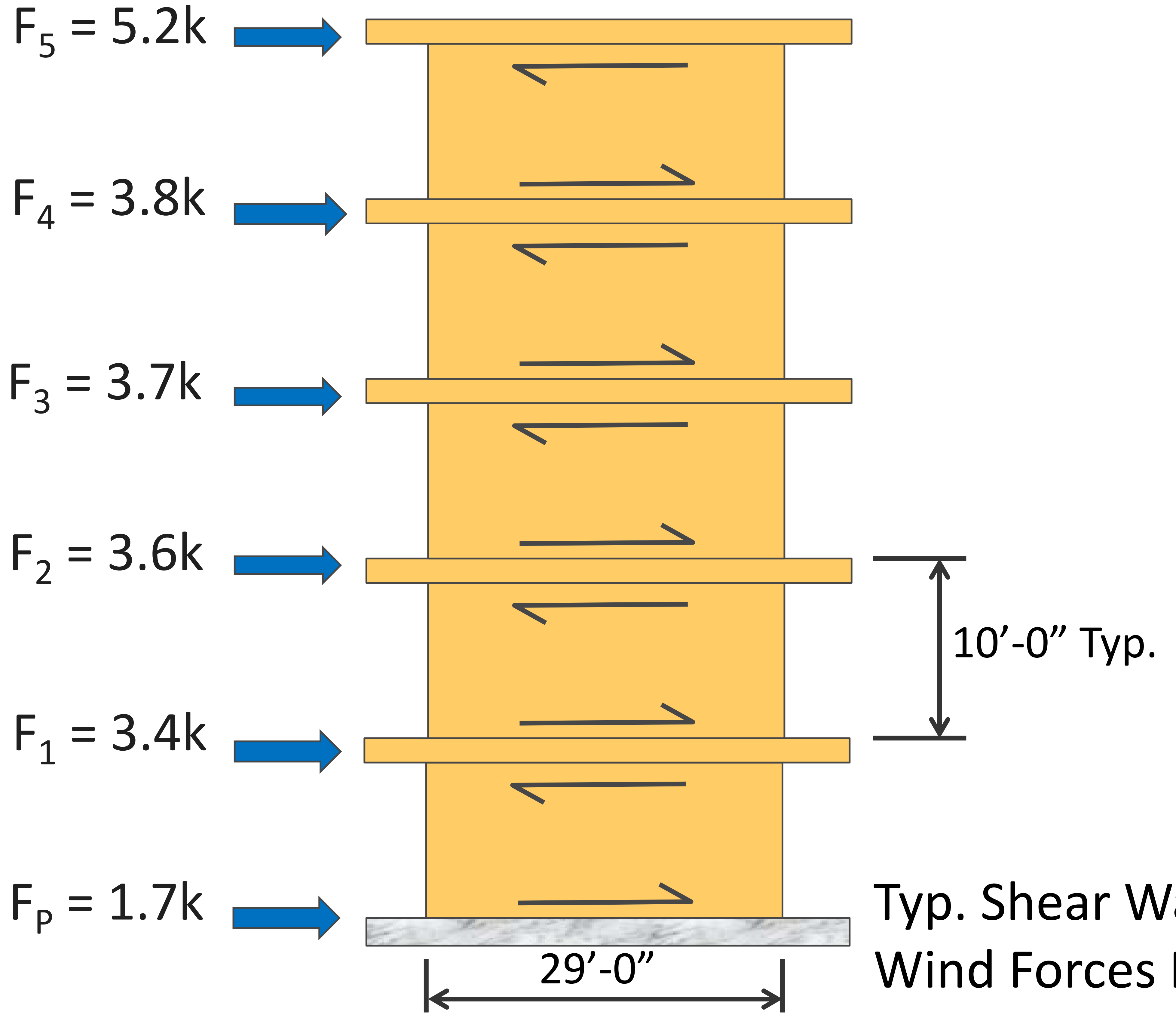
Shearwall Layout

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

Components of Shear Wall Design

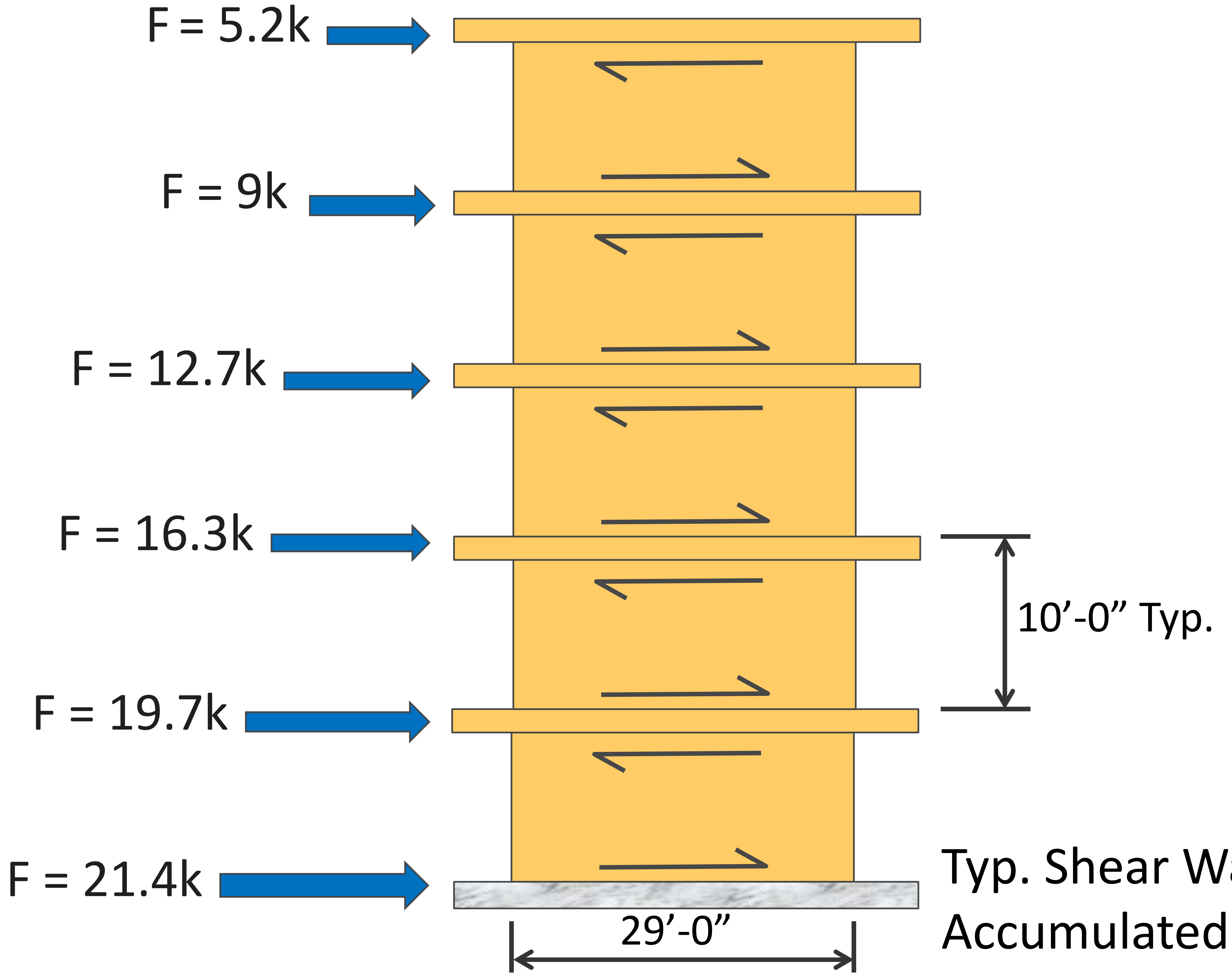


Components of Shear Wall Design



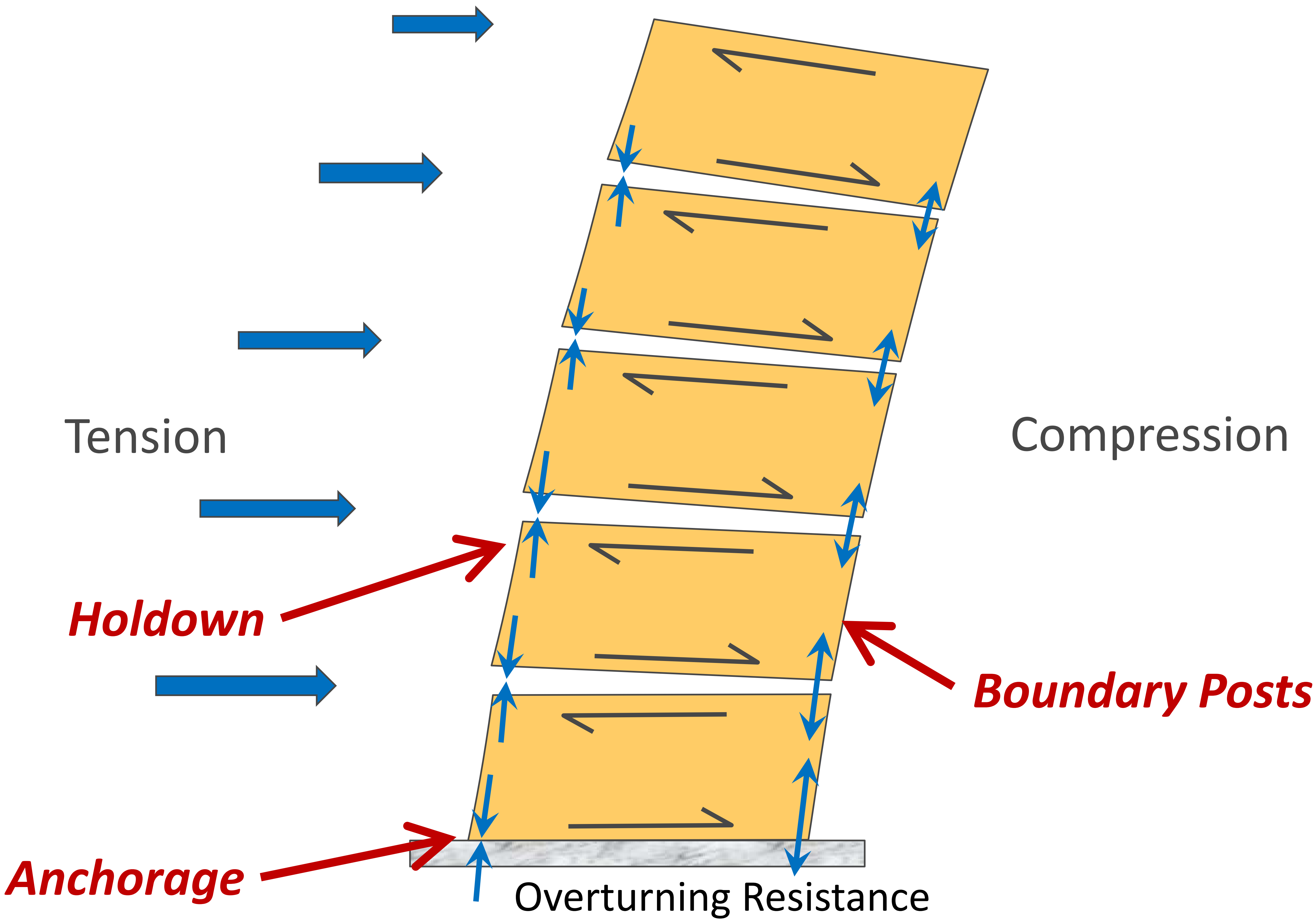
Typ. Shear Wall Elevation
Wind Forces Per Story

Components of Shear Wall Design

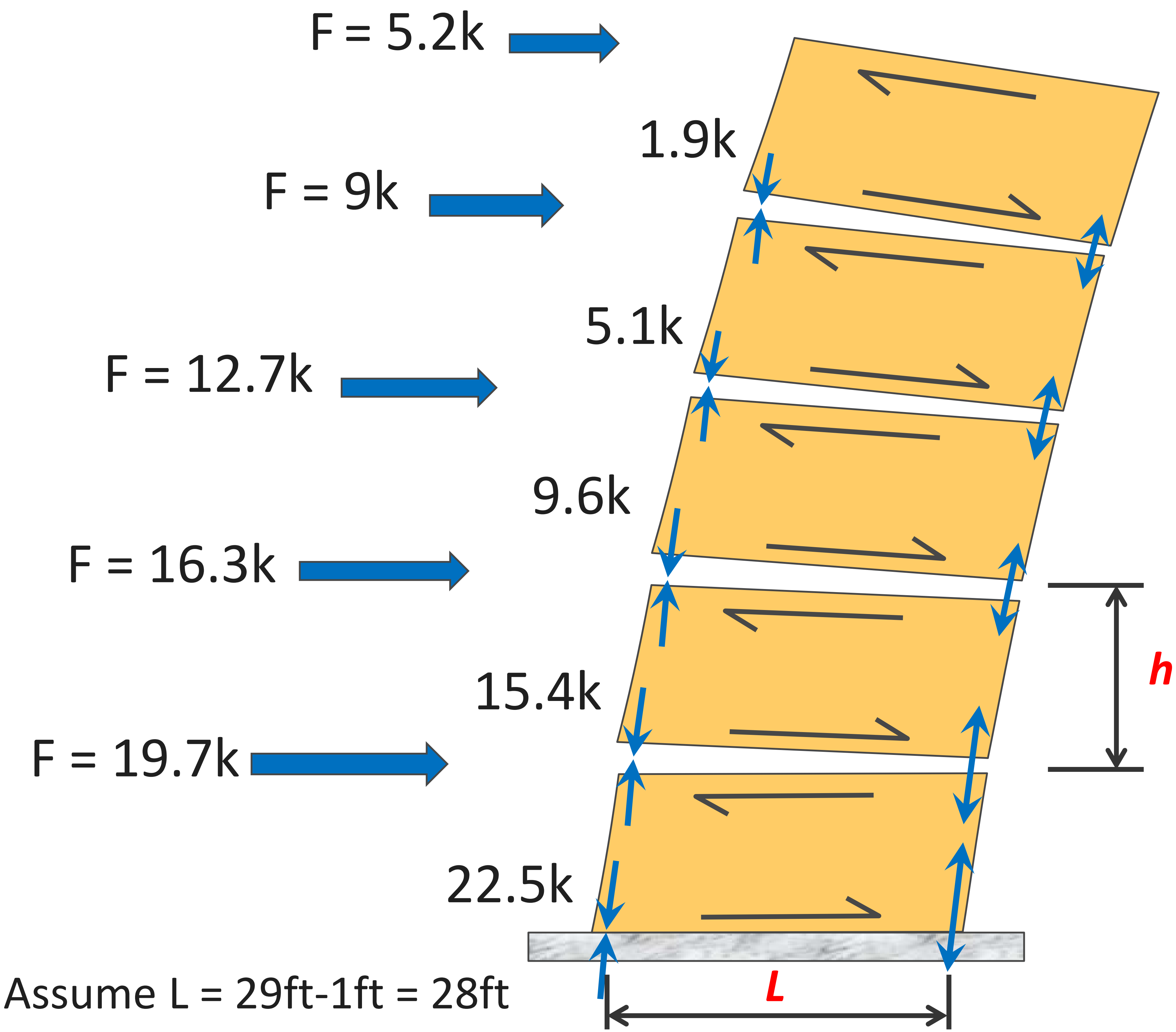


Typ. Shear Wall Elevation
Accumulated Wind Forces

Components of Shear Wall Design



Overturning Force Calculation



Assume L = 29ft-1ft = 28ft

T = C = F*h/L

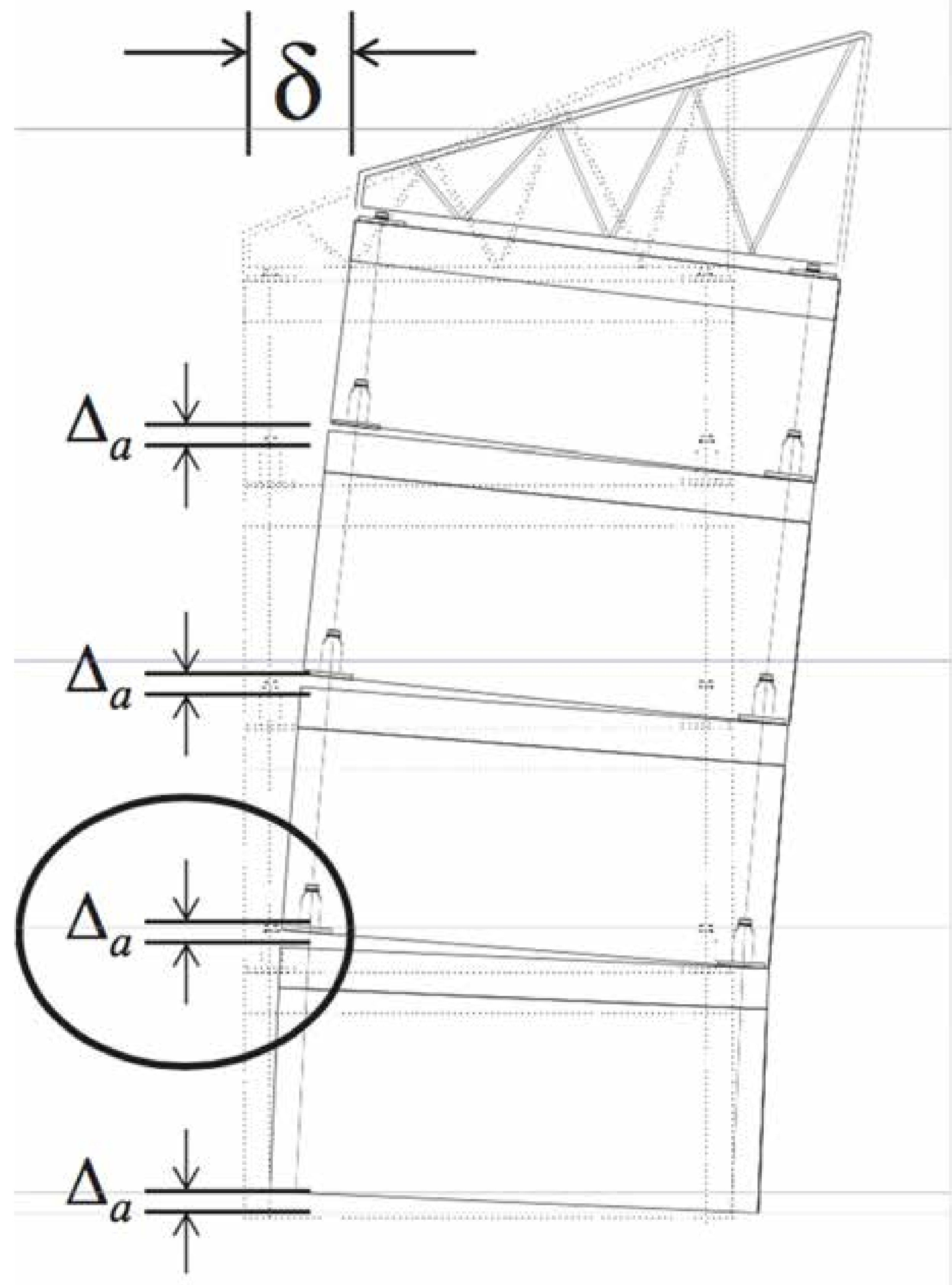
T & C are cumulative at lower stories

L is moment arm, not entire wall length

Shearwall Deformation – System Stretch

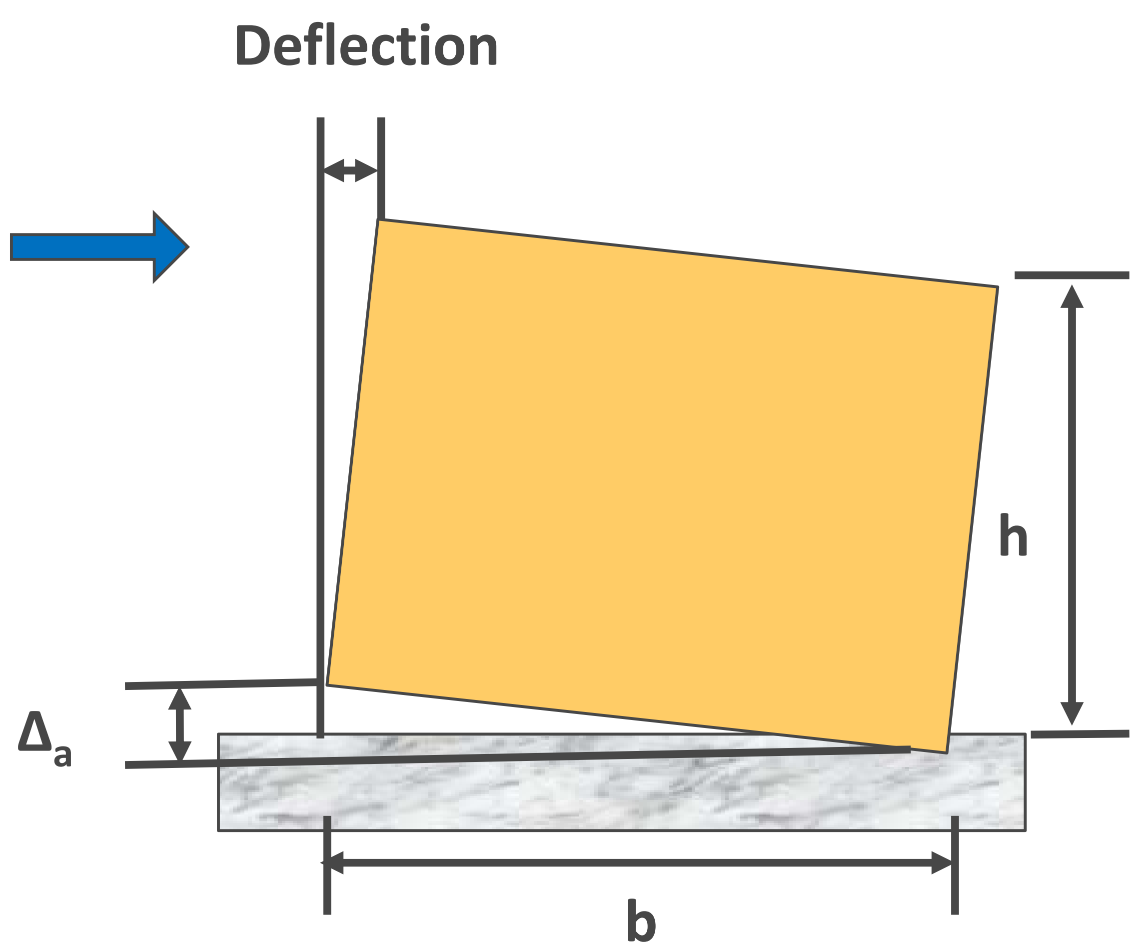
Total system stretch includes:

