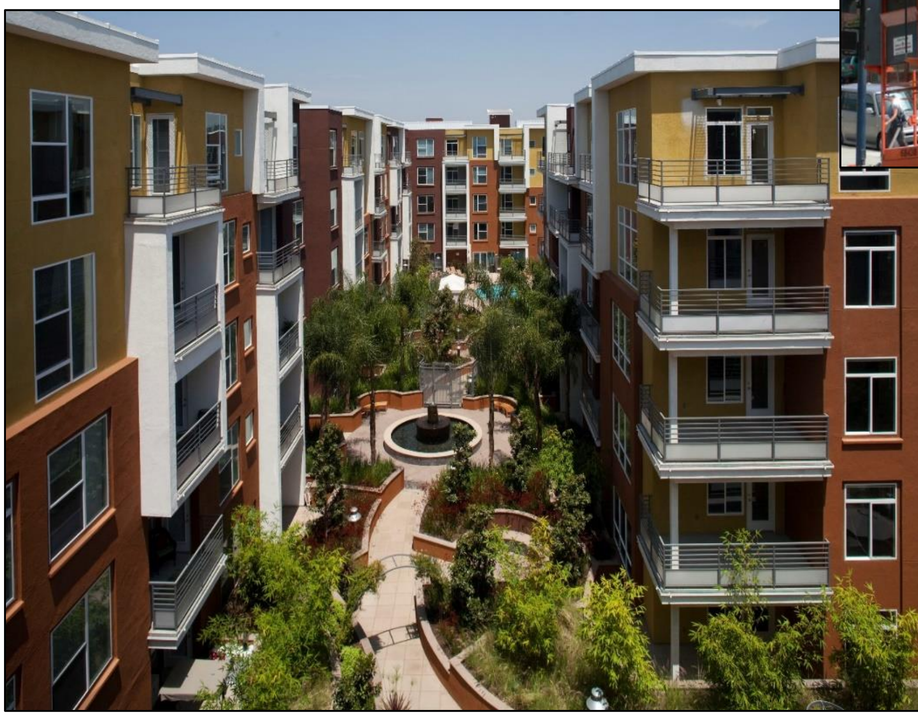
WoodWorks

Mid-rise Shear Walls & Diaphragms

120 Union, San Diego, CA
Togawa Smith Martin



Avalon Bay Communities
Photo credit: Arden Photography

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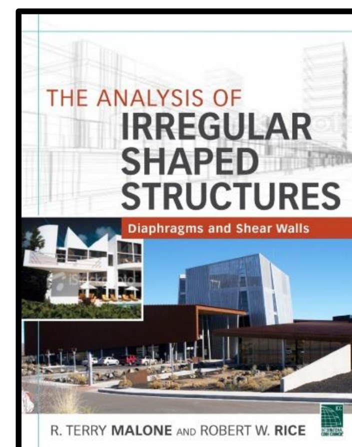
Portions Based On:



Course Description

This presentation will provide an in-depth look at shear wall and diaphragm designs for low-rise and mid-rise buildings, including those with rectangular, offset and open-front plans. This presentation is intended for structural engineers.

Mid-rise Codes and Standards

WoodWorks

Learning Objectives

Learning Objective 1

Discuss the different types of shear walls commonly used in low-rise and mid-rise structures.

Learning Objective 2

Discuss complete load paths and the detailing required to maintain lateral load paths to shear walls.

Learning Objective 3

Review shear wall code requirements.

Learning Objective 4

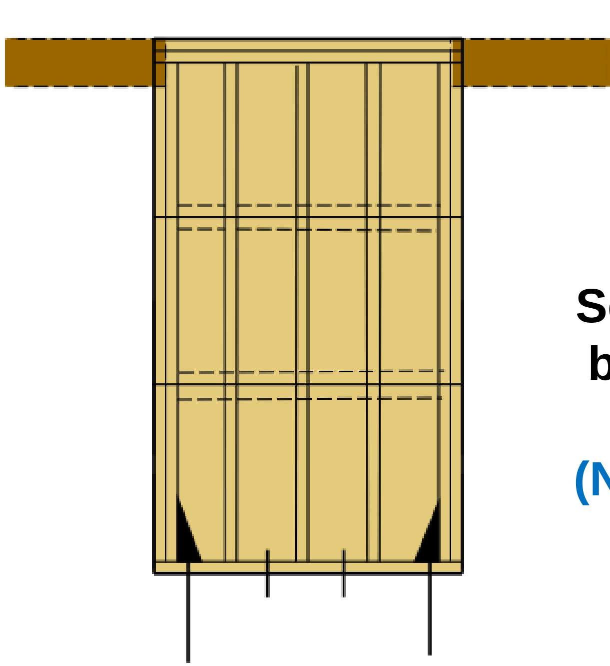
Discuss the methods available to analyze tall shear walls and discover their effects on mid-rise structures.


Continuous Load Paths?????
Interconnection ties?
Holes in diaphragm (web discontinuity)
Diaphragm boundary offset from shear wall (cantilever).
Irregular shaped shear wall with opening (discontinuous)
Large cantilevered diaphragm
Soft story
A quick note on residential projects