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



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

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

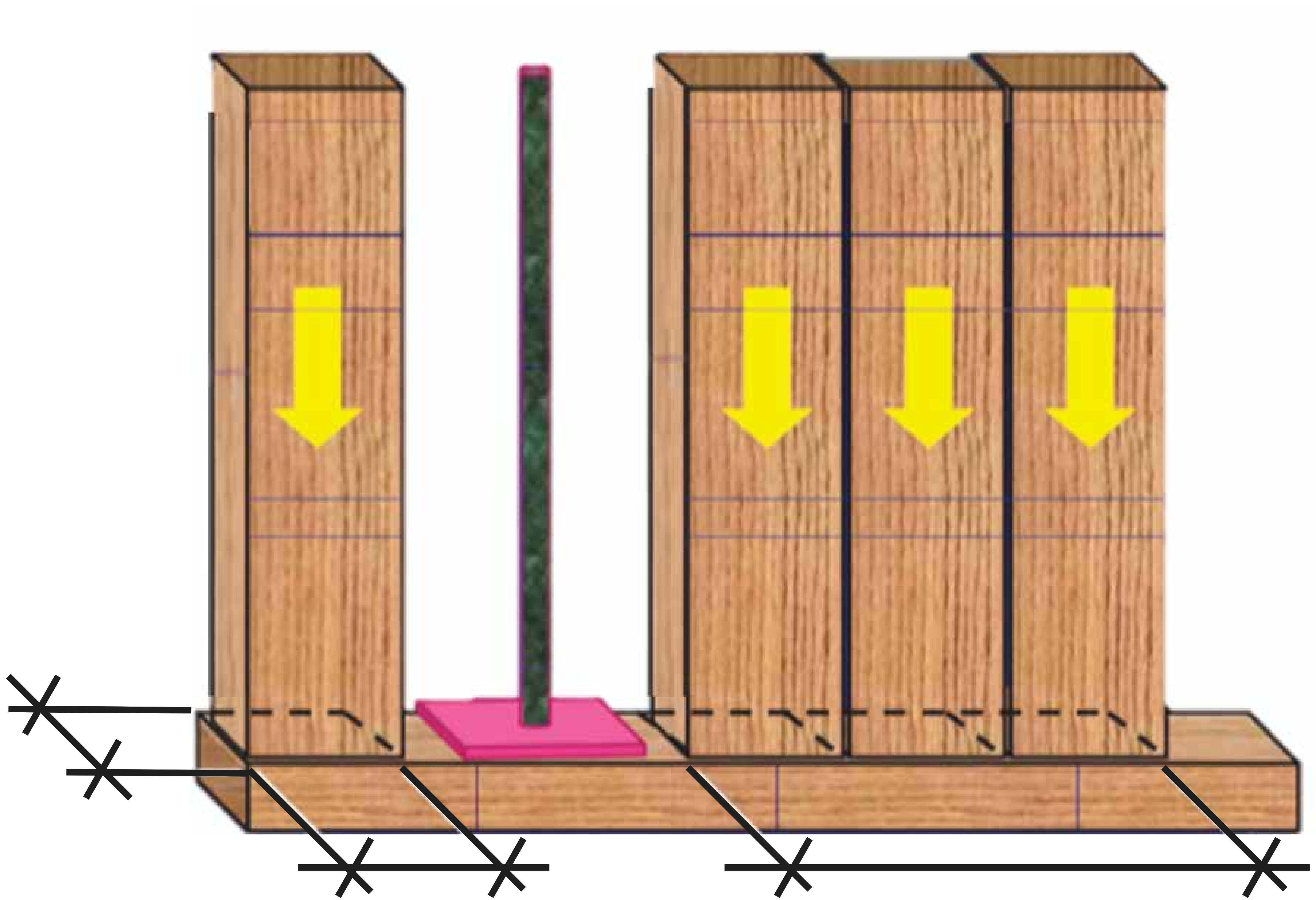
$$\delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{G_v t_v} + 0.75he_n + \boxed{\frac{h}{b}\Delta_a}$$

IBC 2000 to 2015 Eq. 23-2

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

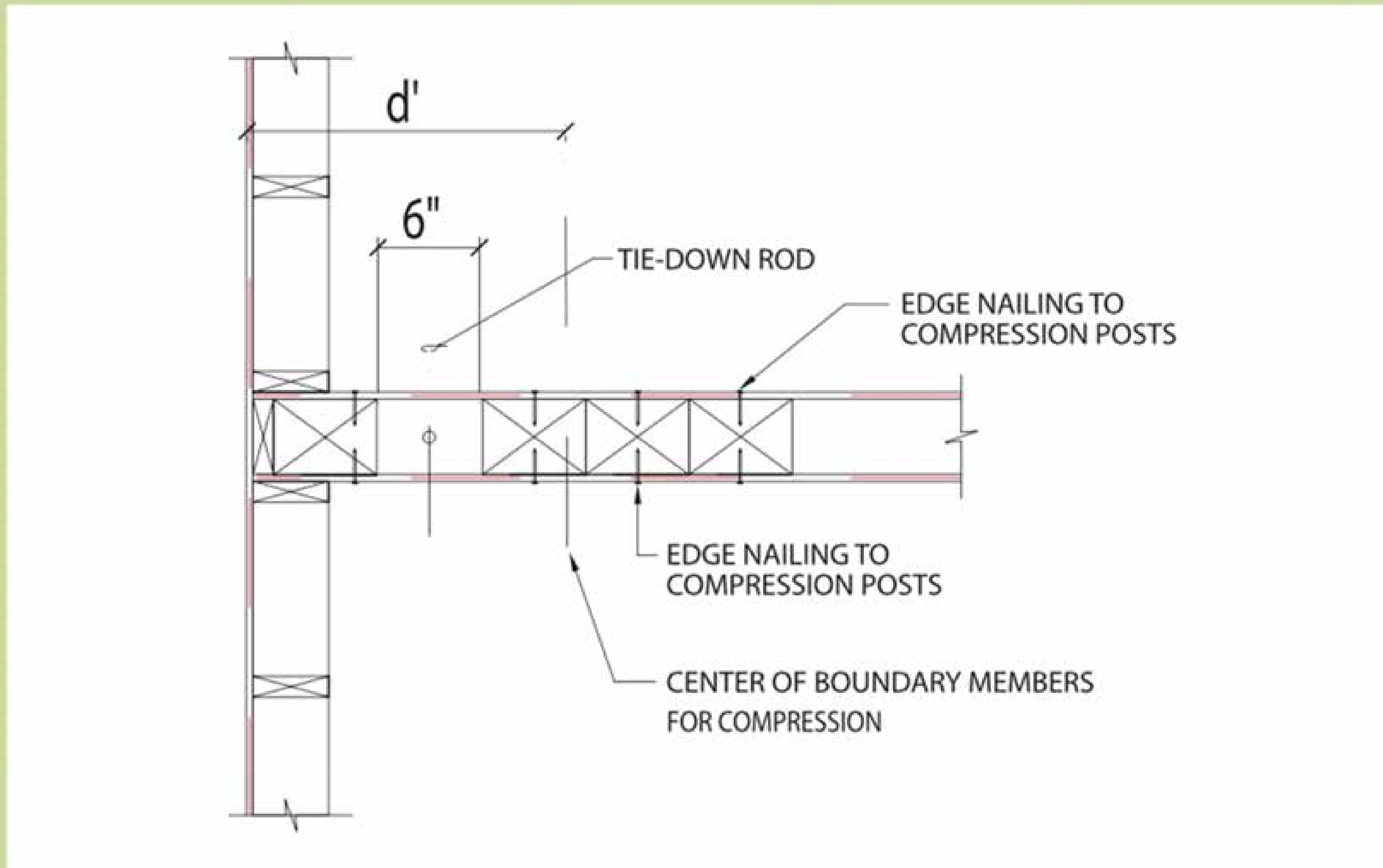
Rigid Body Rotation

Sole Plate Crushing



Increasing Compression Post Size

Figure 10. *Example Plan Section at Boundary Members*



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

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

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

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

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 $F_{c\perp}$)

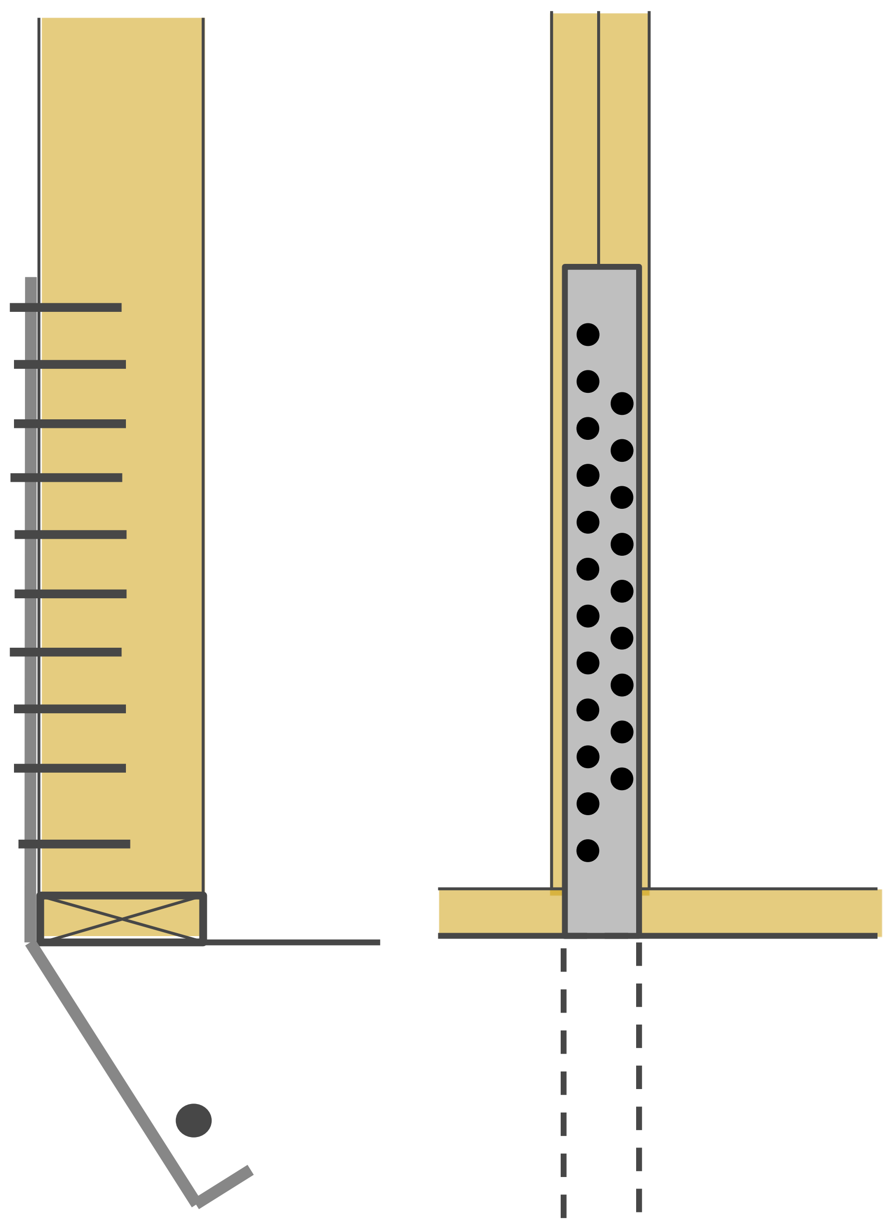
Compression Post Size & Sole Plate Crush

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

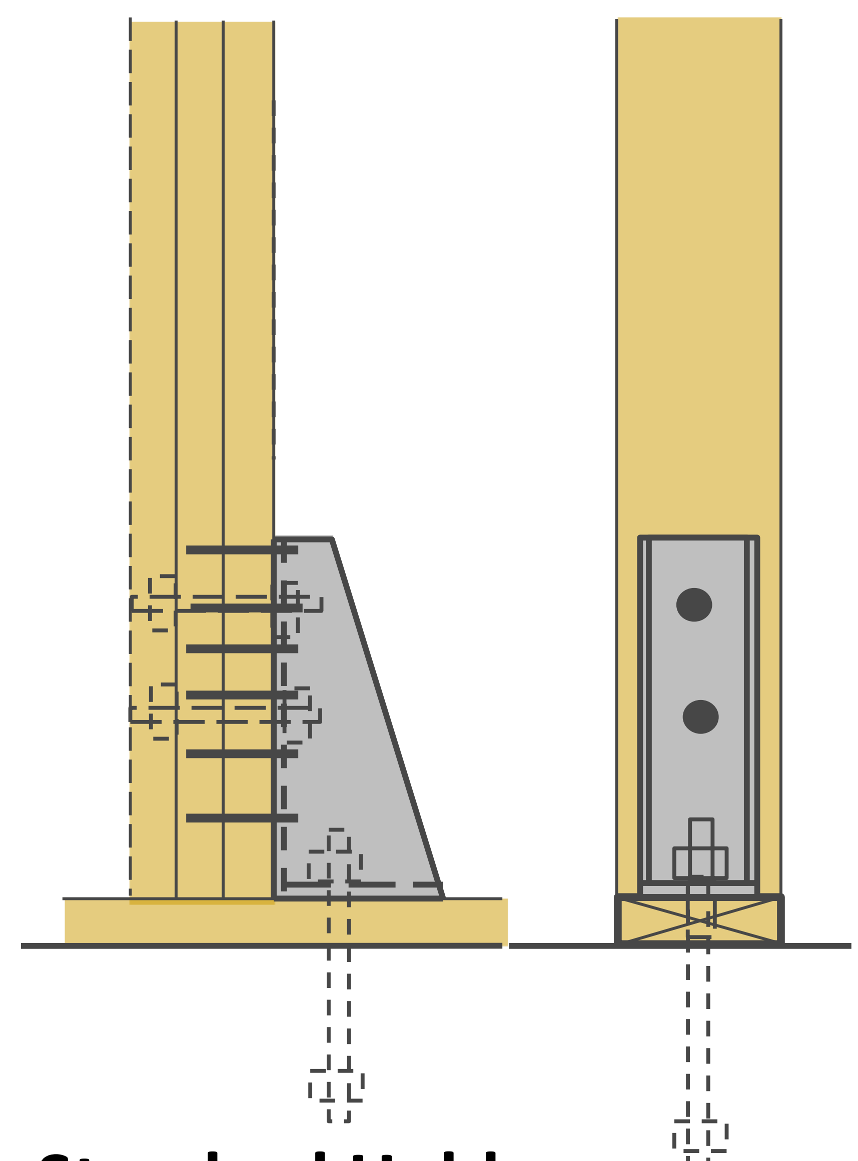
Floor 1 use SYP #2 Sole Plate, F_{cperp} = 565 psi

Shear Wall Holdown Options



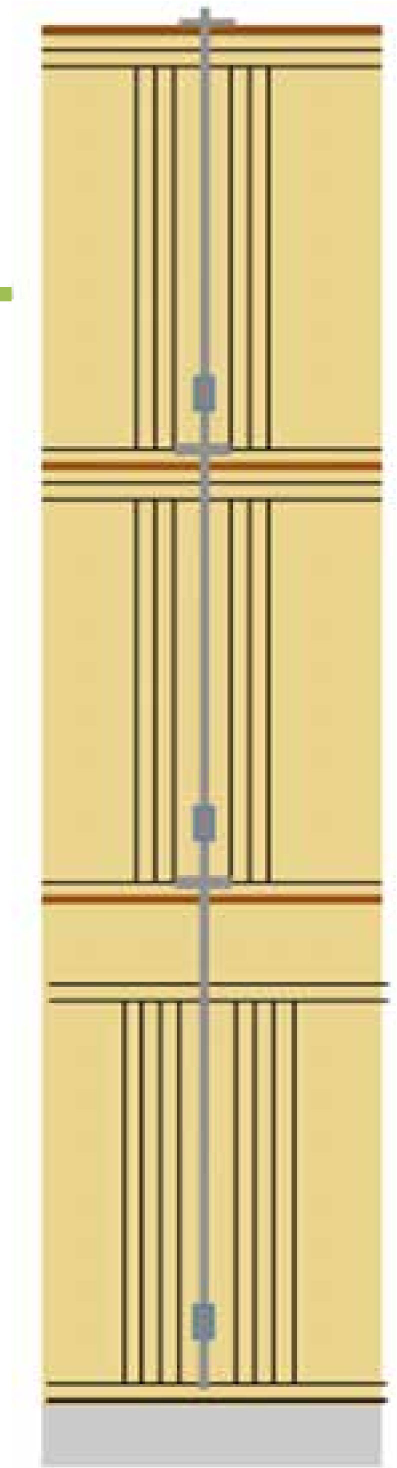
Strap Holdown Installation

6+ kip story to story capacities



Standard Holdown Installation

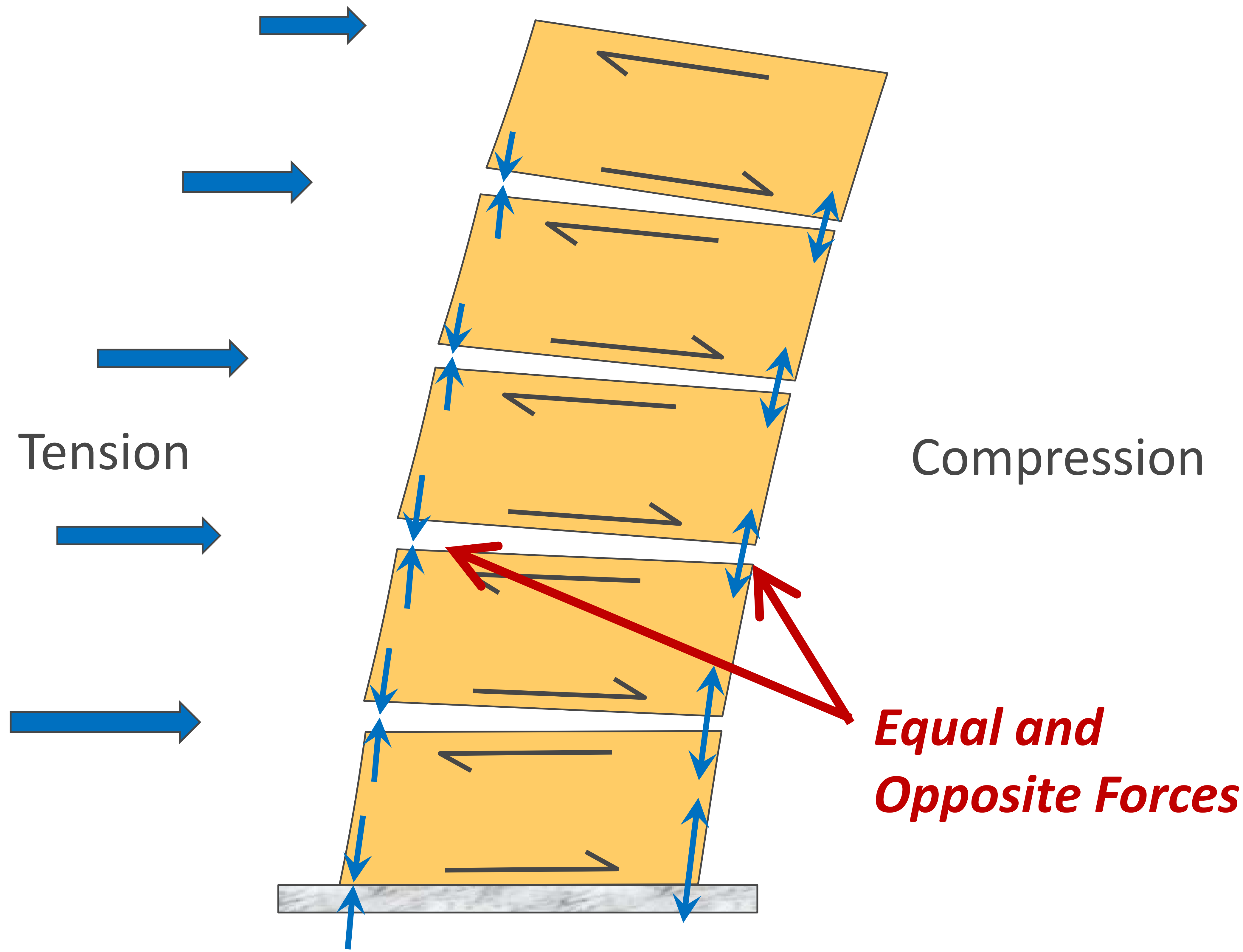
13+ kip capacities



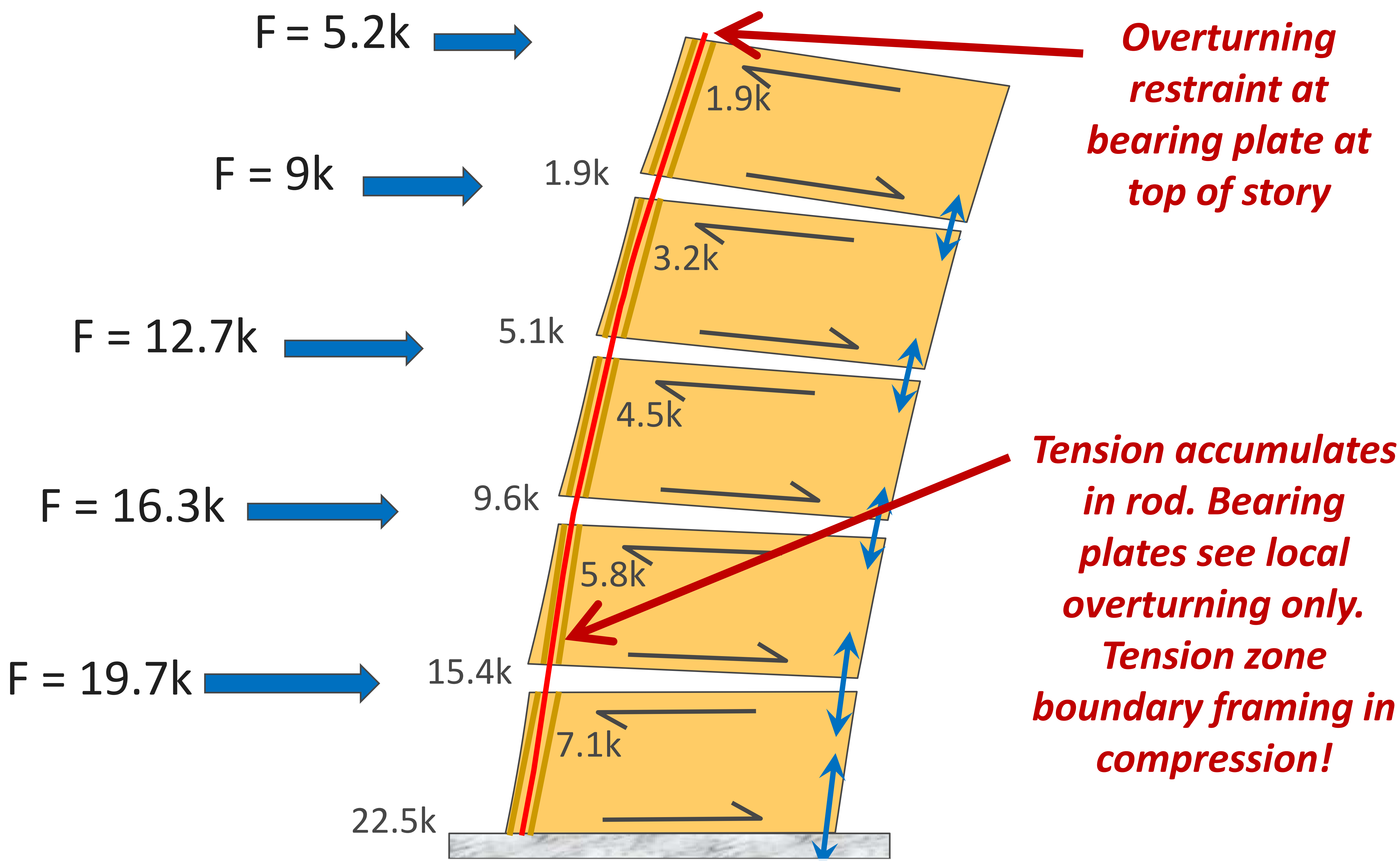
Continuous Rod Tiedown Systems

100+ kip capacities 20+ kips/level

Overturning Tension



Components of Shear Wall Design



Continuous Rod Holdown System

Threaded Rod Tie Down w/Take Up Device

Source: Strongtie

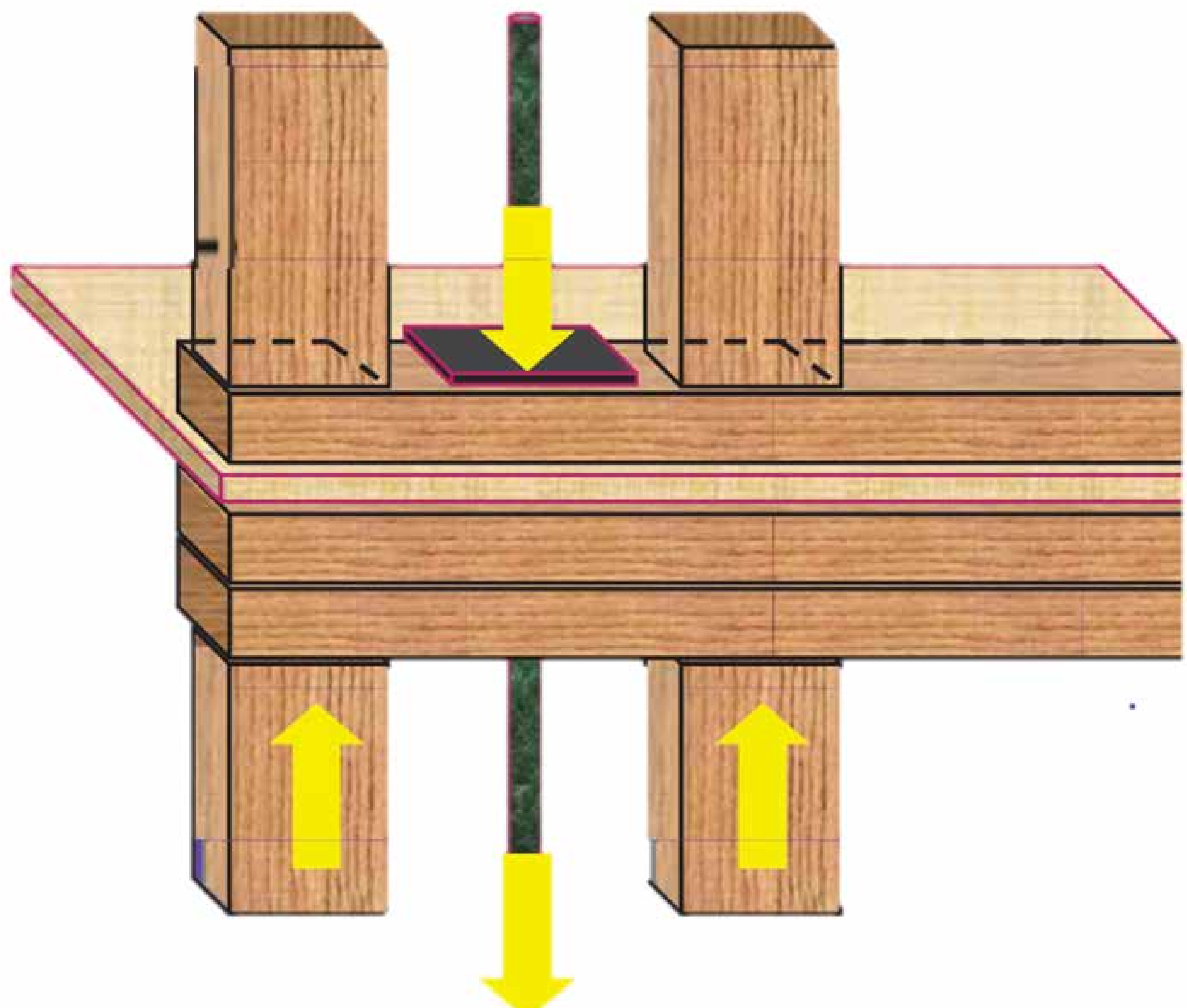
Source: hardyframe.com

Threaded Rod Tie Down w/o Take Up Device

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

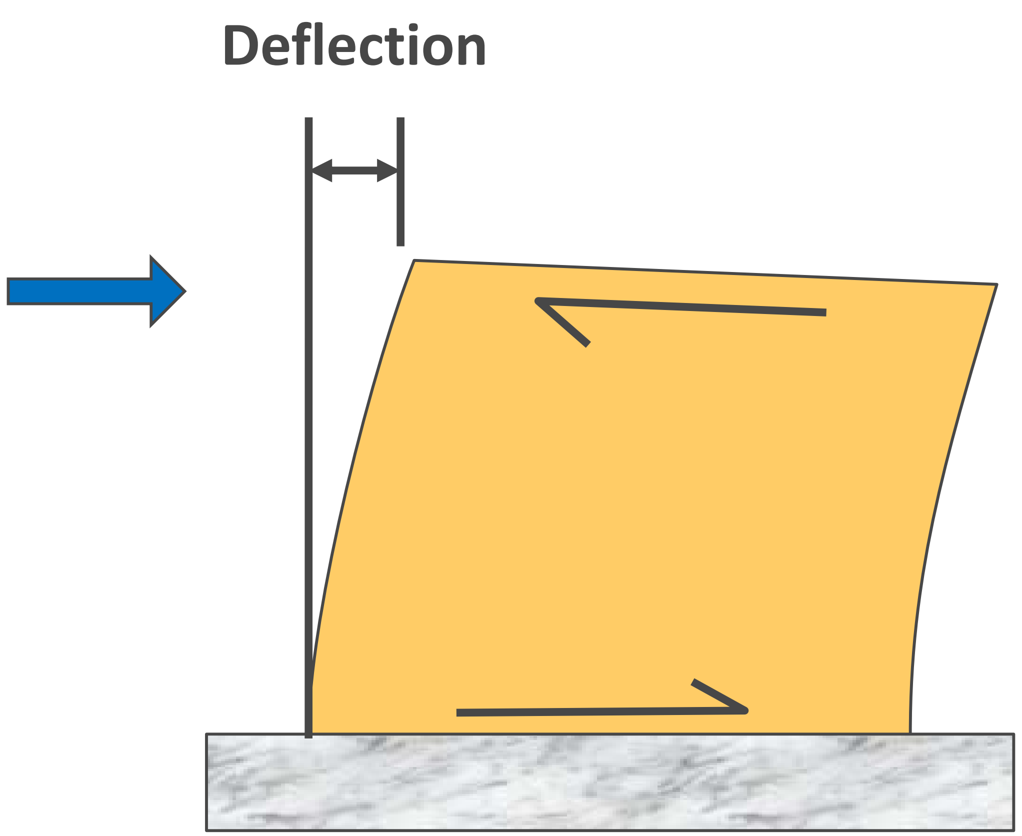
Level	Bearing Plate					Bearing Load	Allow. Bearing Capacity	Bearing Plate Crush
	W	L	T	Hole Area	A_{brng}			
5th Floor	3 in	3.5 in	3/8”	0.25 in^2	10.25 in^2	1.9 k	4.4 k	0.012”
4th Floor	3 in	3.5 in	3/8”	0.518 in^2	9.98 in^2	3.2 k	4.2 k	0.022”
3rd Floor	3 in	5.5 in	1/2”	0.518 in^2	15.98 in^2	4.5 k	6.8 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”

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”

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

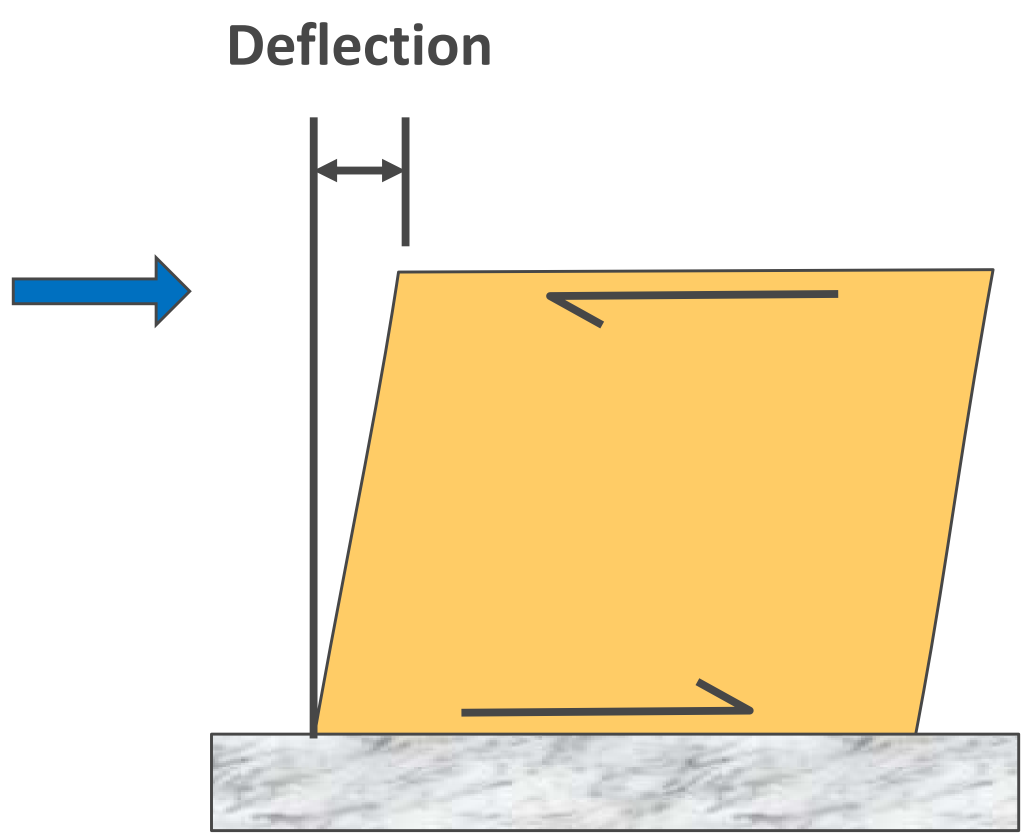
$$\delta_{SW} = \boxed{\frac{8vh^3}{EAb}} + \frac{vh}{G_v t_v} + 0.75he_n + \frac{h}{b}\Delta_a$$

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



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

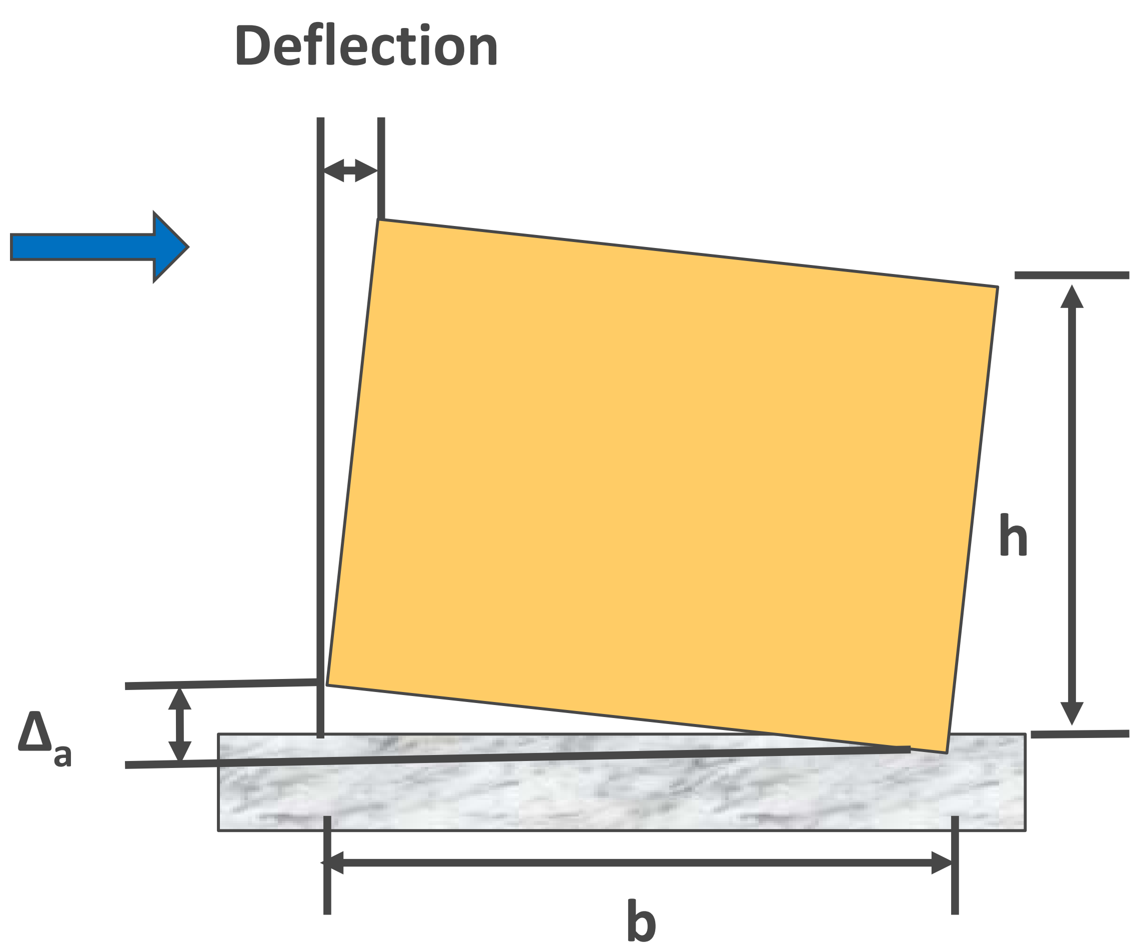
$$\Delta = \frac{8vh^3}{EAb} + \boxed{\frac{vh}{Gt} + 0.75he_n} + d_a\frac{h}{b}$$

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

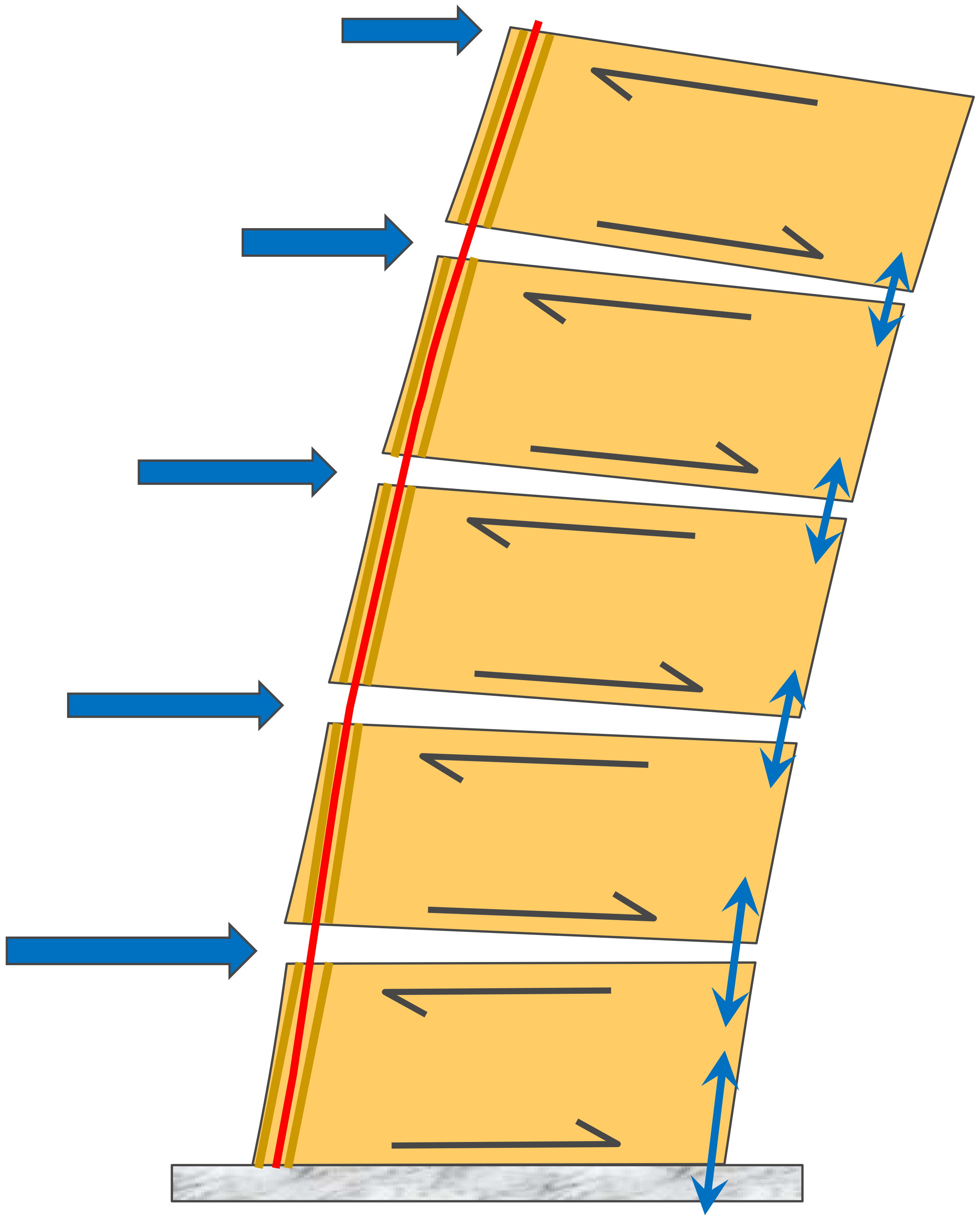
$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \boxed{d_a\frac{h}{b}}$$

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”

Shear Wall Deflections In Practice



SDPWS 2008 Eq 4.3-1

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

2D/3D Modeling of all the components of a shear wall system not practical in design.