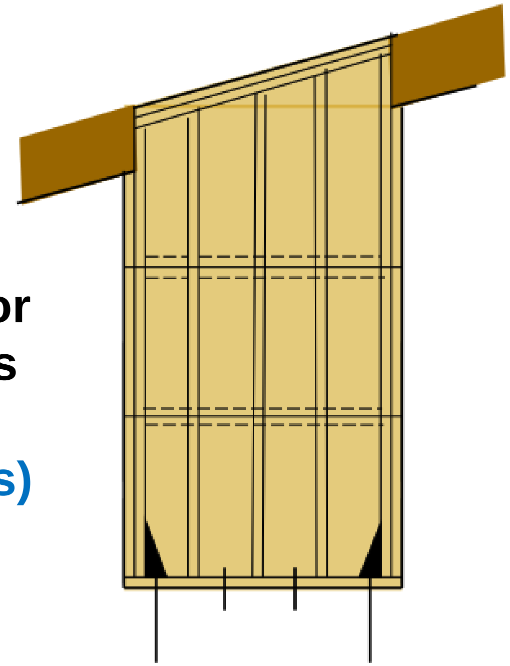
Presentation Topics

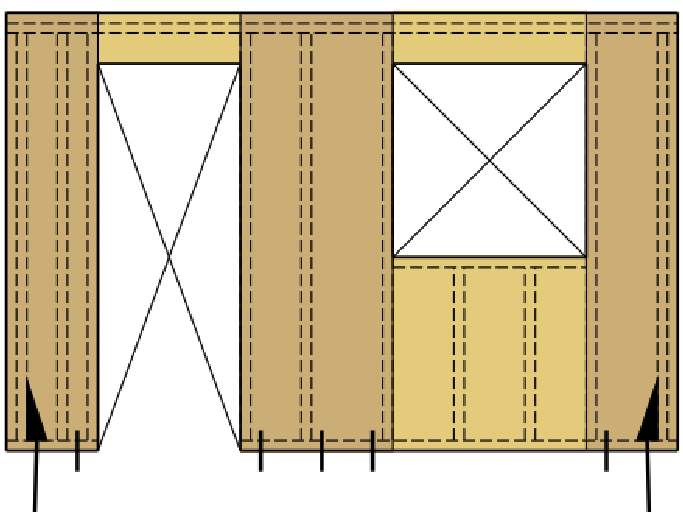
- **Shear Wall Types**
- Shear wall Anchorage
- SDPWS Code Requirements
- Complete Load Paths
- Offset Shear Walls and Relevant Code Requirements
- Mid-rise Diaphragms and Tall Shear Walls

Shear Wall Configuration Options

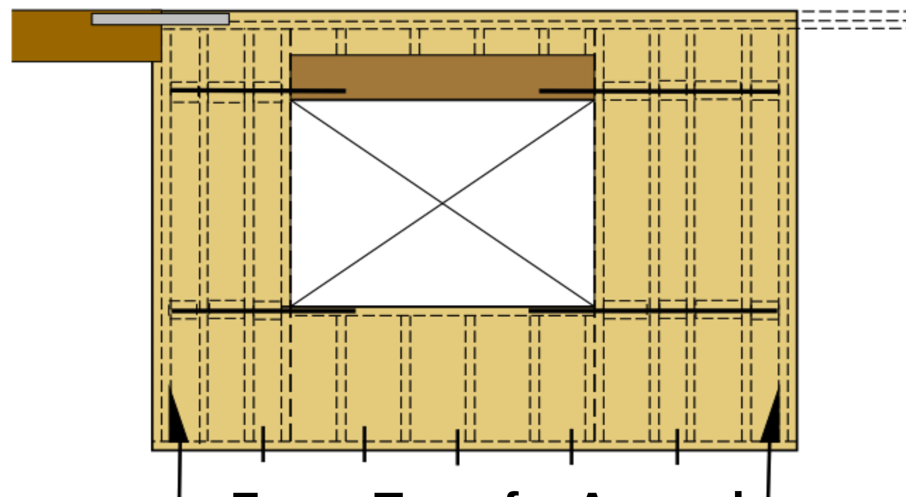


Segmented or braced Walls (IBC/IRC)
(No openings)