Approaches used in Mid Rise include:

- **Spreadsheets**
- **Equivalent cantilever columns models in commercial analysis software**
- **Equivalent FEM wall models in commercial analysis software**
- **Proprietary calculations by system manufacturer**

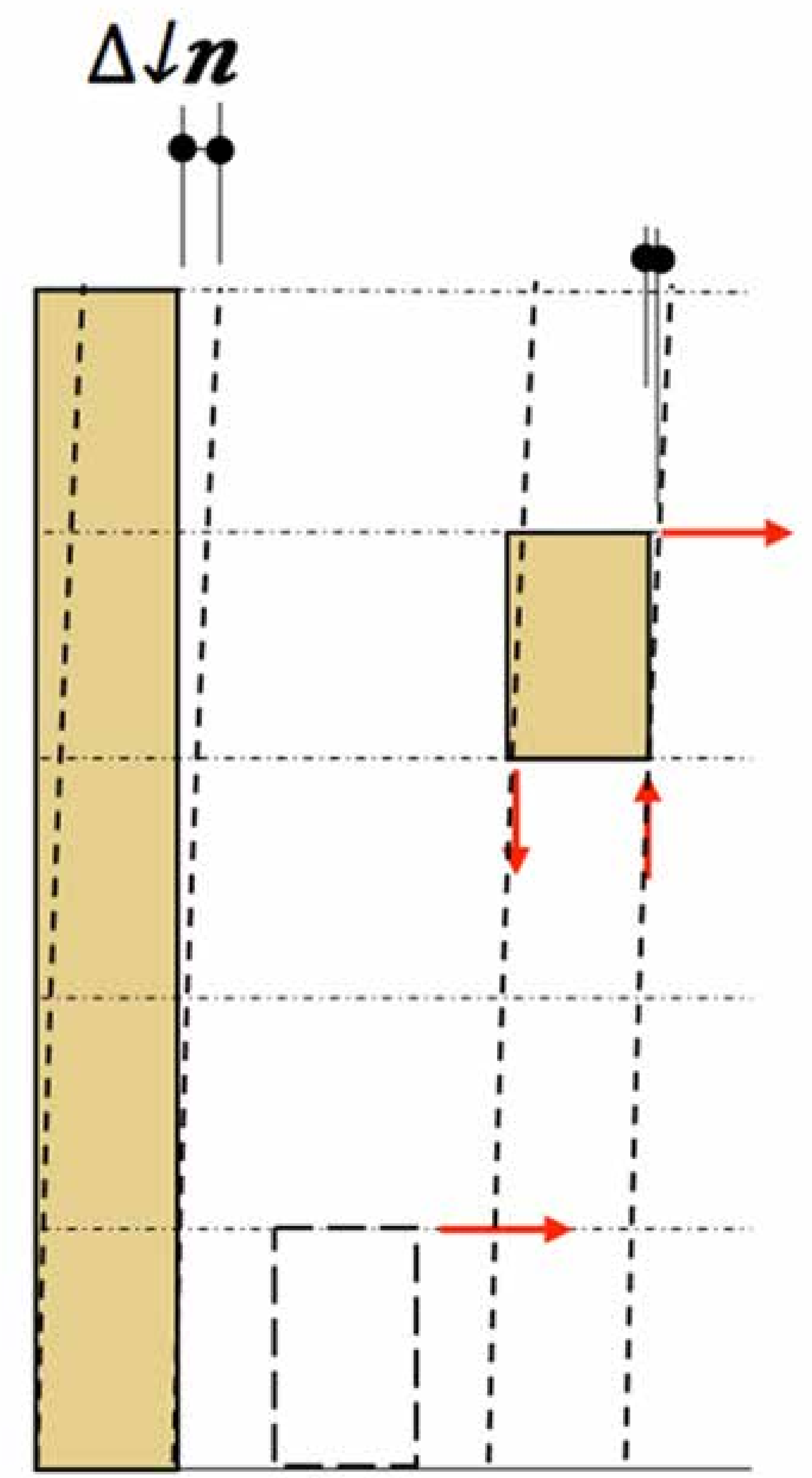
Shearwall Deflection Methods

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

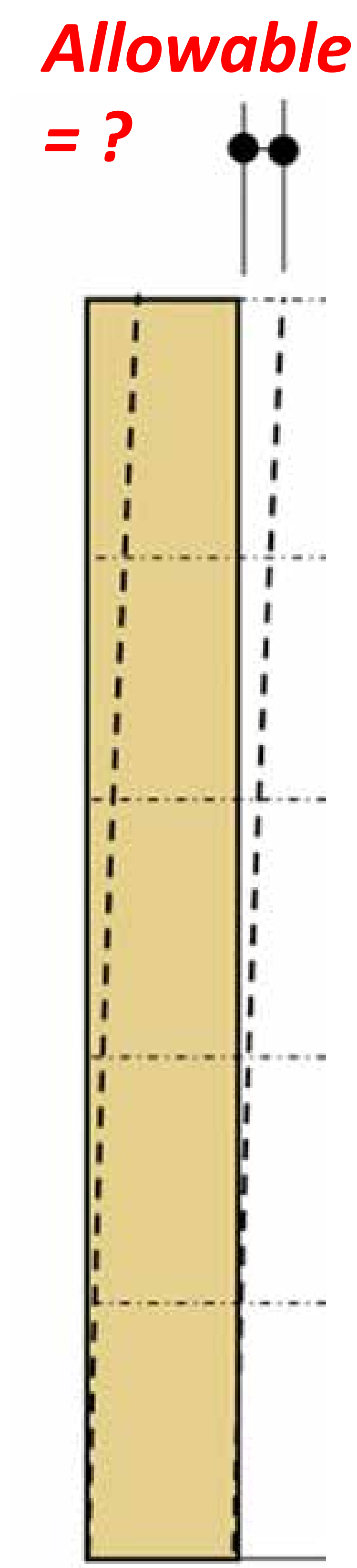
Serviceability check to minimize damage to cladding and nonstructural walls

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

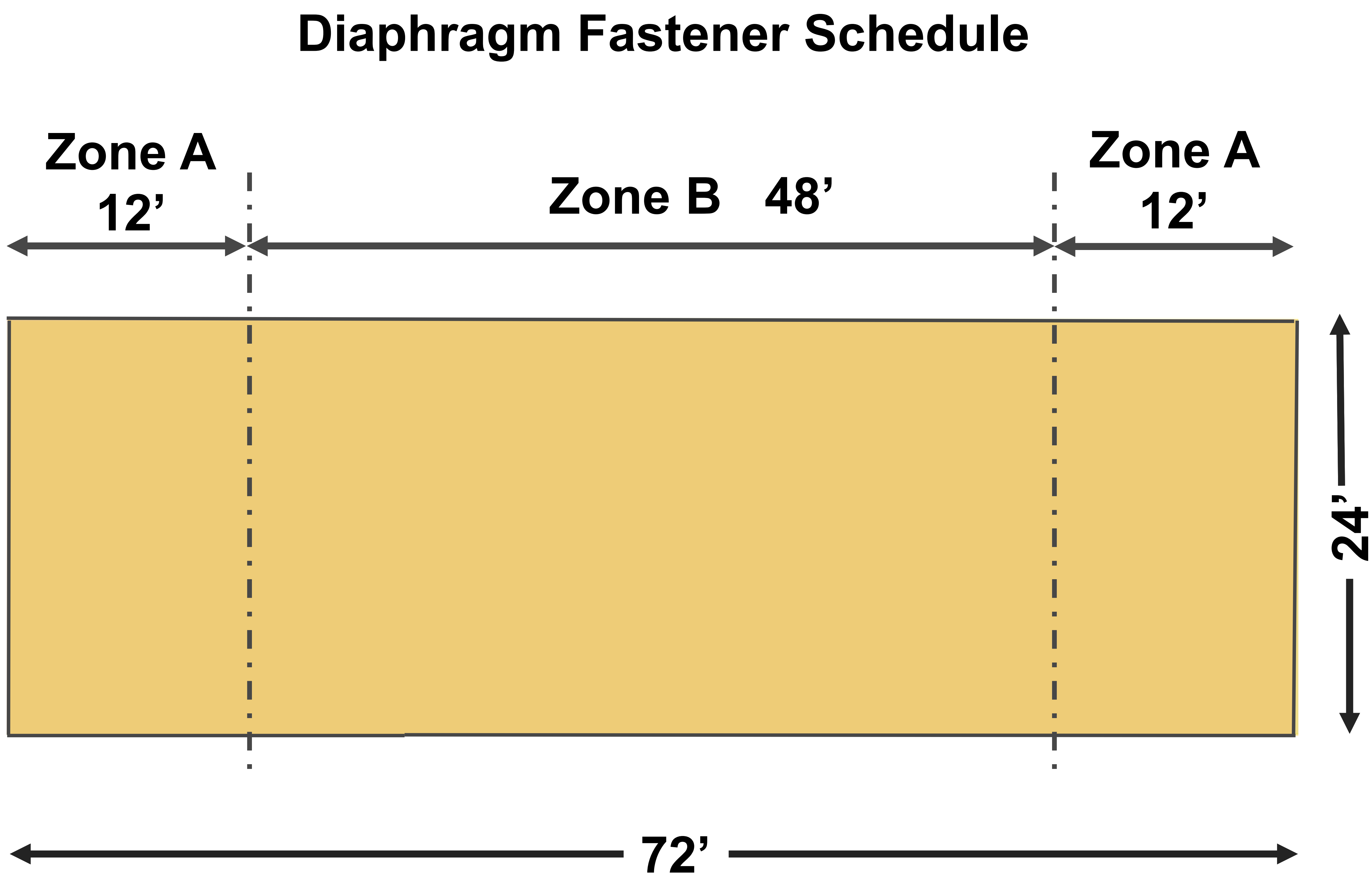
- Commentary to ASCE 7-10 Appendix C suggests that some recommend using 10 year return period wind forces:
    - ~ 70% of 700 return period wind (ultimate wind speed for risk category II buildings)
- Others (AISC Design Guide 3) recommend using 75% of 50 year return period forces

Drift Criteria

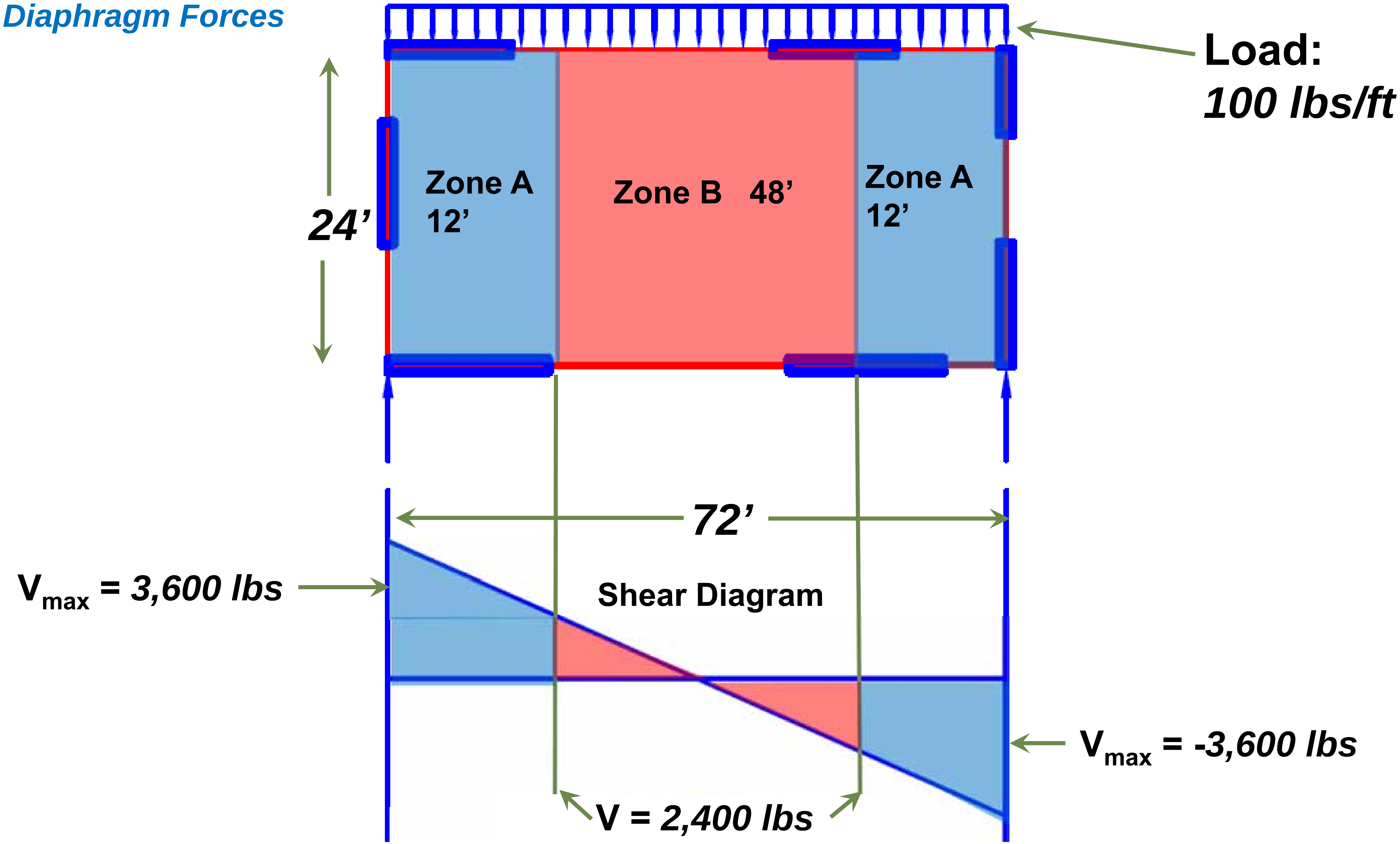
Can vary widely with brittleness of finishes but generally recommendations are in the range of H/240 to H/600

Diaphragm Design

Calculating Diaphragm Forces



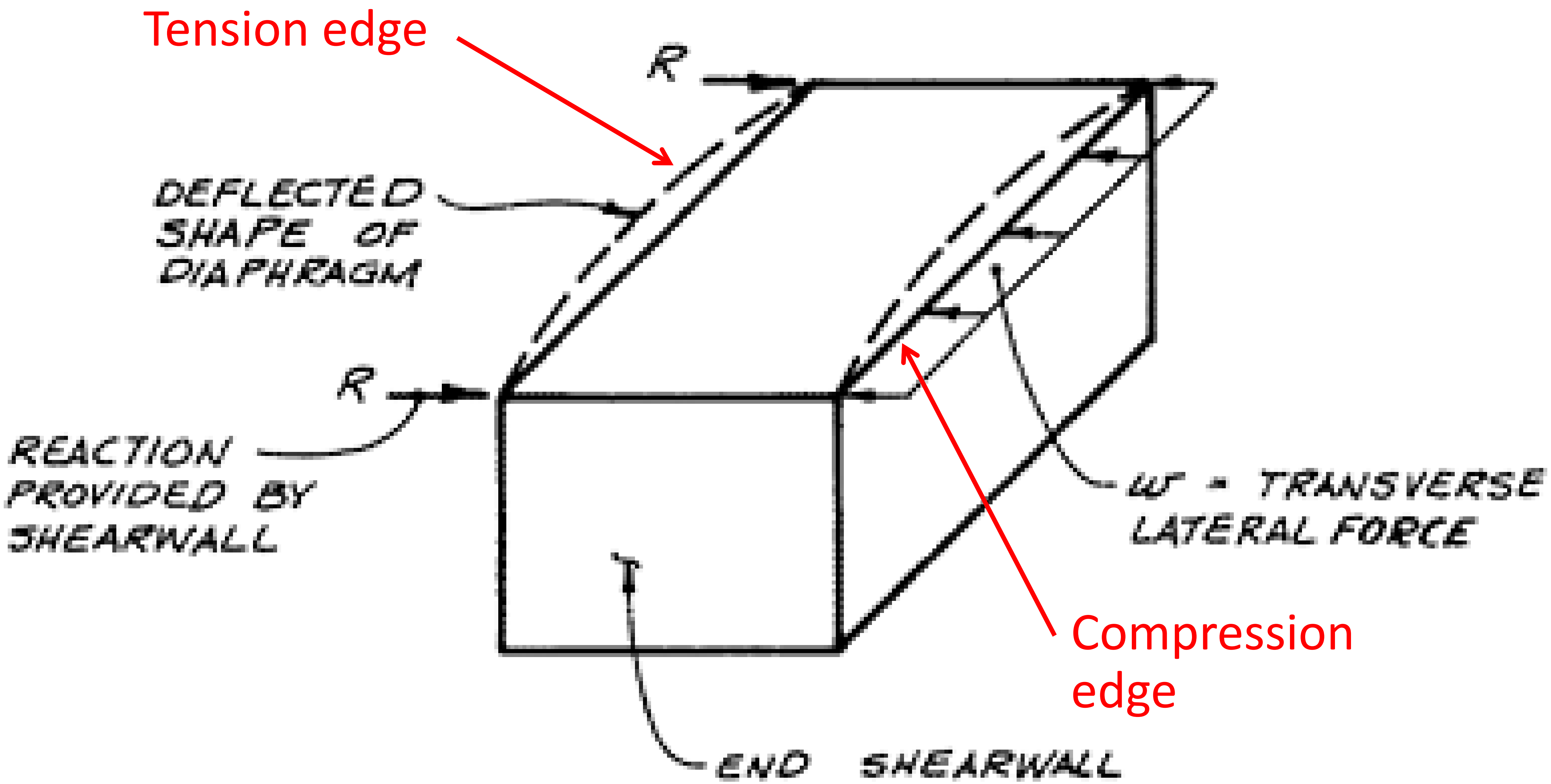
Diaphragm Forces



Diaphragm Fastener Schedule

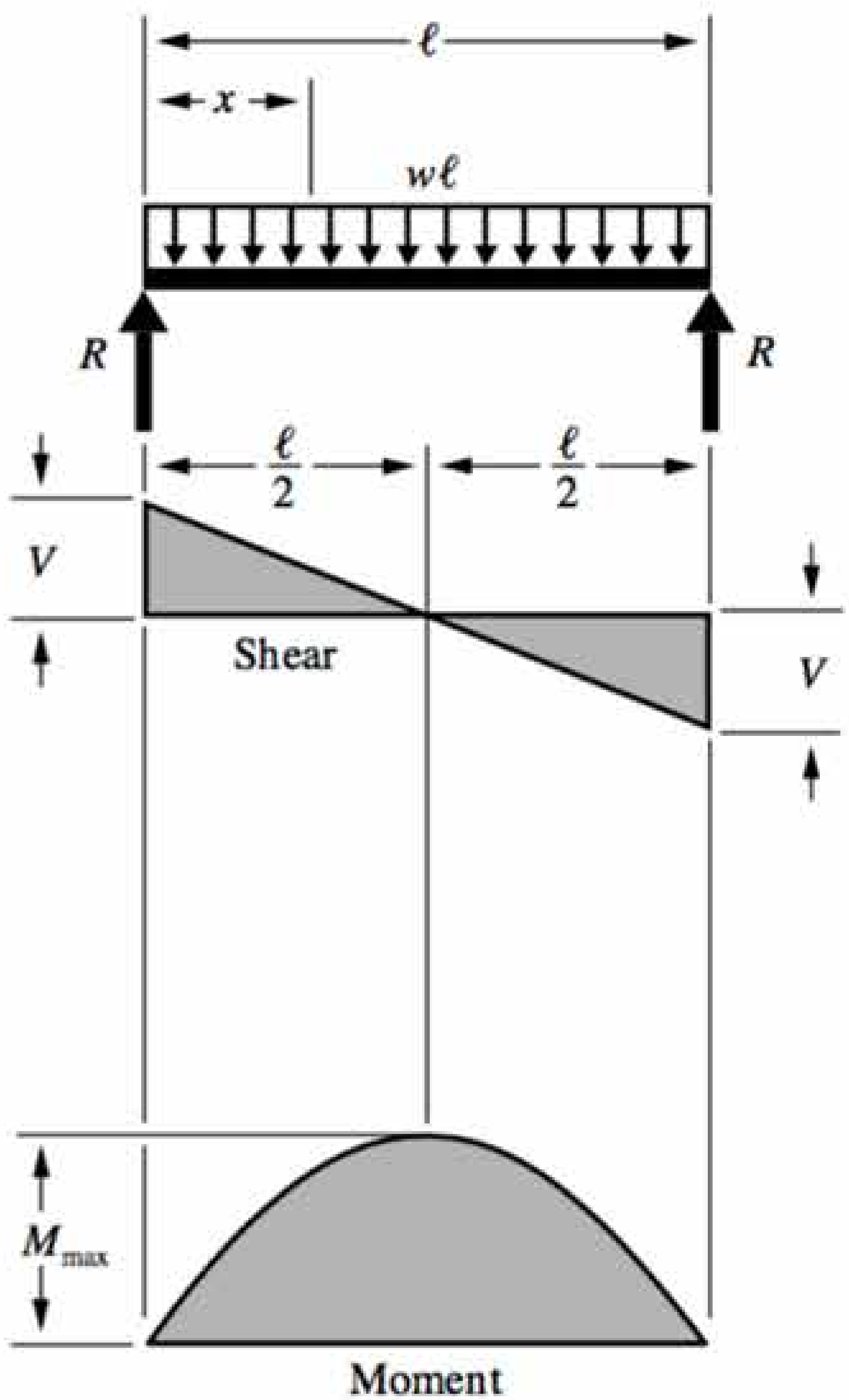
- *Zone A: Nailing Pattern 1*
- *Zone B: Nailing Pattern 2*

Diaphragm – Bending Member



Diaphragm Chord Forces

Figure 1 Simple Beam – Uniformly Distributed Load



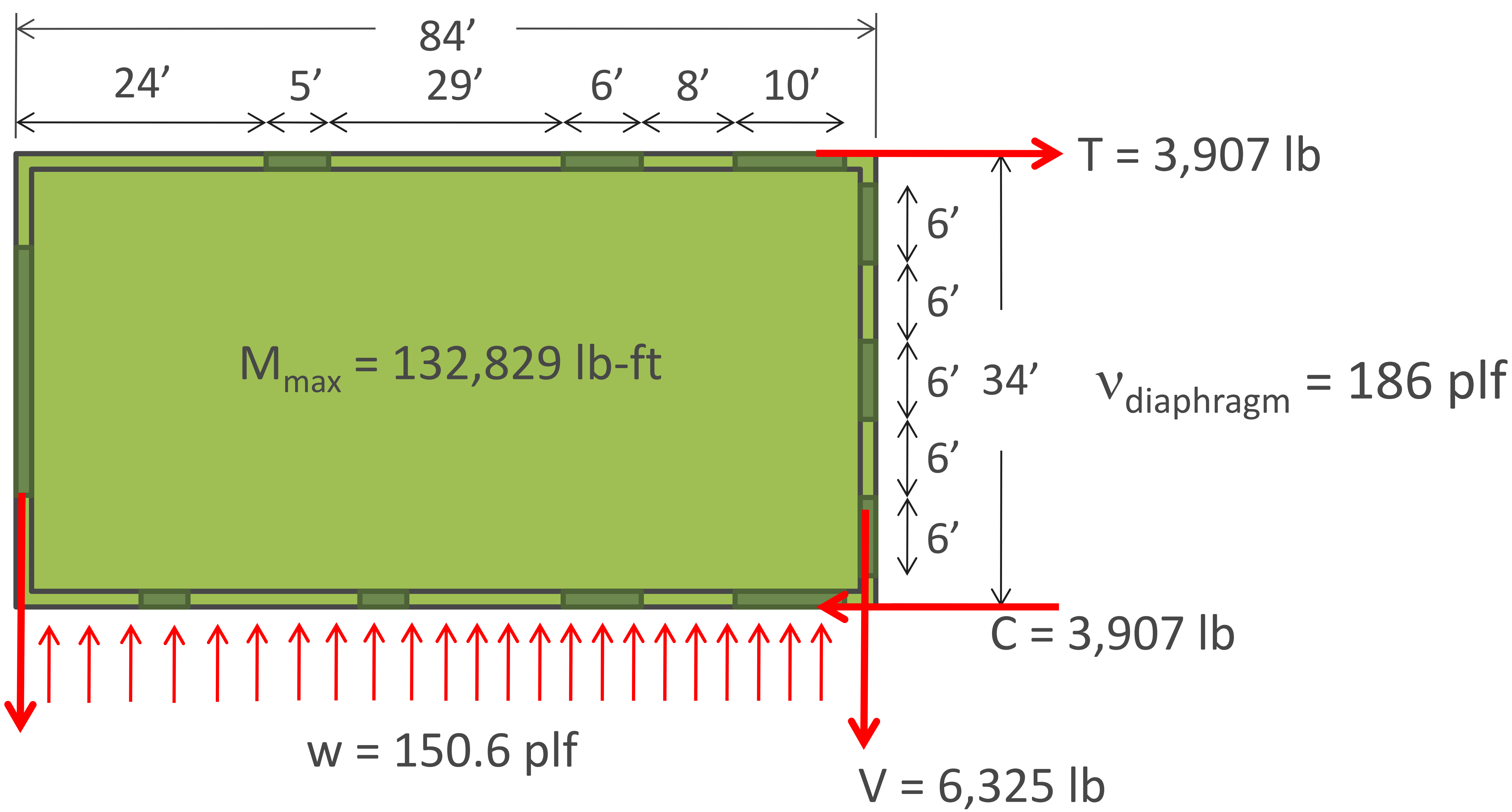
Diaphragm Chord Forces:

- 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 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:

$$\delta_{dia} = \overset{\text{(bending, chord deformation excluding slip)}}{\frac{5vL^3}{8EAW}} + \overset{\text{(shear, panel deformation)}}{\frac{vL}{4G_v t_v}} + \overset{\text{(shear, panel nail slip)}}{0.188Le_n} + \overset{\text{(bending, chord splice slip)}}{\frac{\sum(x\Delta_c)}{2W}} \quad \text{(C4.2.2-1)}$$

Diaphragm Design – Deflection

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

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

v = 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

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

Diaphragm Design – Deflection

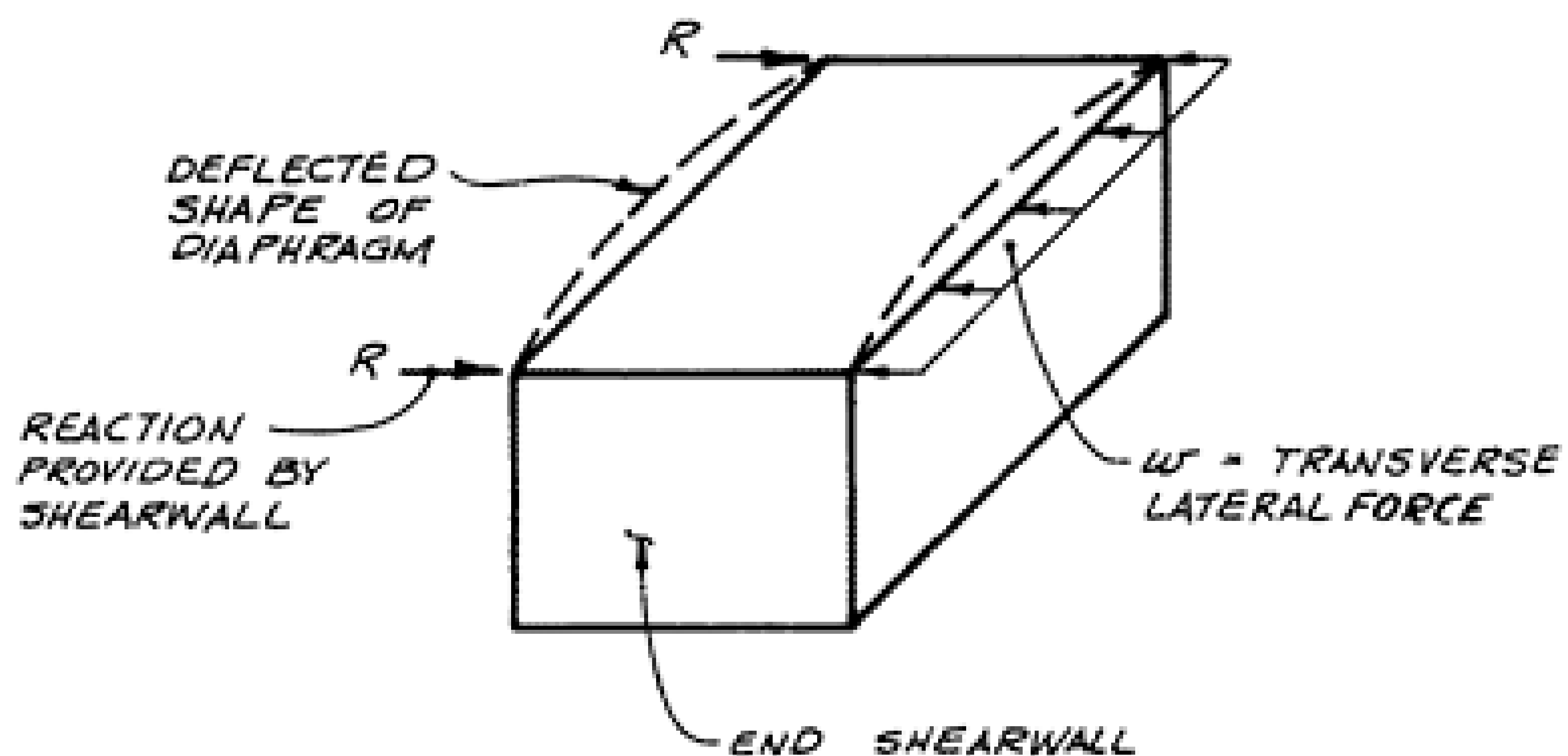
(bending, chord deformation excluding slip) | (shear, panel deformation) | (shear, panel nail slip) | (bending, chord splice slip)

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

$\Delta_{bending}$ = 5νL³ / 8EAW

= 5*(186 plf)*(84')³ / [8*(1,400,000 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)

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

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

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)

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

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

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***

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

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

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	Fastener Slip, e_n (in.) 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)

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

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

$\Delta_c = 2(T \text{ or } C) / \gamma n$ *(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)*

$\delta_{dia} = 0.78''$

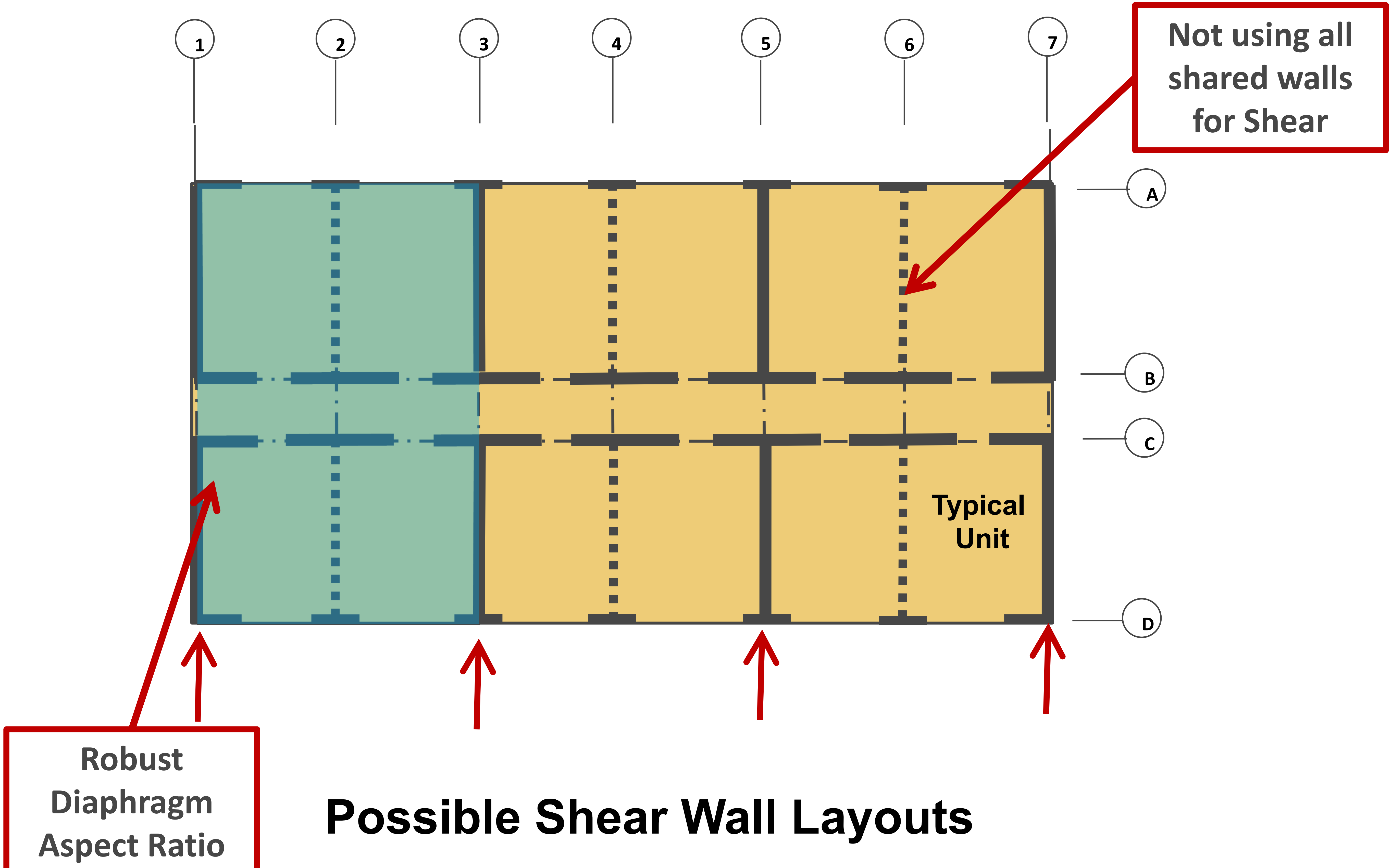
Rigid or Flexible Diaphragm?

Light Frame Wood Diaphragms often default to Flexible Diaphragms

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

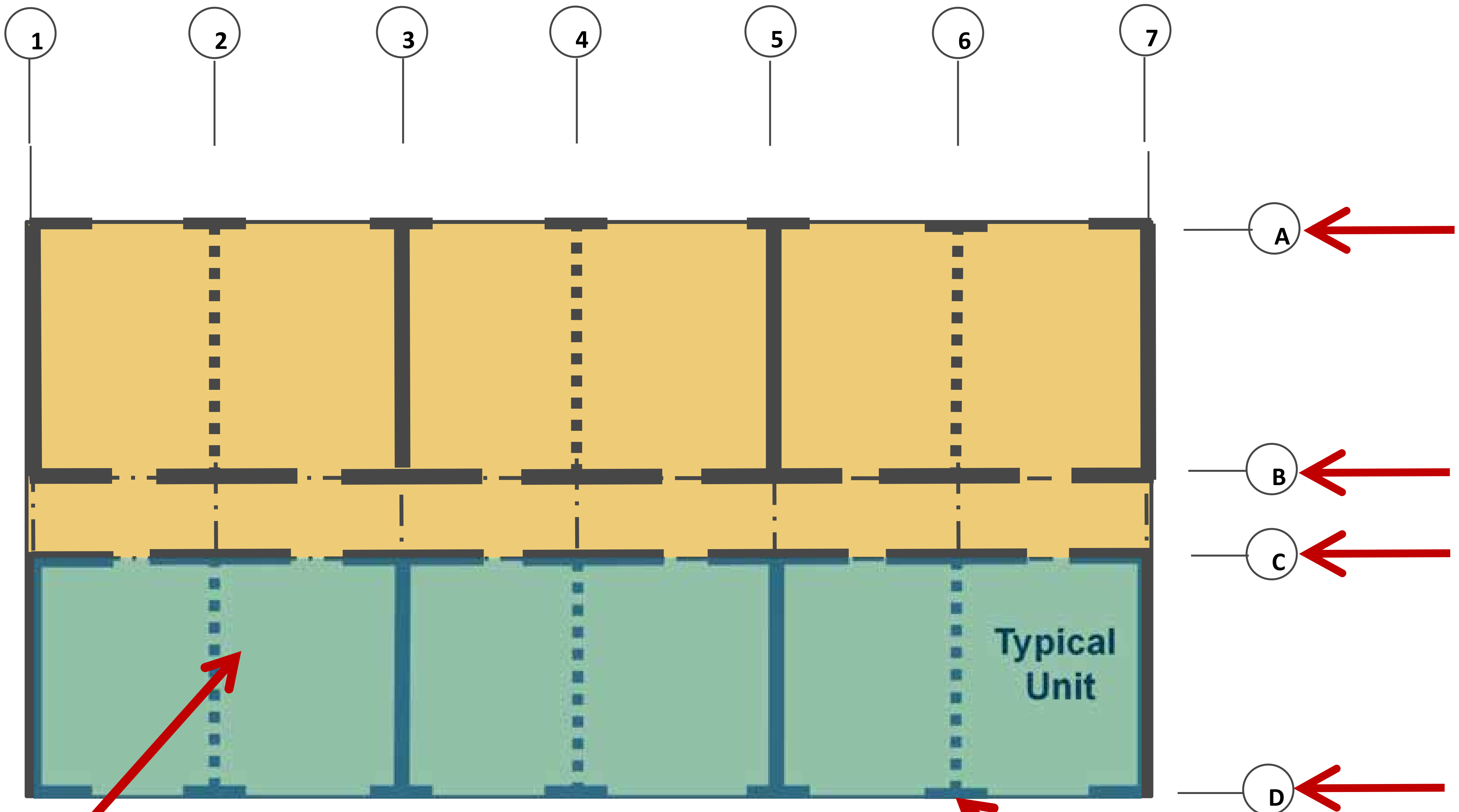
Diaphragms constructed of wood structural panels are permitted to be idealized as flexible

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)

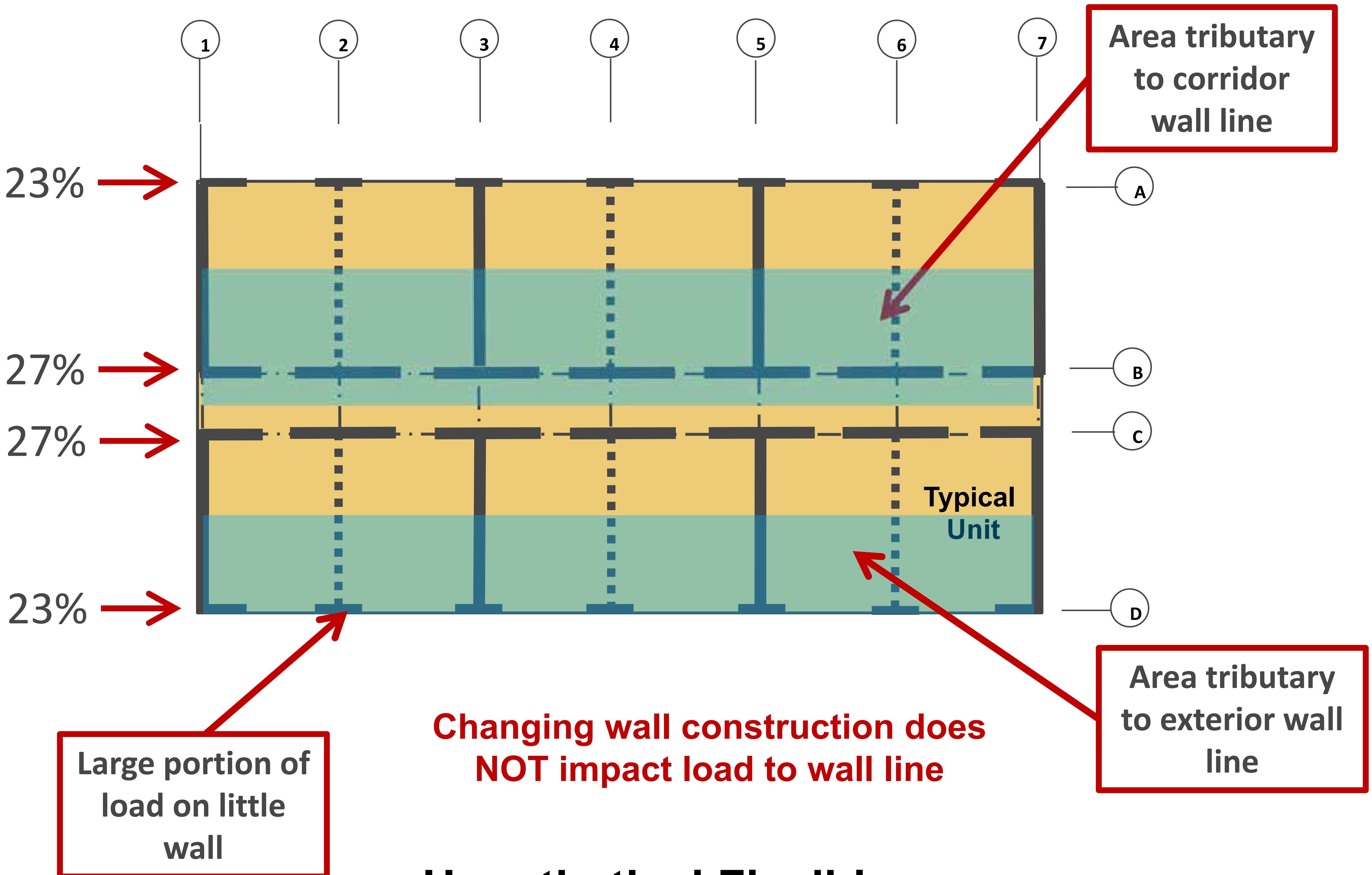
Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