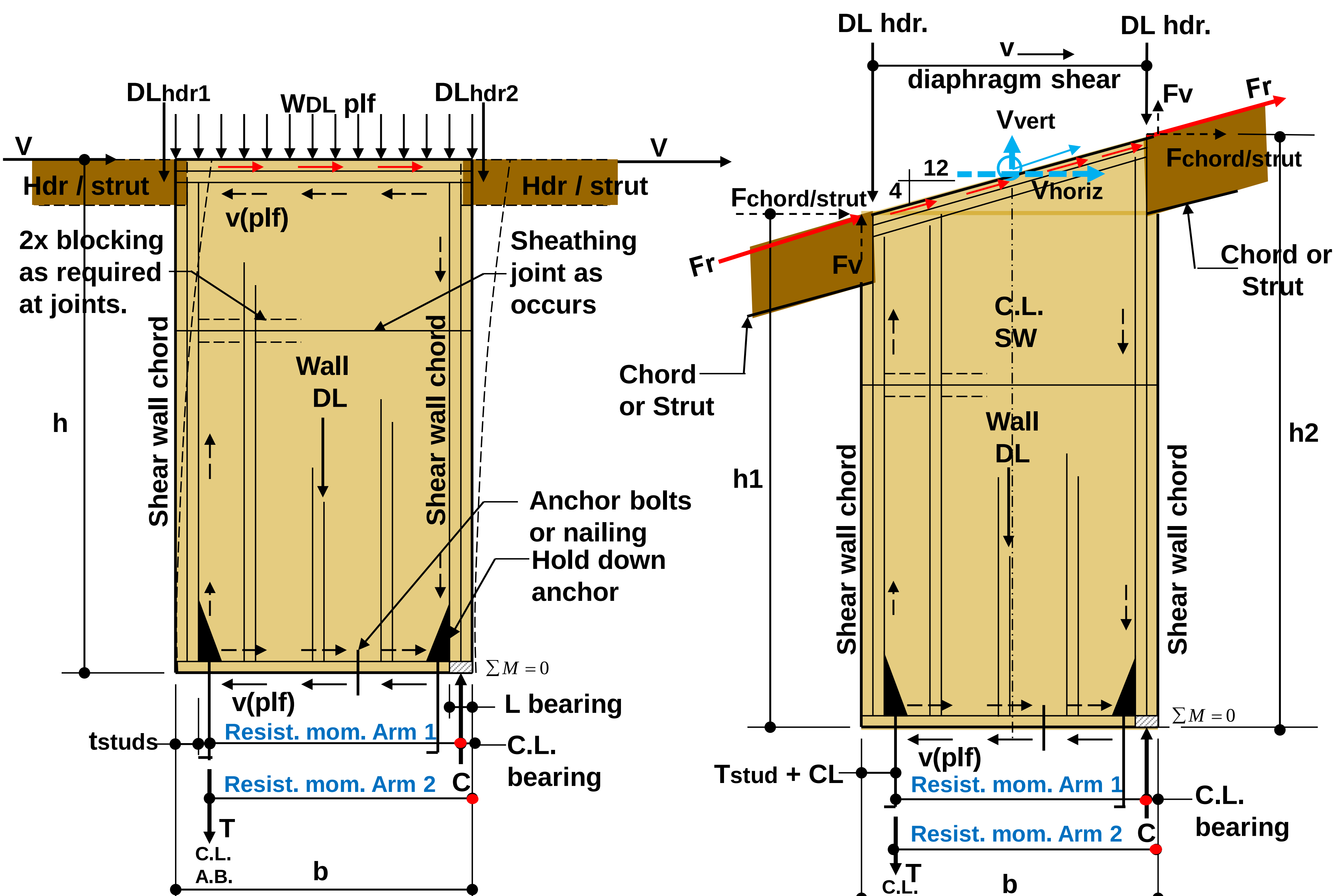


Perforated Walls



Force Transfer Around Openings Walls

Segmented Wall Types- Standard & Sloped



Segmented Shear Walls

Allowable Aspect Ratios & Adjustment Factors

AWC SDPWS Table 4.3.4-Maximum shear wall dimension ratios-Wind and seismic

C4.3.4.2

Type	Maximum height-width ratios
Wood structural panels – Unblocked	2: 1
Wood structural panels – Blocked	3.5:1
Particleboard – Blocked	2:1
Diagonal sheathing, conventional	2:1
Gypsum board	2:1(1)
Portland Cement Plaster	2:1 (1)
Structural Fiberboard	3.5:1

Footnotes

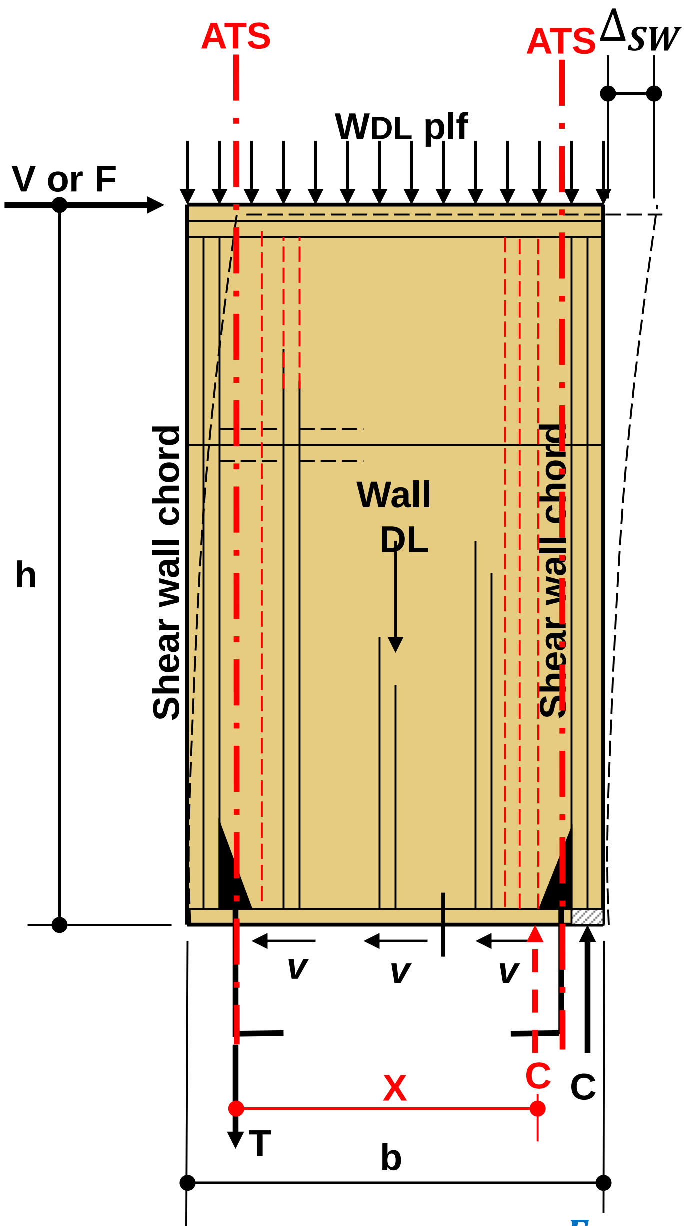
1. Walls having aspect ratios exceeding 1.5:1 shall be blocked shear walls.

4.3.4.2

For wood structural panel shear walls with aspect ratios (h/b) greater than 2:1, the nominal shear capacity shall be multiplied by the Aspect Ratio Factor (WSP) =1.25-0.125h/b.

For structural fiberboard shear walls with aspect ratios (h/b) greater than 1:1, the nominal shear capacity shall be multiplied by the Aspect Ratio Factor (fiberboard) =1.09-0.09 h/b.

Segmented Shear Wall Deflection and Stiffness



Wall stiffness k = $\frac{F}{\Delta}$

Traditional 4 term deflection equation

SDPWS combines

$$\Delta_{SW} = \underbrace{\frac{8vh^3}{EAb}}_{\text{Bending}} + \underbrace{\frac{vh}{G_v t_v}}_{\text{Shear}} + \underbrace{0.75he_n}_{\text{Nail slip}} + \underbrace{\frac{h\Delta_a}{b}}_{\text{Rod elongation (Wall rotation)}} \qquad \text{C4.3.2-1}$$

SDPWS 3 term deflection equation

$$\Delta_{SW} = \underbrace{\frac{8vh^3}{EAb}}_{\text{Bending}} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad \text{4.3-1}$$