[…]

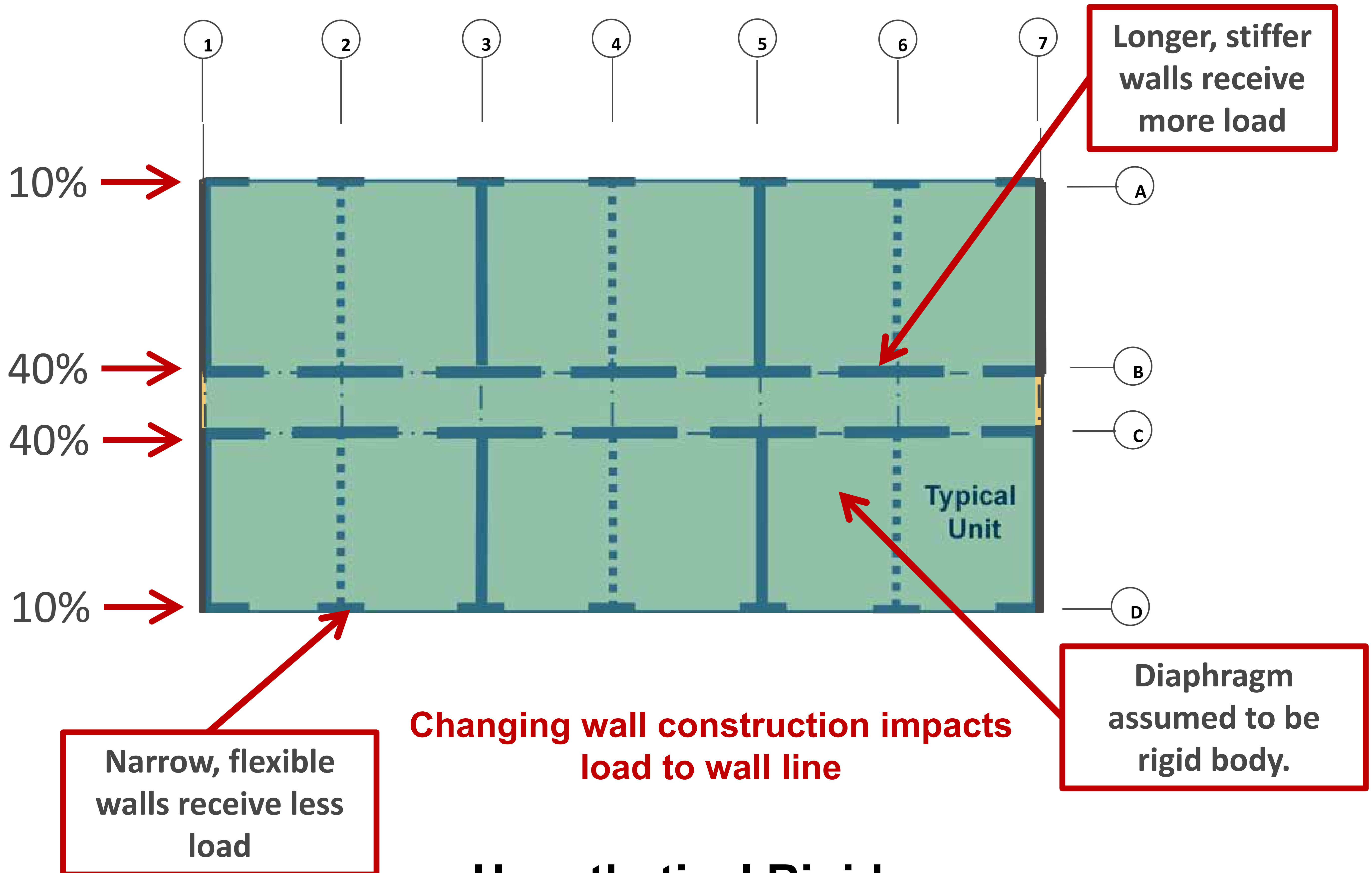
c. In structures of light-frame construction where all of the following conditions are met:

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

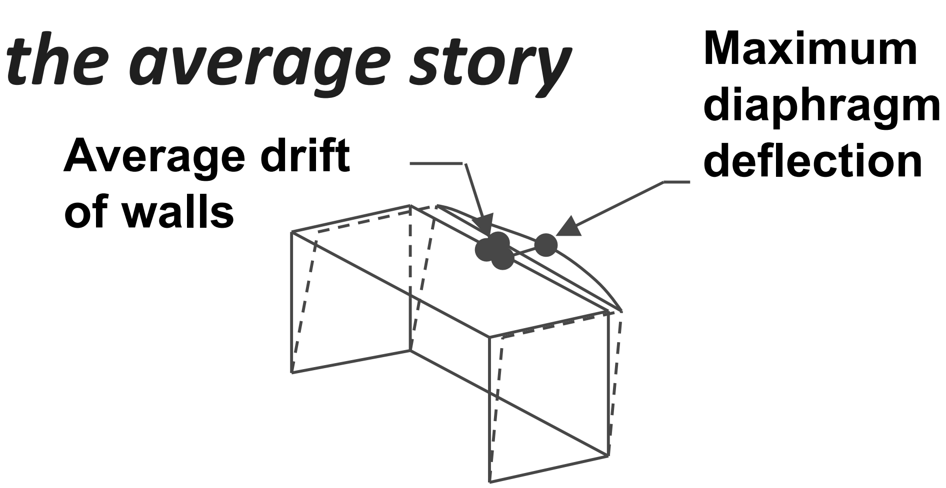
Can a Rigid Diaphragm be Justified?

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**.*



Rigid Diaphragm Analysis

Some Advantages of Rigid Diaphragm

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

Some Disadvantages of Rigid Diaphragm

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

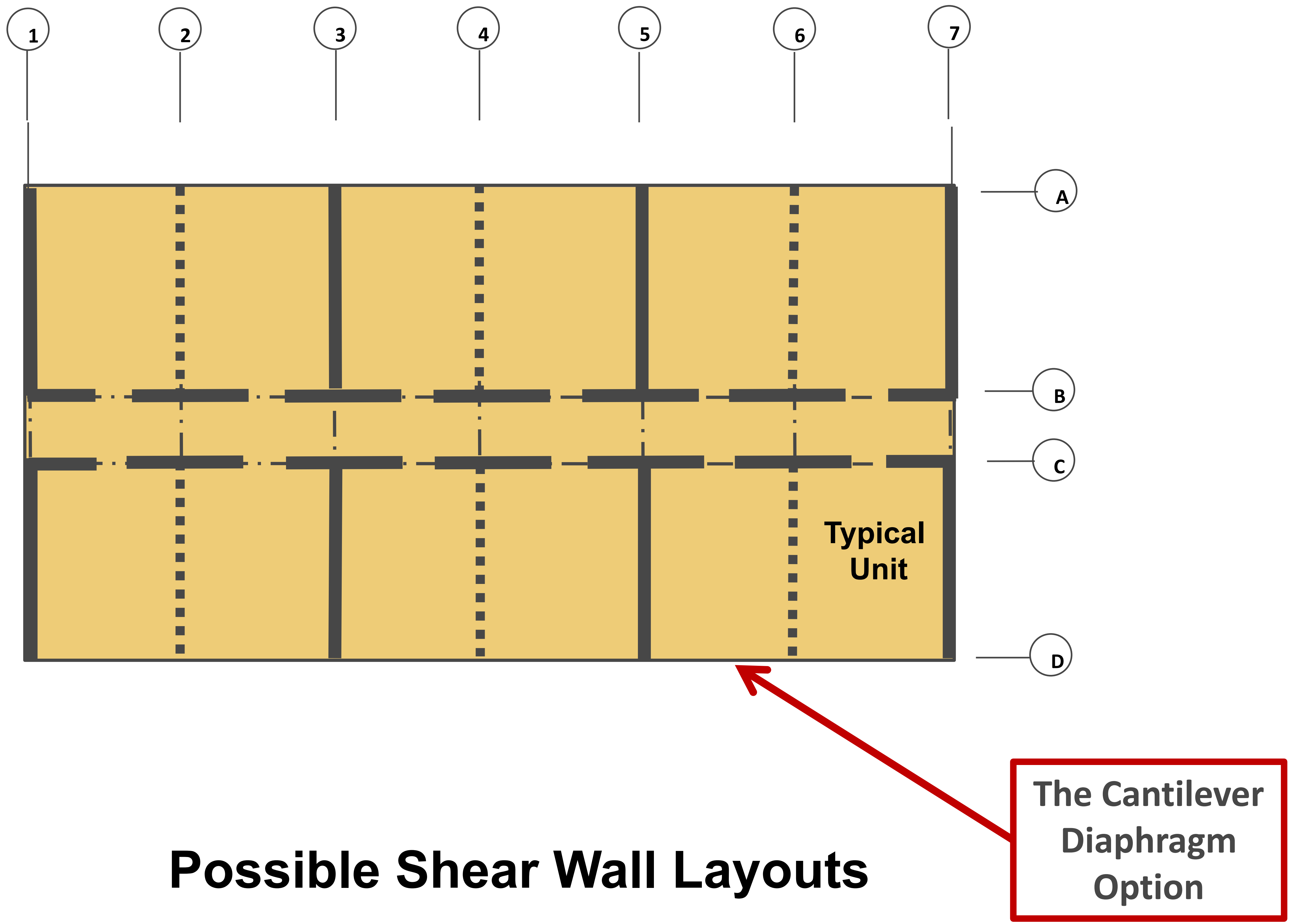
Two More Diaphragm Approaches

Semi-Rigid Diaphragm Analysis

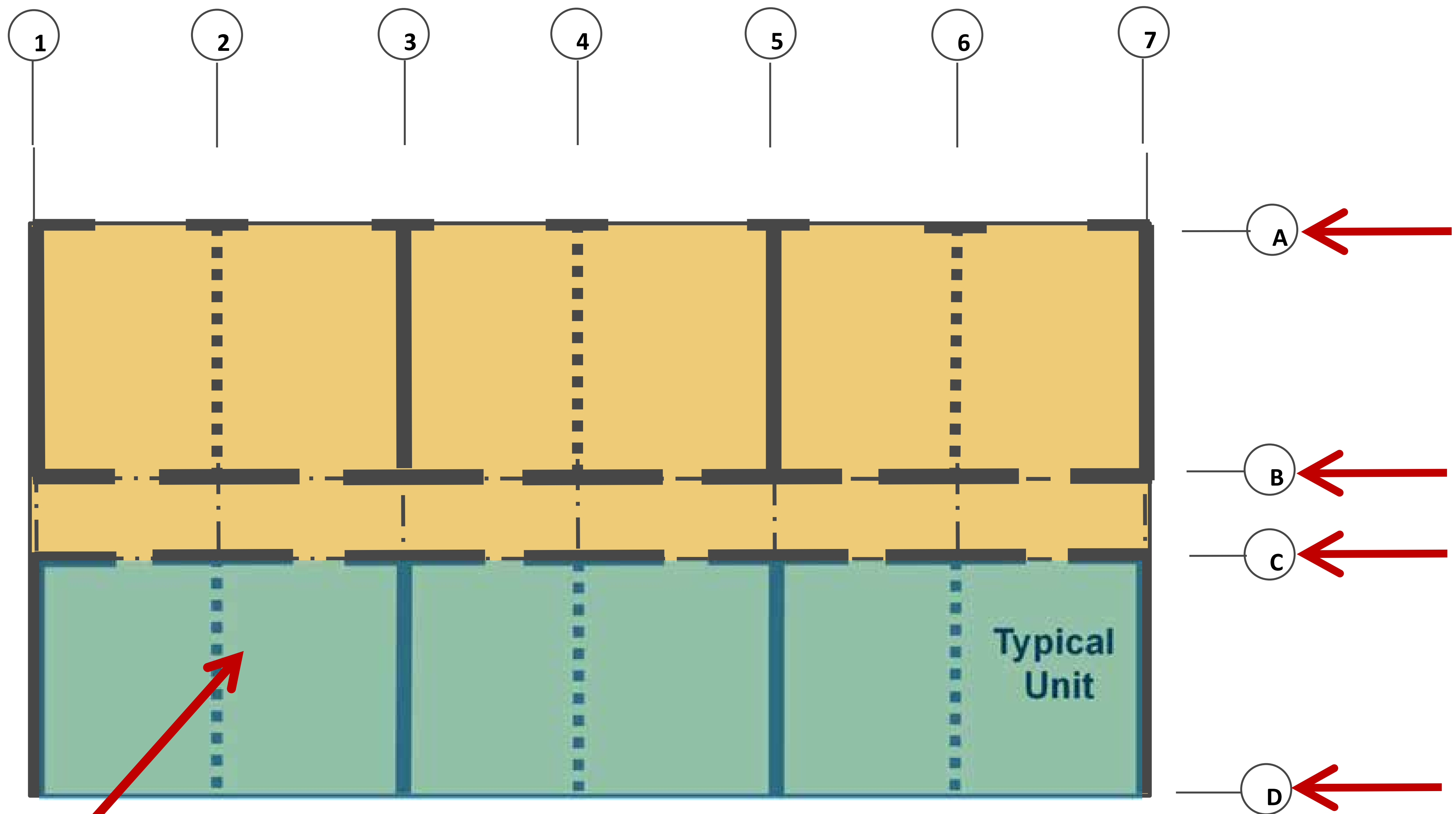
- Neither idealized flexible nor idealized rigid
- Explicit modeling of diaphragm deformations with shear wall deformations to distribute lateral loads
- Not easy

Enveloping Method

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



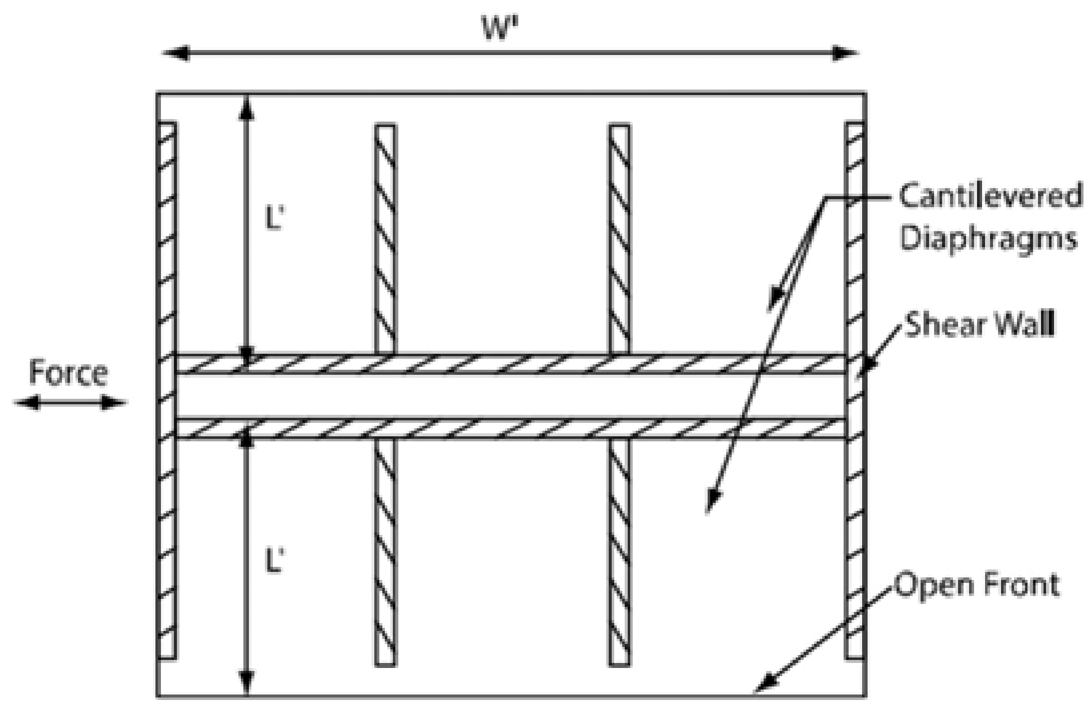
Possible Shear Wall Layouts



Possible Shear Wall Layouts

Open Front Structure & Cantilevered Diaphragms in SDPWS 2015

Cantilevered Diaphragm



SDPWS 4.2.5.2

L'/W' ≤ **1.5**

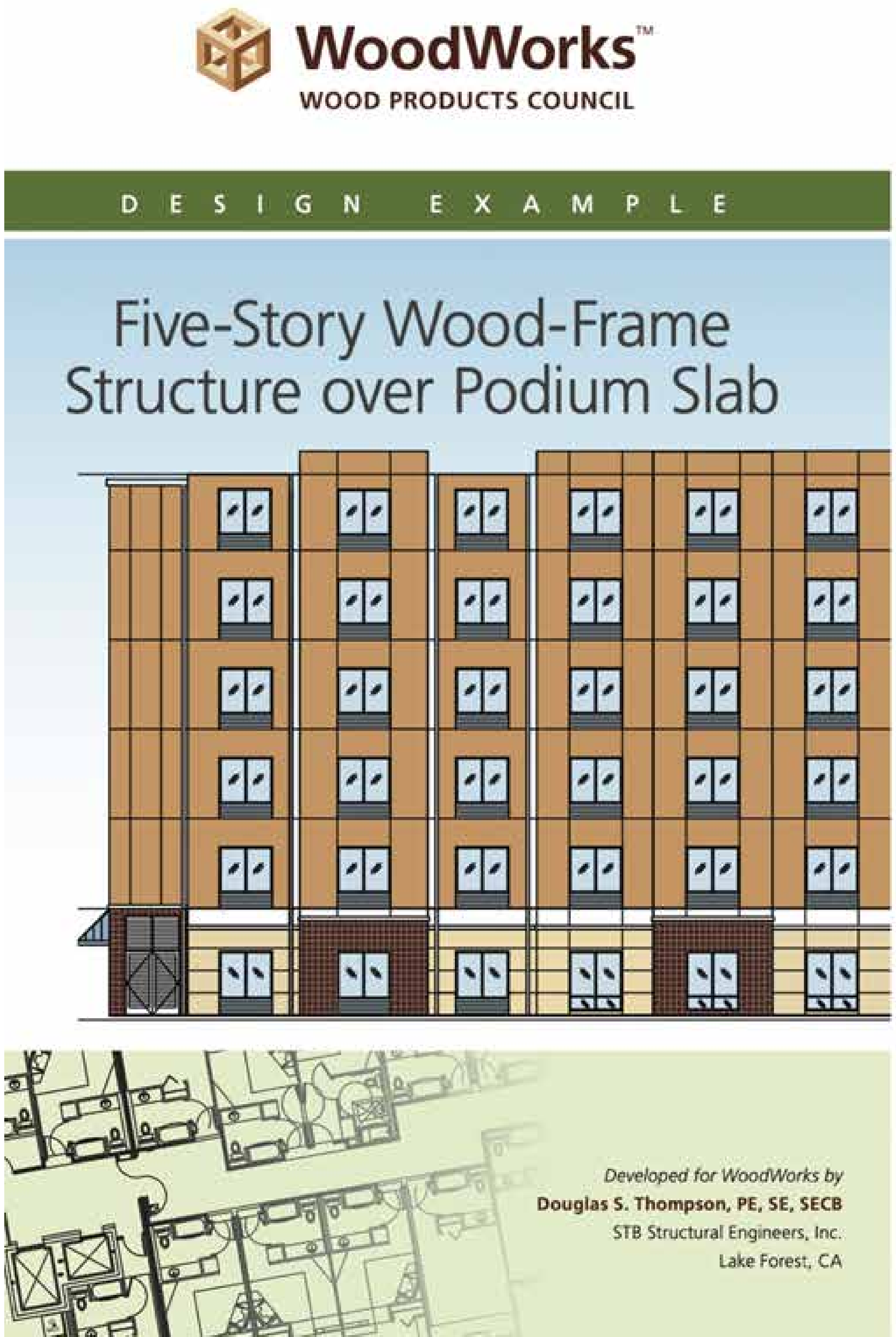
When Torsionally Irregular

L'/W' ≤ 1, one story

2/3, multi-story

L' ≤ **35** ft

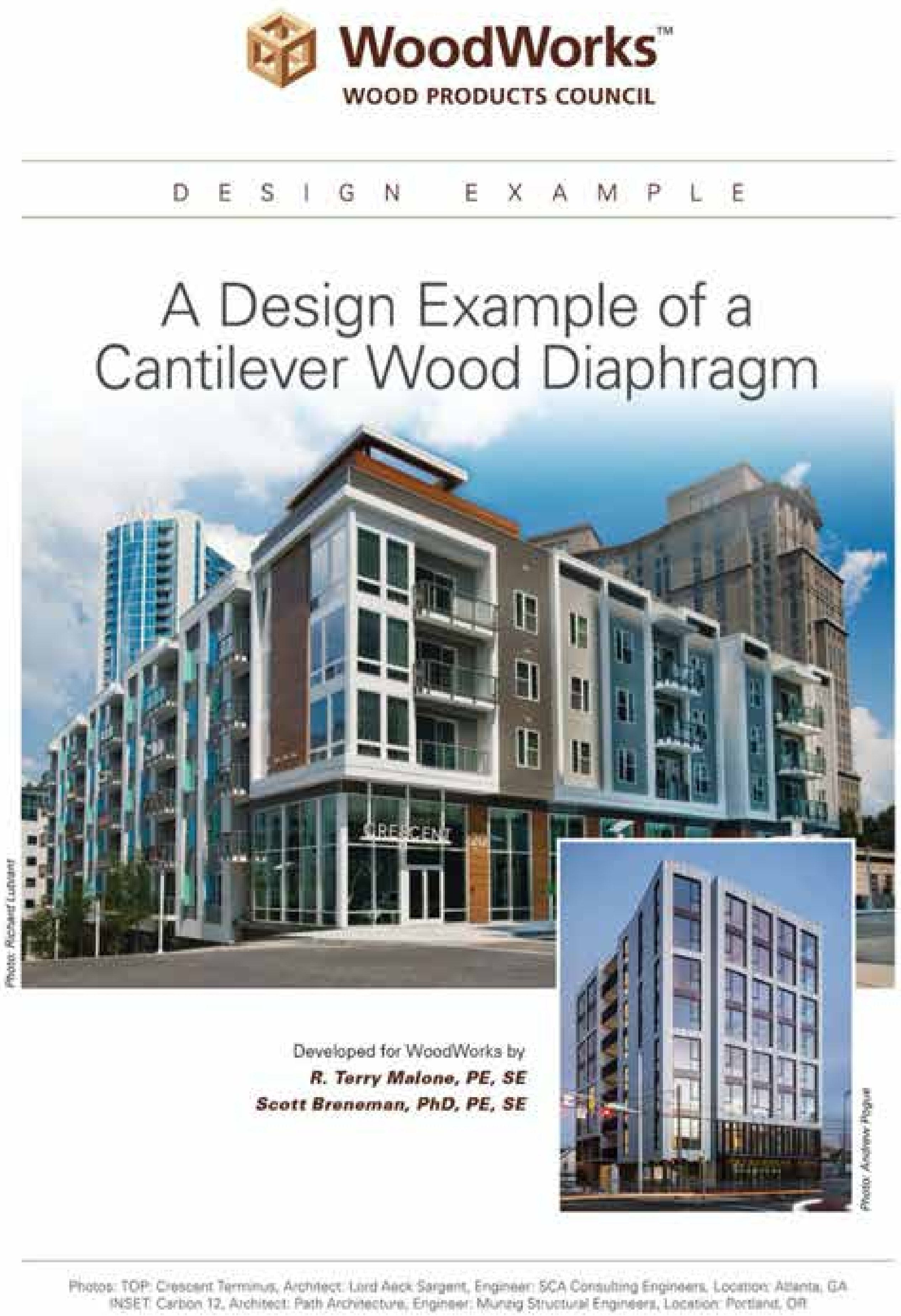
Provided diaphragms modelled as rigid or semi-rigid and for seismic, the story drift at each edge of the structure within allowable story drift of ASCE 7. Story drifts include torsion and accidental torsional loads and deformations of the diaphragm.


WoodWorks™
WOOD PRODUCTS COUNCIL

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

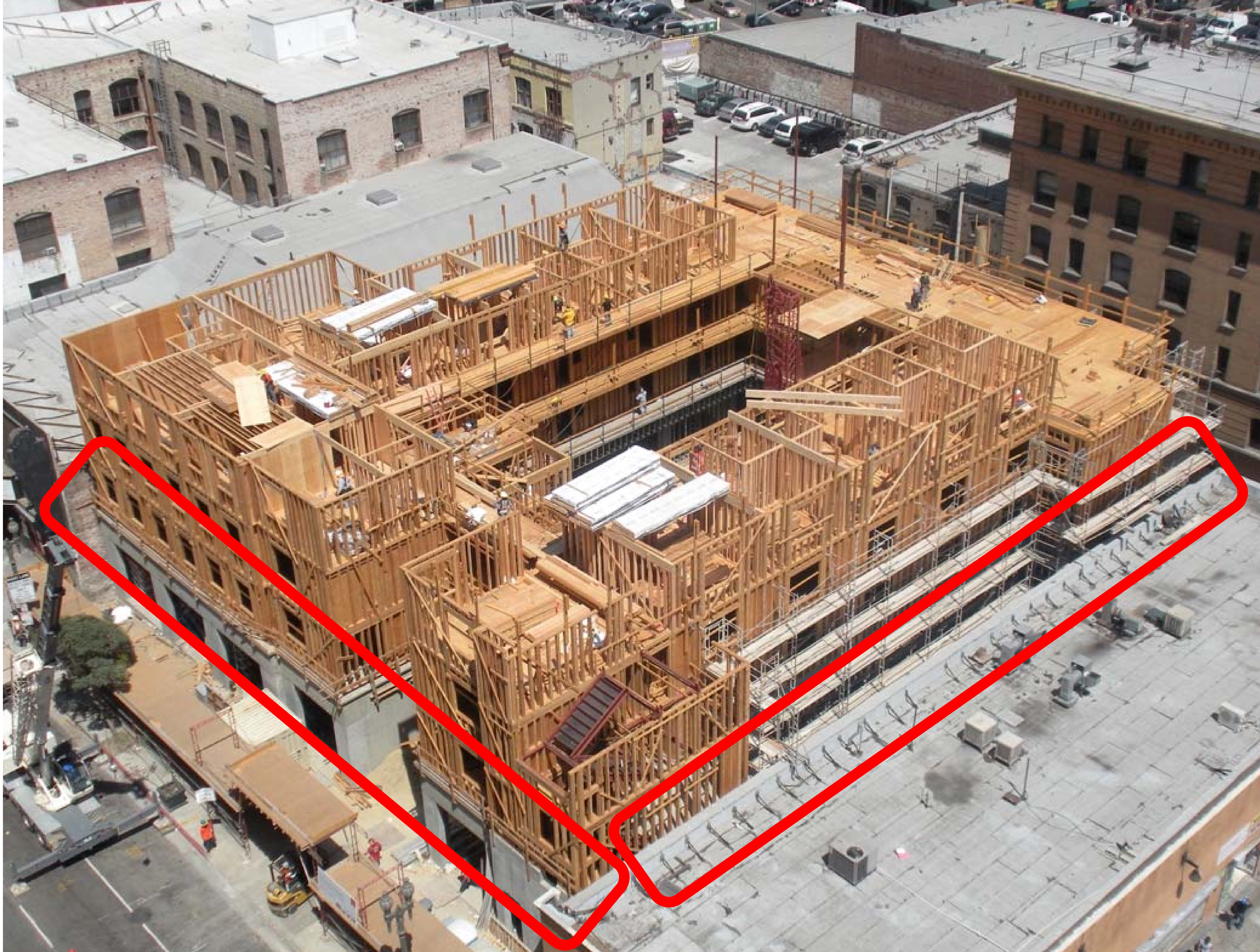
A Design Example of a Cantilever Wood Diaphragm

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



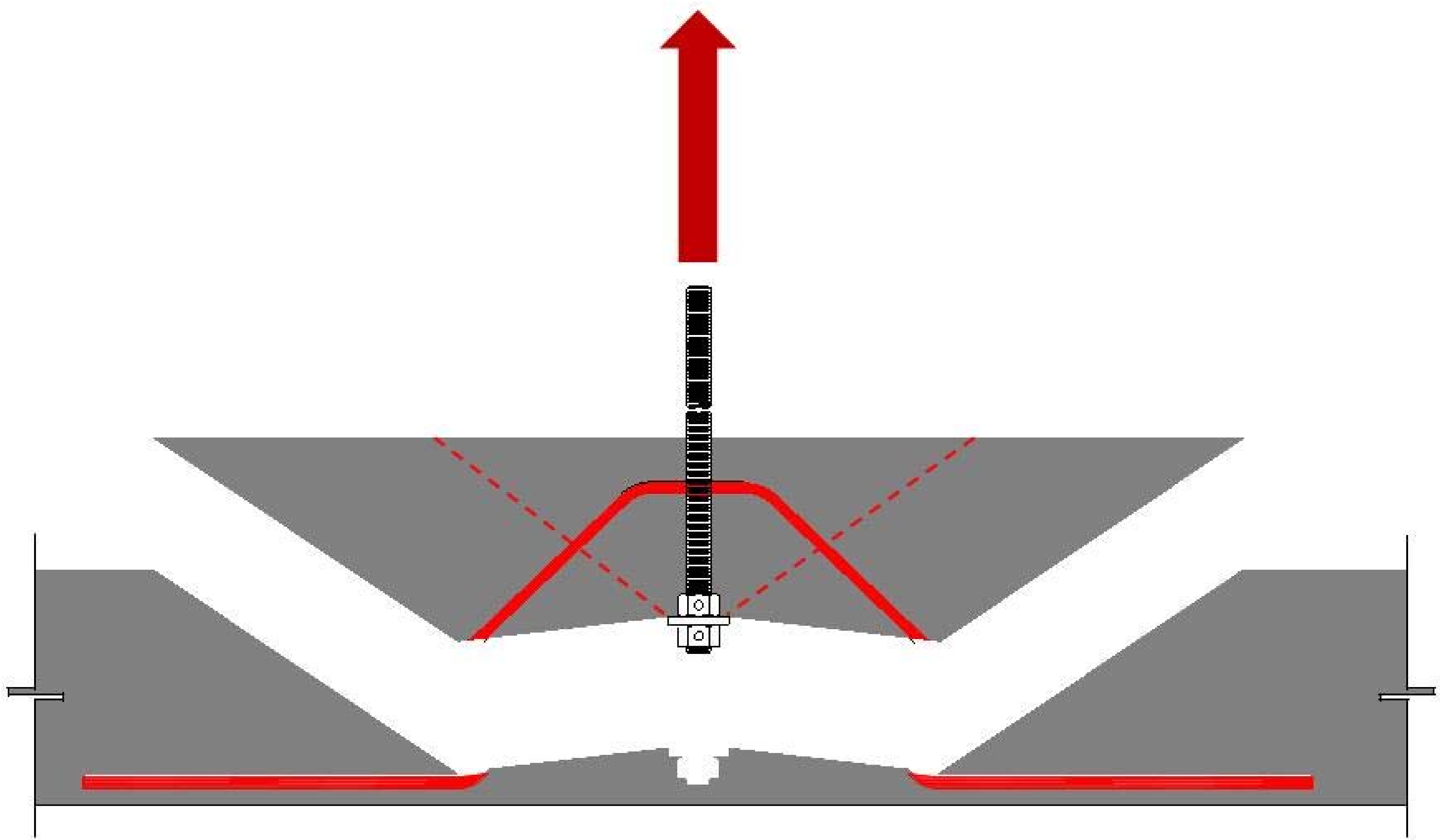
Shear Wall to Podium Slab Interface

- 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

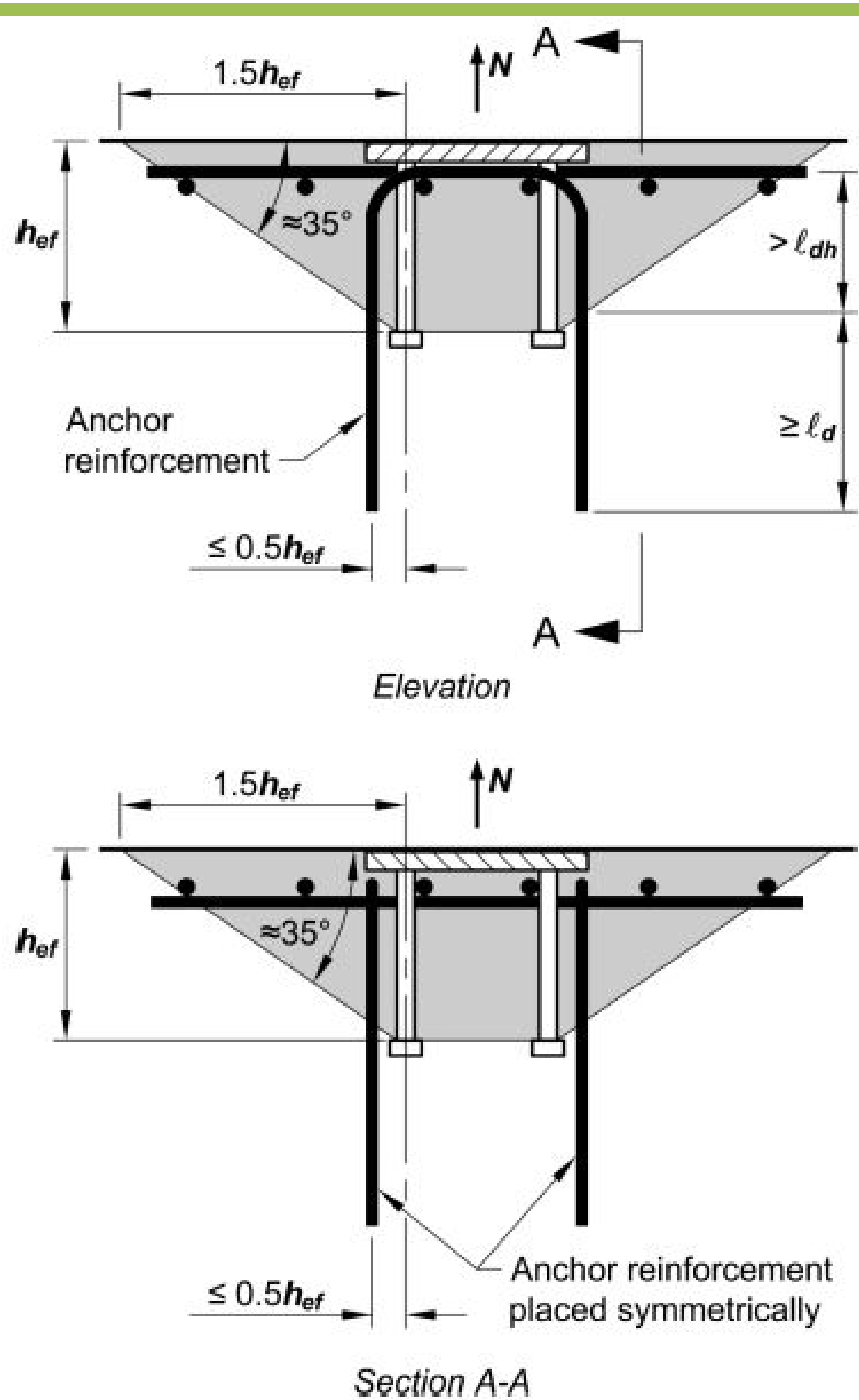
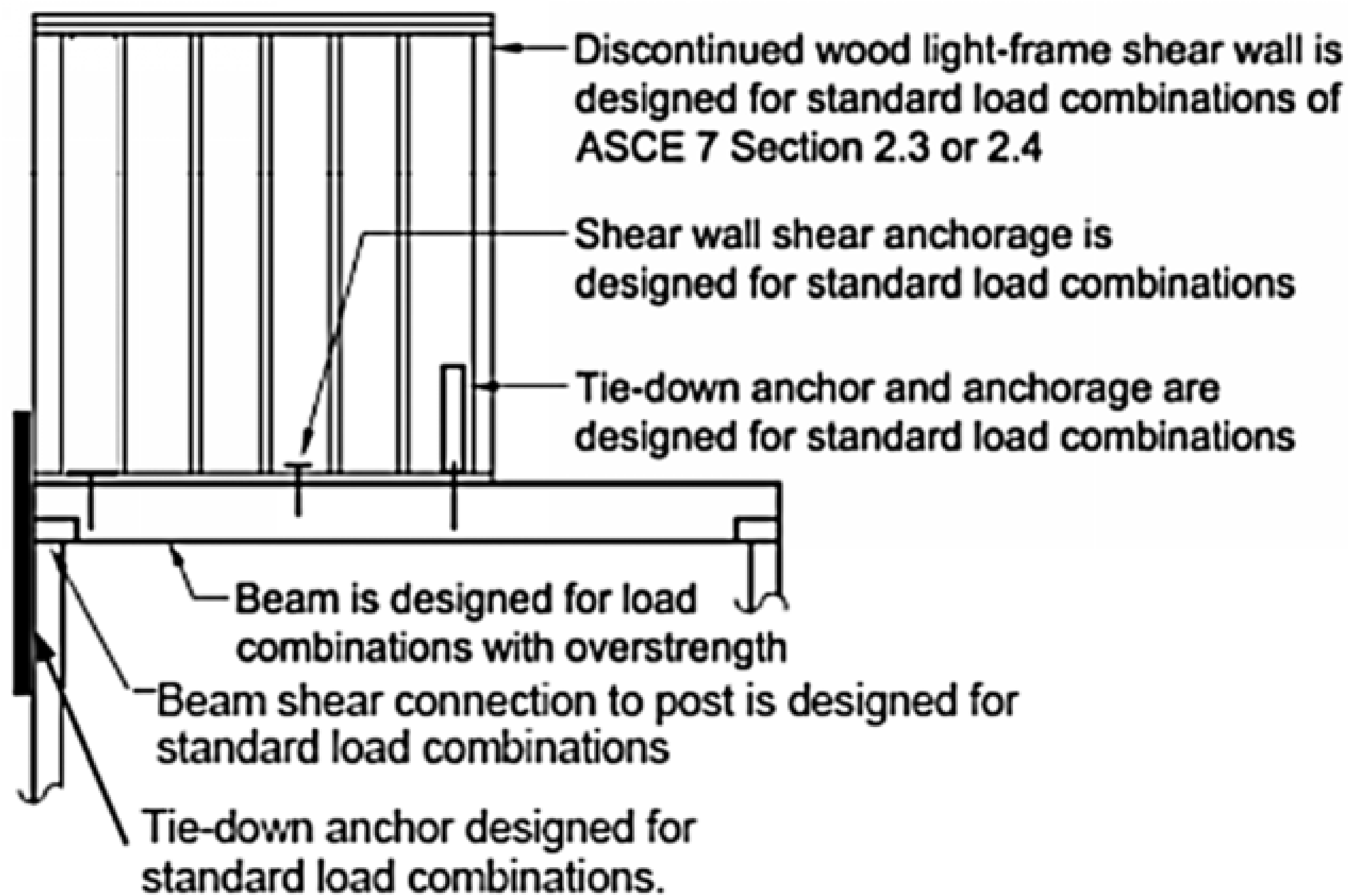


Fig. RD.5.2.9—Anchor reinforcement for tension.