Vertical elongation
- Device elongation
- Rod elongation

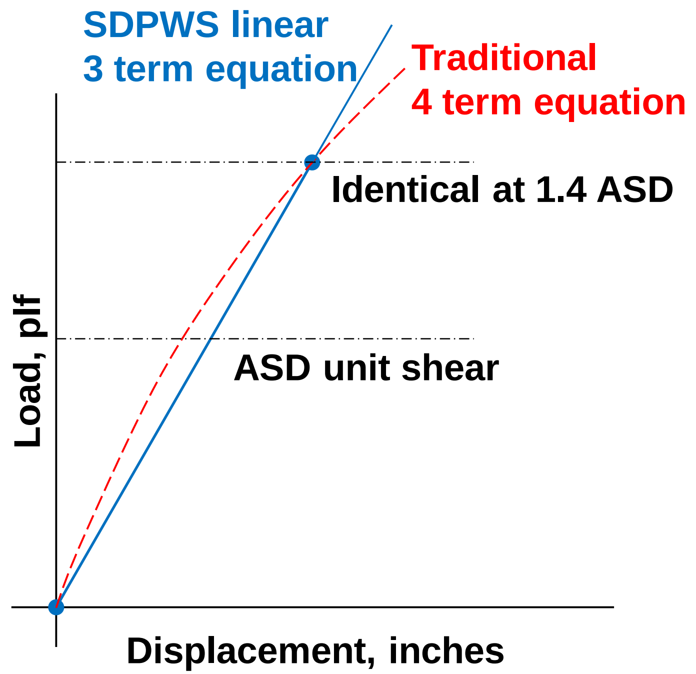
Apparent shear stiffness
- Nail slip
- Panel shear deformation

Deflection of unblocked segmented shear wall
- Use Eq.4.3-1 with v/Cub per 4.3.2.2 and Table 4.3.3.2

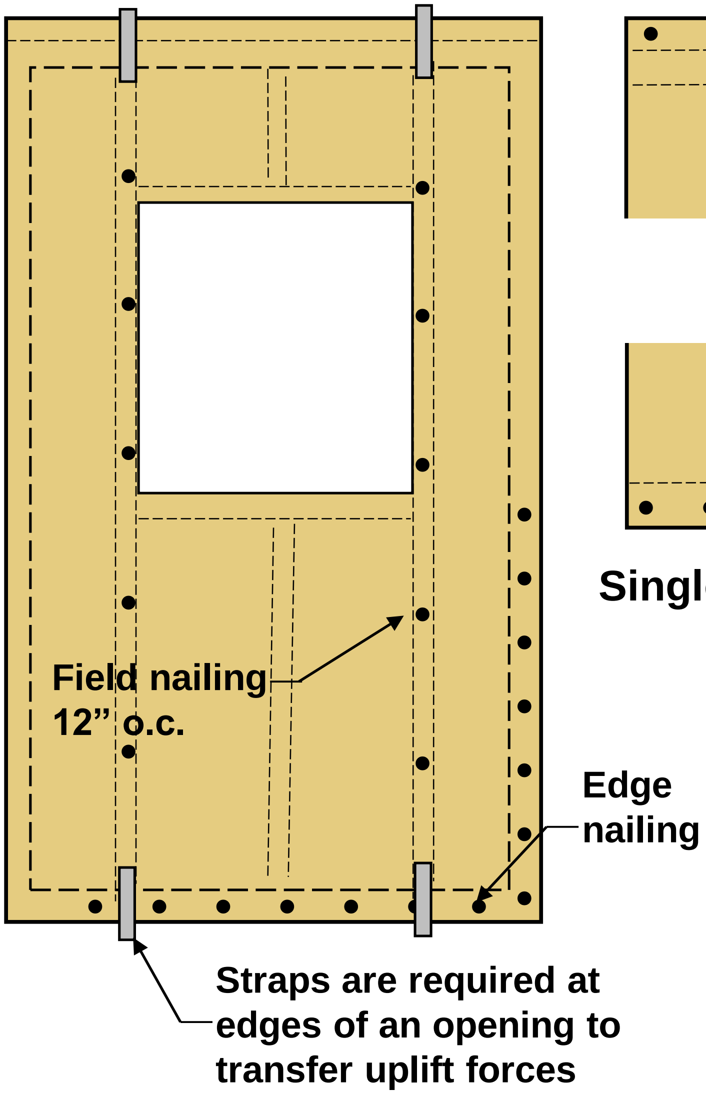
$$\Delta_{sw} = \frac{8\left(\frac{v}{C_{ub}}\right)h^3}{EAB} + \frac{\left(\frac{v}{C_{ub}}\right)h}{1000G_a} + \frac{h}{b}\Delta_a$$

- Max. height unblocked=16 feet

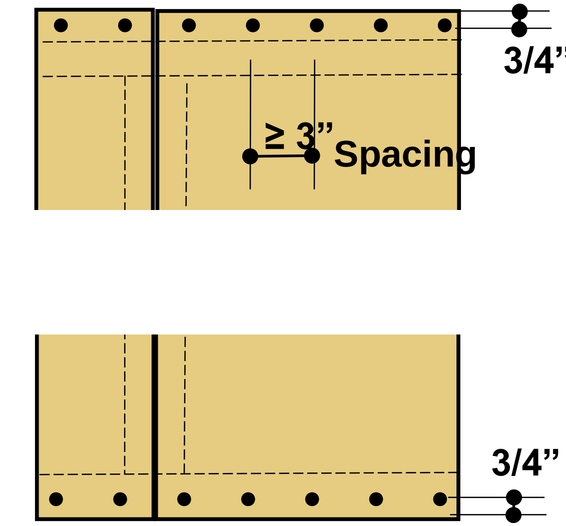
Segmented Shear Wall Deflection



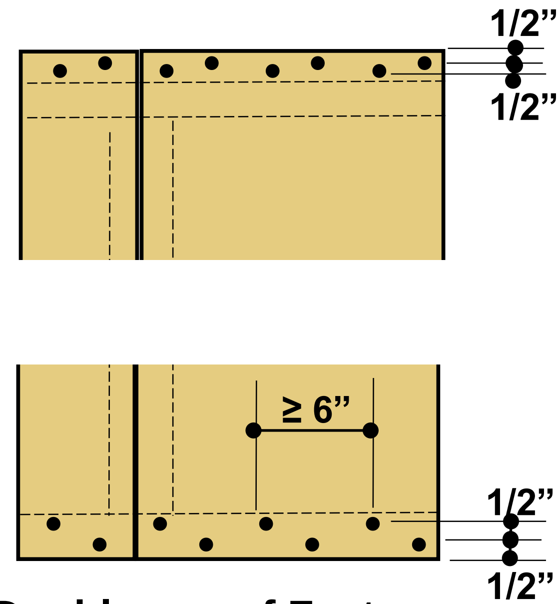
Combined Shear and Uplift Shear Walls-WSP-APA SR-101C