Tie Down Anchors – Precast Through Bolt

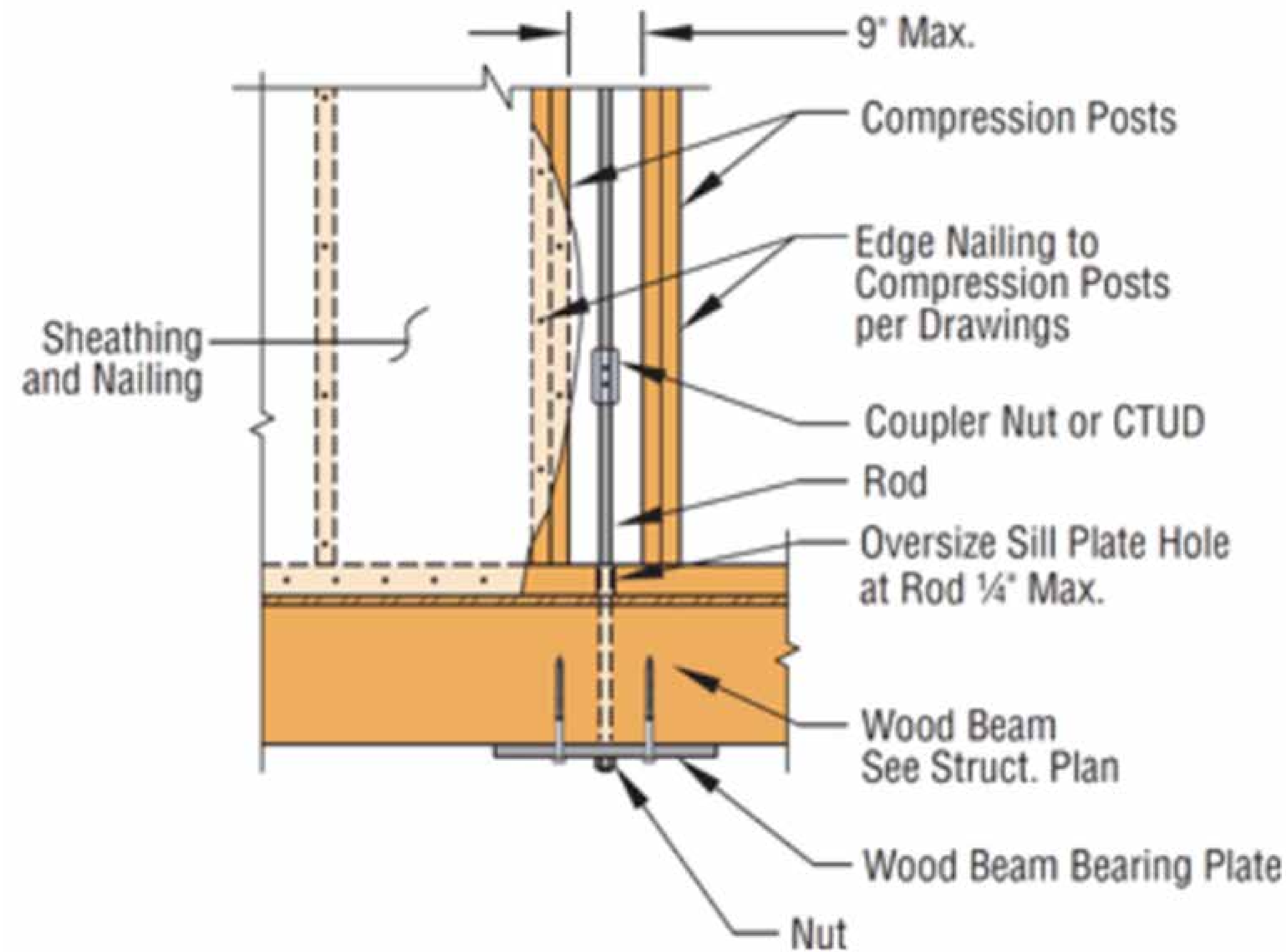
Tie Down Anchors – Through Podium

Shear Wall to Podium Slab Interface



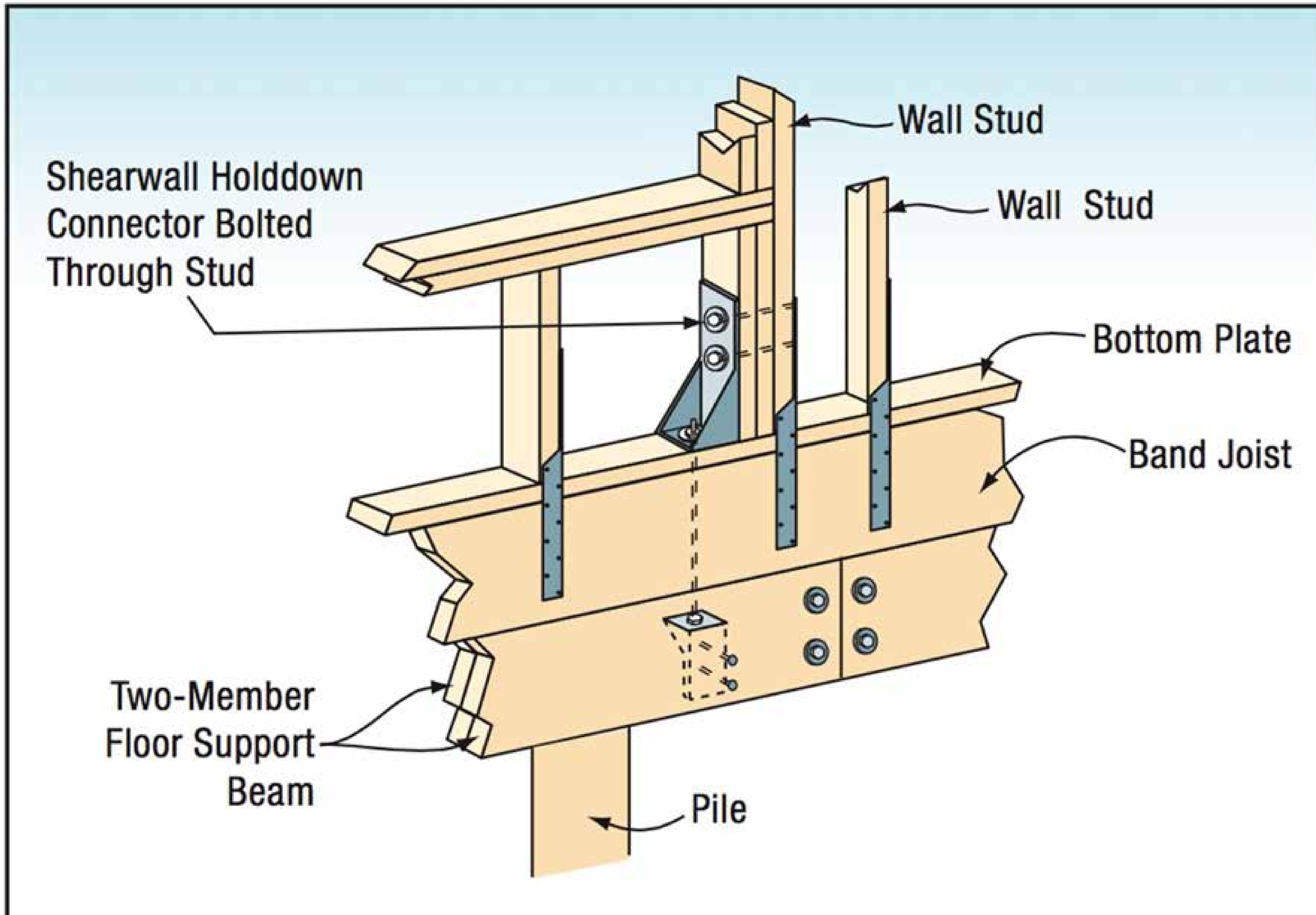
ASCE 7-10 Section 12.3.3.3 and Commentary C12.3.3.3 provides guidance on seismic load requirements for various elements supporting discontinuous shear walls

Offset Shear Wall Overturning Resistance



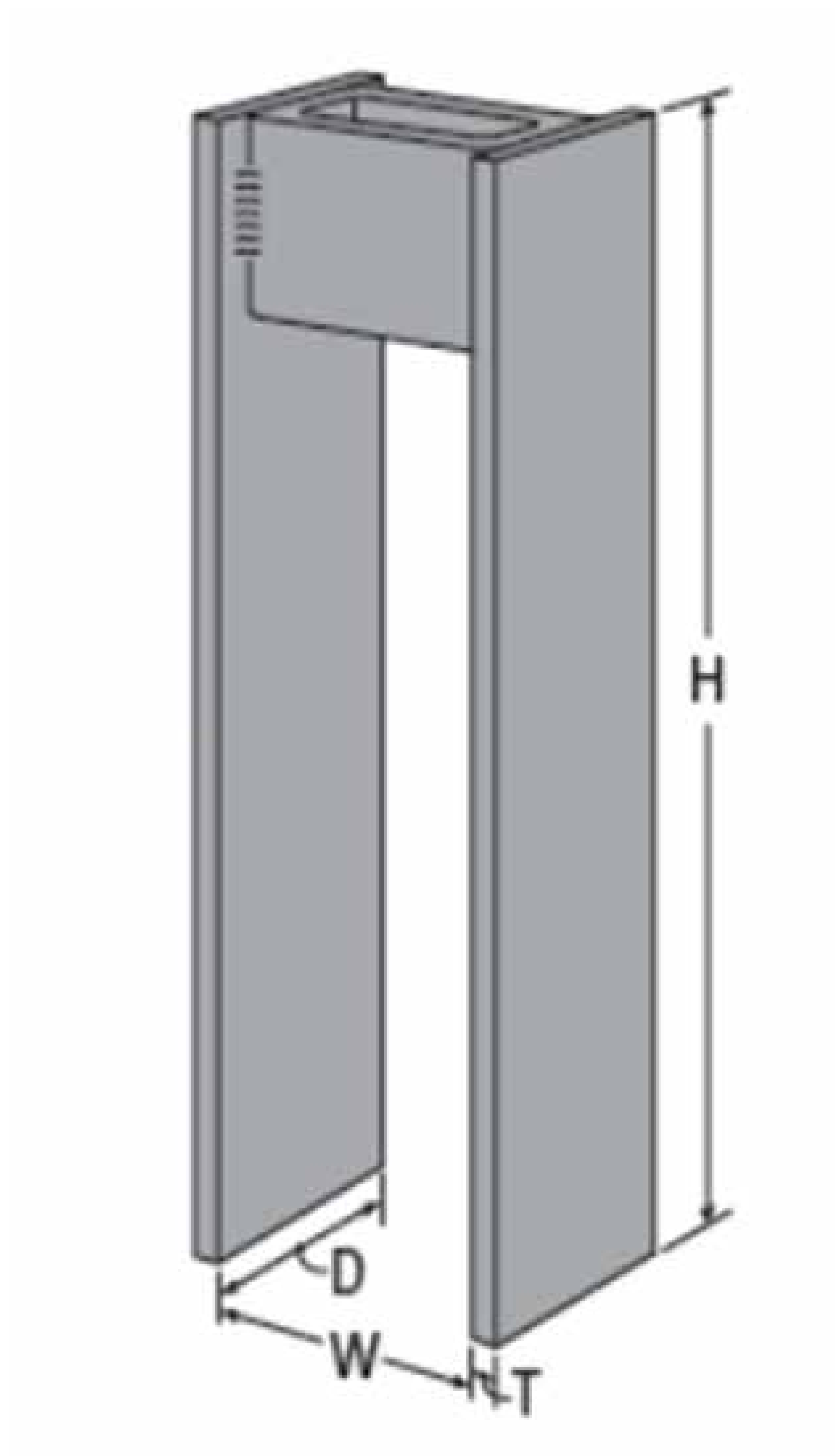
Source: Strongtie

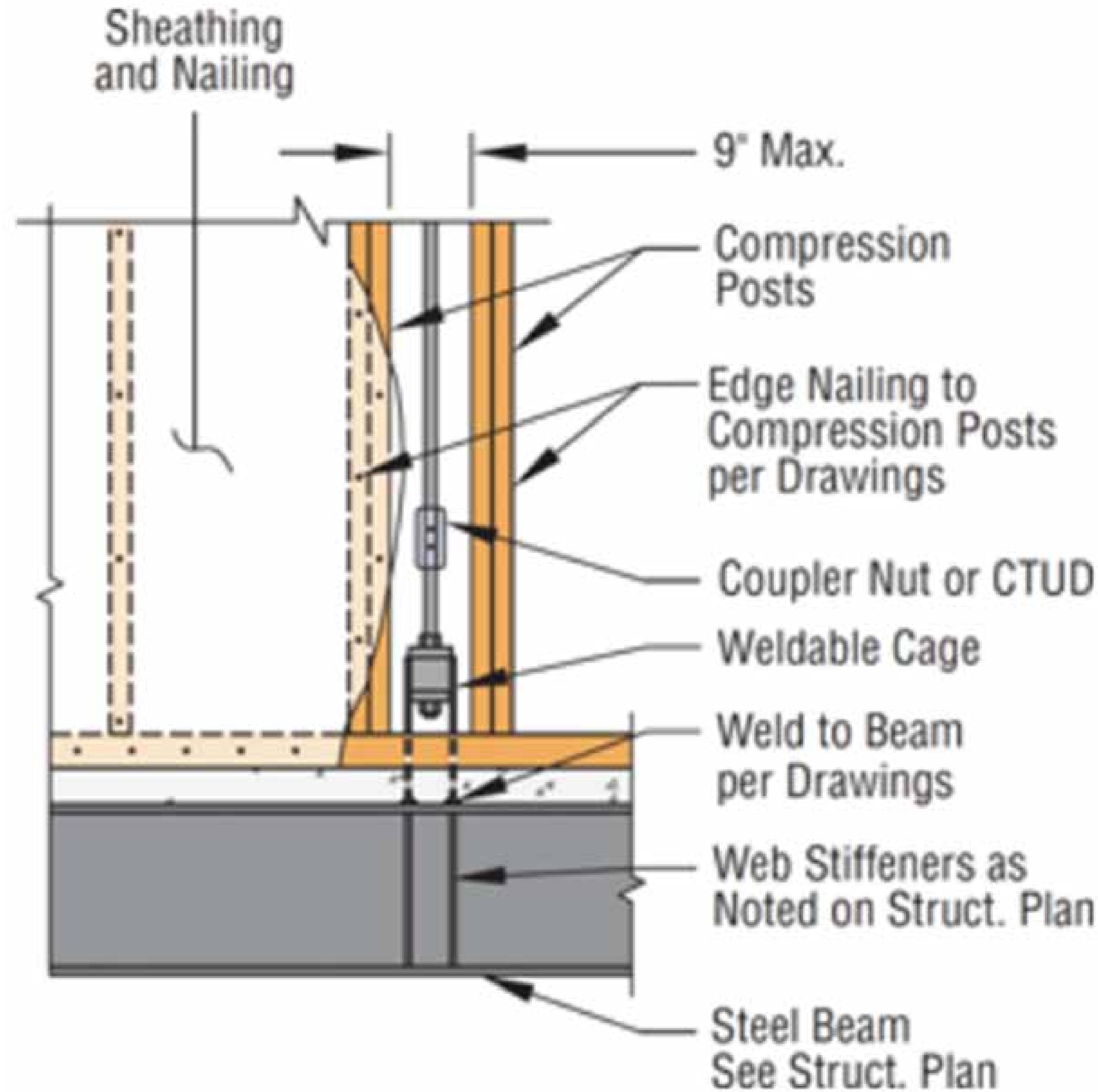
Offset Shear Wall Overturning Resistance



Source: FEMA 55

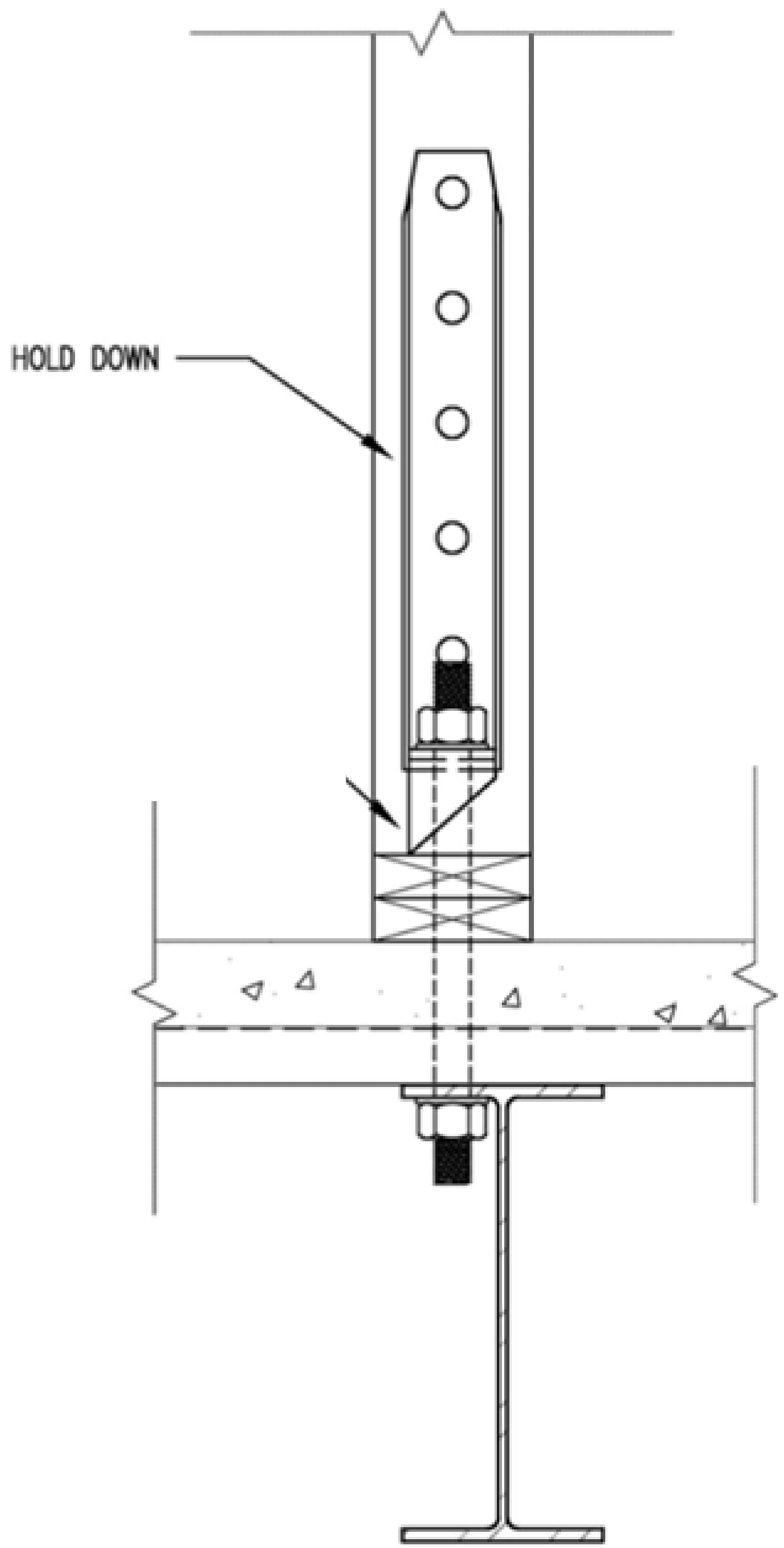
Tie Down to Steel Beam Attachment

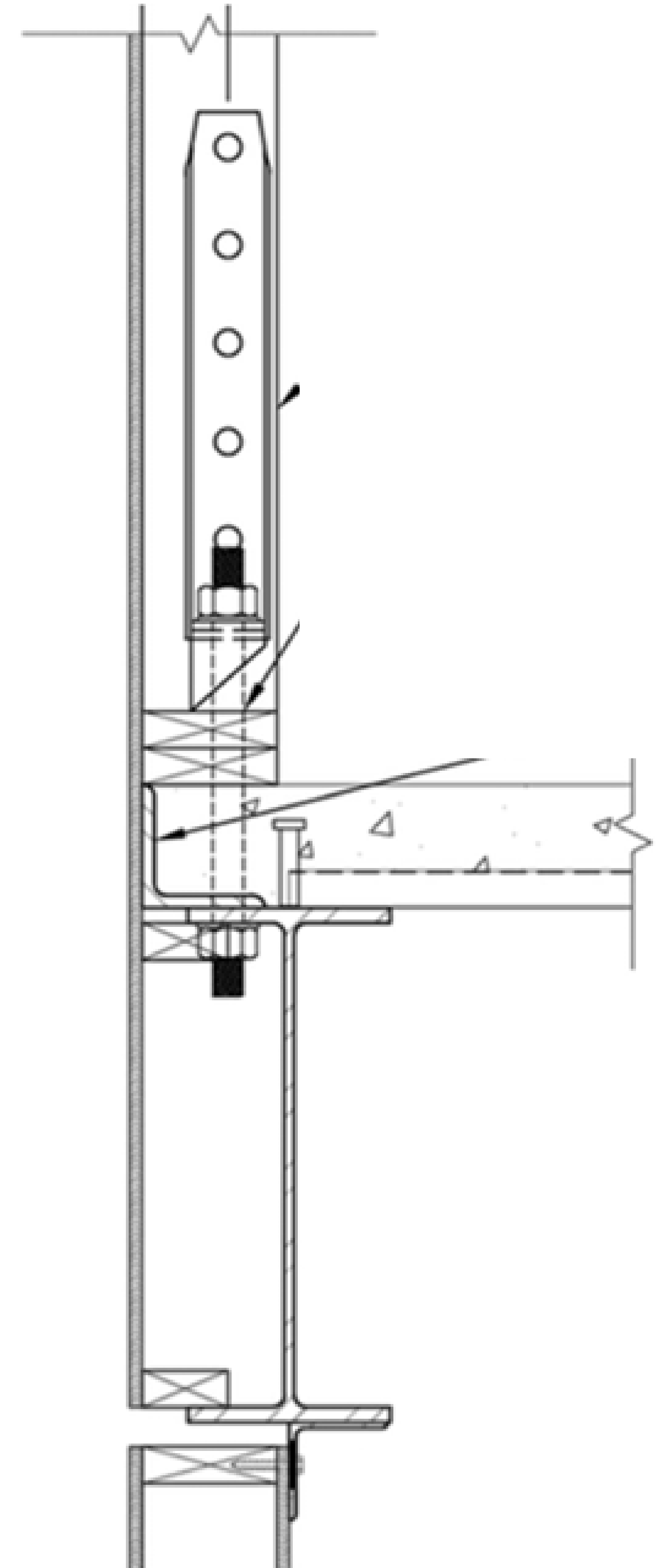




Source: Strongtie

Tie Down to Steel Beam Attachment





QUESTIONS?

This concludes The American Institute of Architects Continuing Education Systems Course

Anthony Harvey, PE

Regional Director

anthony.harvey@woodworks.org

Copyright Materials