Typical Panel Fastening



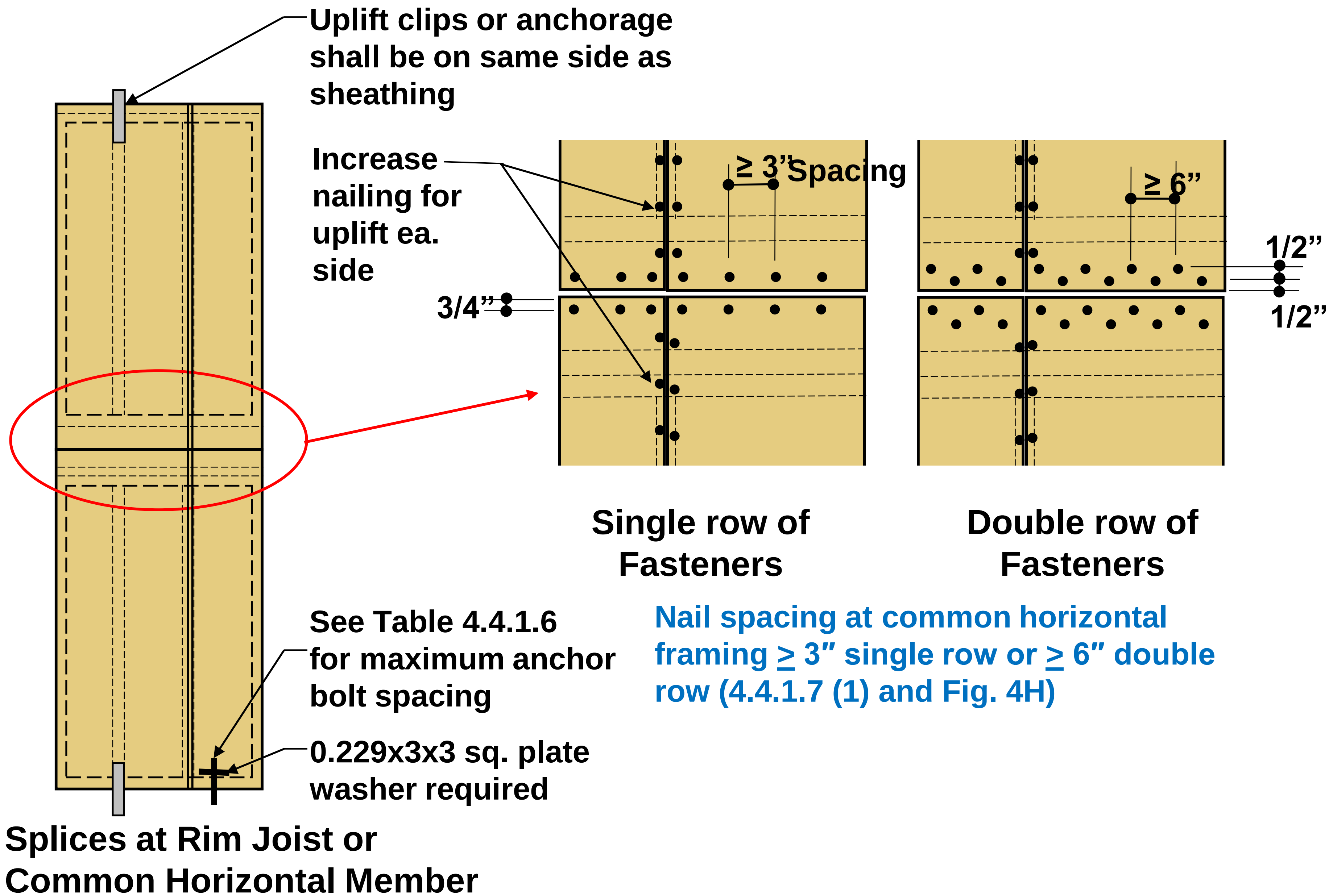
Single row of Fasteners



Double row of Fasteners

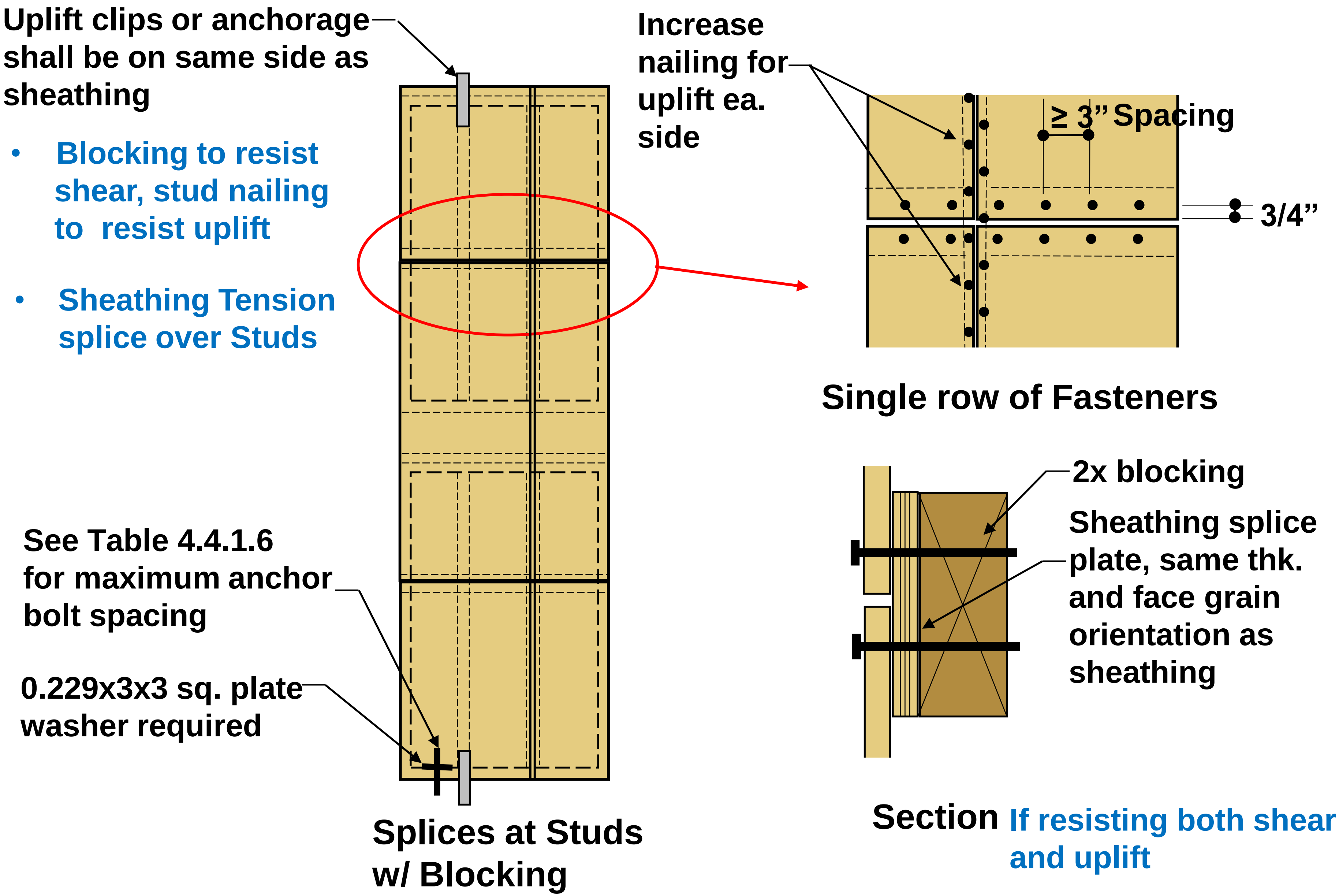
- Min. panel thickness=7/16”
- Sheathing can be applied vertically or horiz.
- Min. fastener spacing=3” single row, 6” Dbl. row
- All horiz. edges shall be blocked
- Applies to all shear wall types
- Capacity: Table 4.4.1-shear + uplift
  Table 4.4.2-uplift only

  VASD=Vn/2, Vstr=0.65Vn



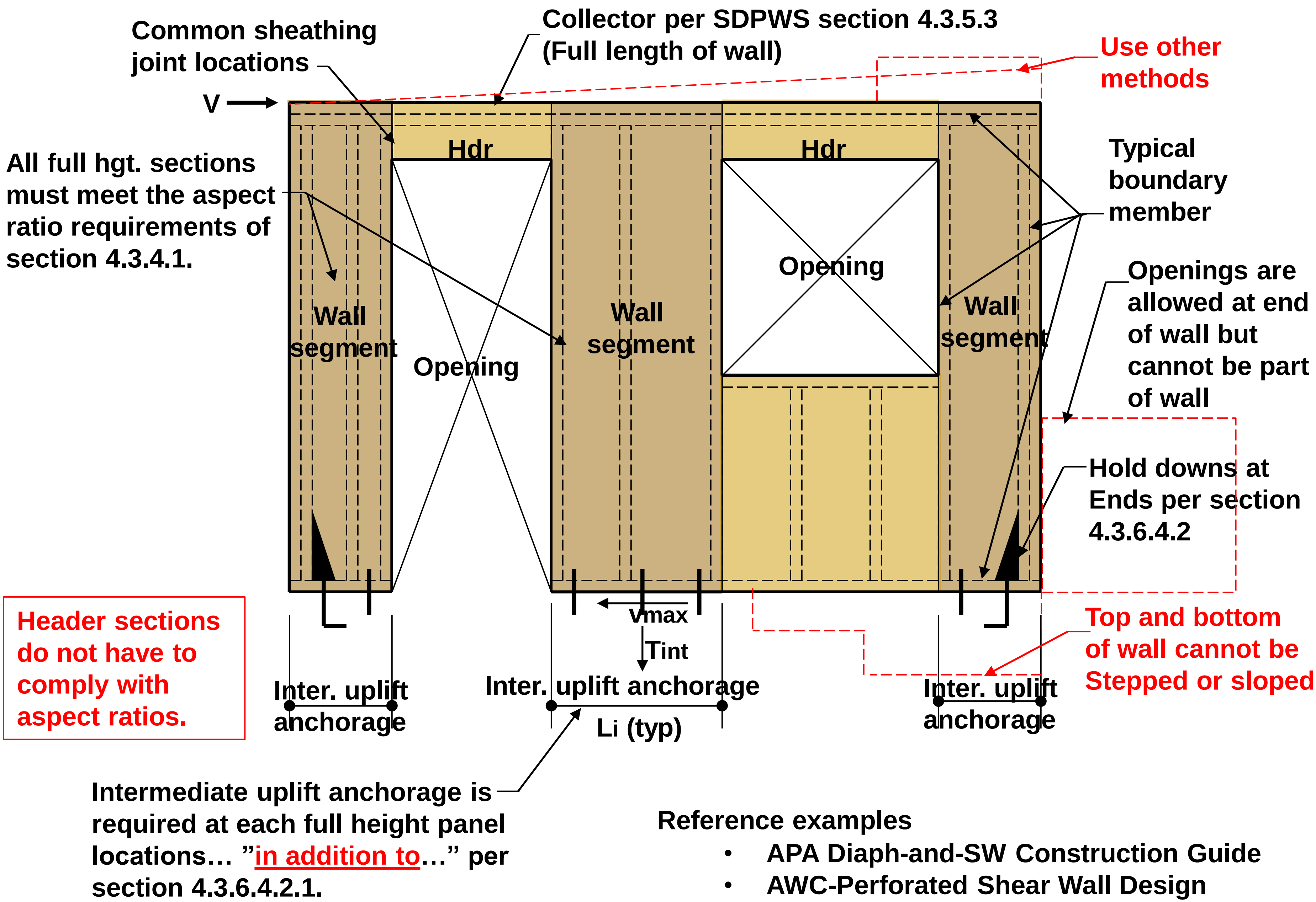
Nail spacing at common horizontal framing ≥ 3″ single row or ≥ 6″ double row (4.4.1.7 (1) and Fig. 4H)

No Horizontal Sheathing Joint Over Studs



Horizontal Sheathing Joint Over Studs

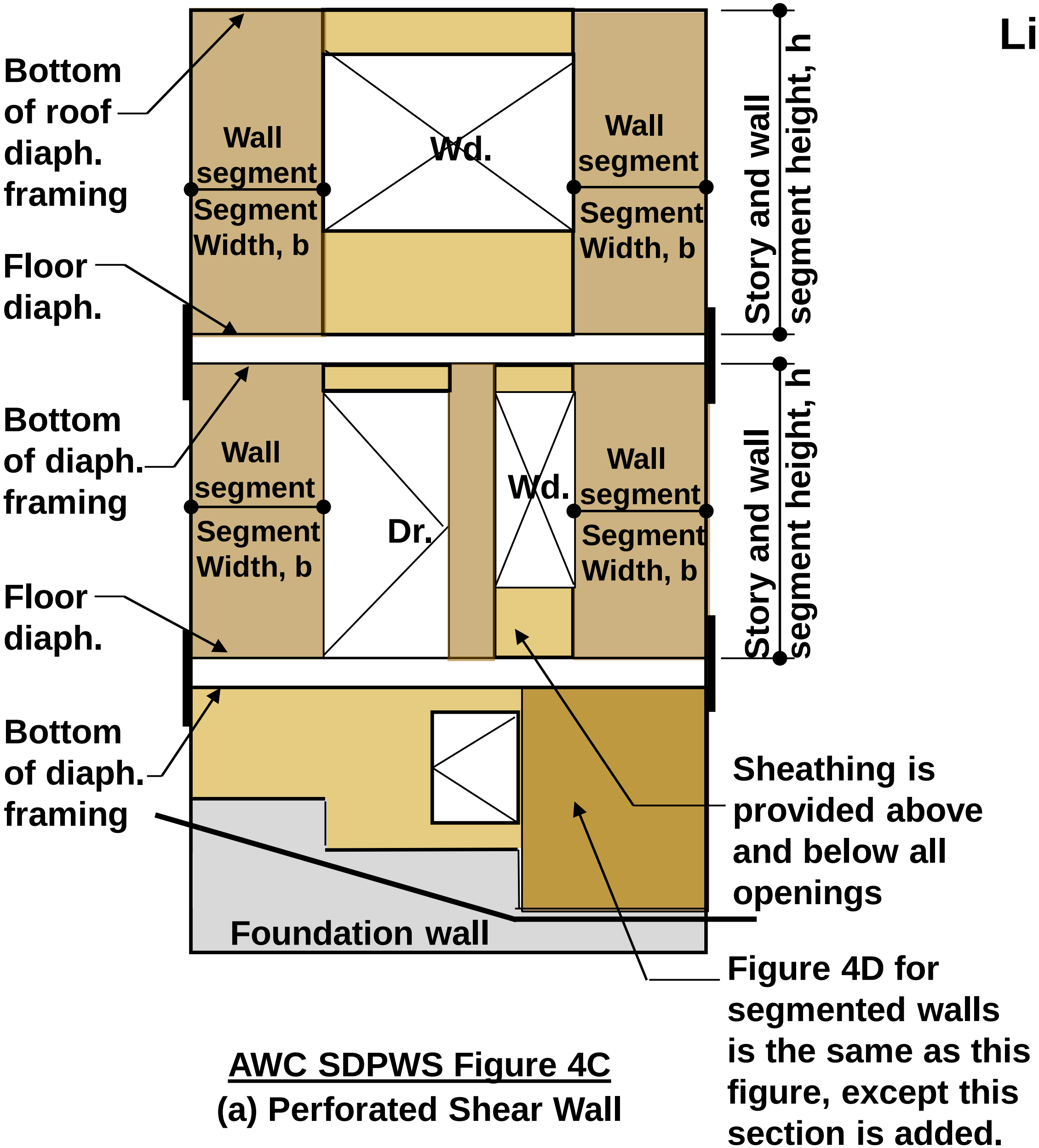
Perforated Shear Walls- Empirical Design



Reference examples

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

Allowable Perforated Shear Wall Aspect Ratios

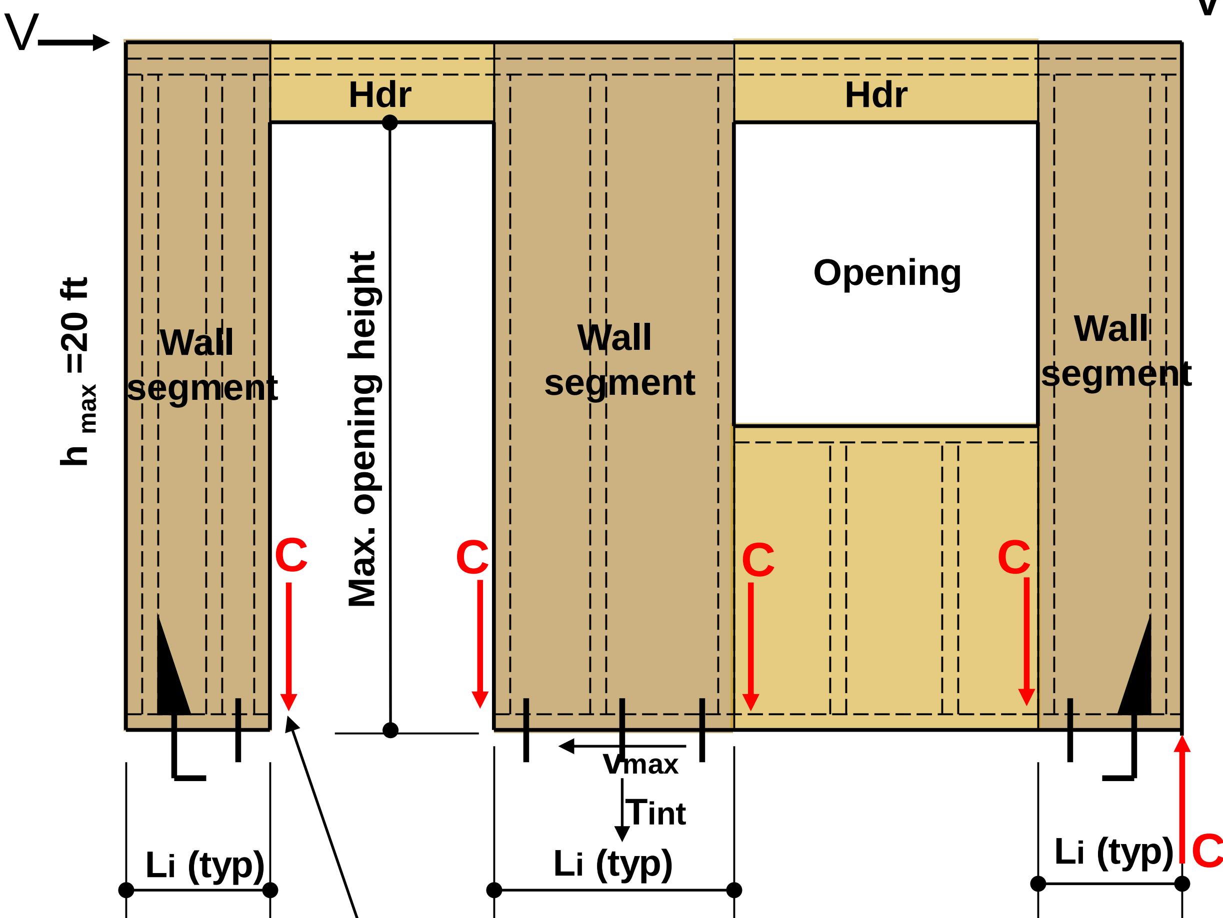


<u>AWC SDPWS Figure 4C</u>
(a) Perforated Shear Wall

Limitations:

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

<u>Empirically</u> determined shear resistance reduction factors, C_0 – based on maximum opening height and percentage of full height wall segments.



AWC Section 4.3.6.1.3
Each end of each segment shall be designed for the compression force C.

$V_{allow} = (v_{Tabular}) \times (C_O) \times \sum Li$

Where:

$$C_0 = \left(\frac{r}{3-2r}\right)\frac{L_{tot}}{\Sigma L_i}$$ **Per SDPWS Table 4.3.3.5**

$$r = \frac{1}{1+\frac{A_0}{h\Sigma L_i}}$$ **Sheathing area ratio**

A_0 = total area of openings

Maximum Shear @ full hgt. sect.

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

Hold down at ends

$$T = C = \frac{Vh}{C_0 \Sigma L_I}$$

At full-hgt. segments

$$v_{max} = T_{int} = \frac{V}{C_0 \Sigma L_i}$$

Sugiyama Method-Reduced shear capacity is multiplied by the full length of the wall.

IBC/SDPWS Method- Reduced shear capacity is multiplied by the sum of the lengths of the full height sections (slightly more conservative).

Table 4.3.3.5 Shear Capacity Adjustment Factor, C_o

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

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

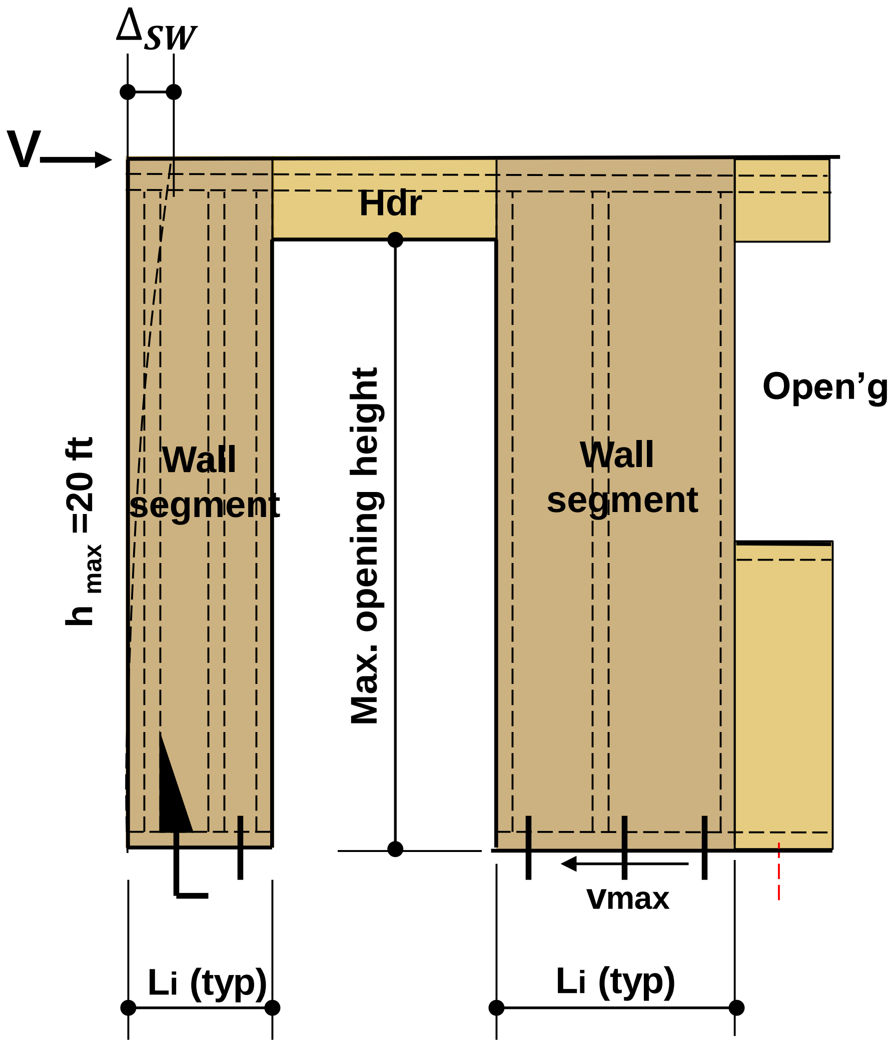
2 The sum of the perforated shear wall segment lengths, $\sum L_i$, divided by the total length of the perforated shear wall.

4.3.4.3

In the design of perforated shear walls, the length of each perforated shear wall segment with an aspect ratio greater than 2:1 shall be multiplied by 2b/h for the purposes of determining Li and ∑Li. The provisions of Section 4.3.4.2 and the exception to Section 4.3.3.4.1 (1.25-0.125h/b) shall not apply to perforated shear wall segments.

Where perforated shear walls have WSP on 1 side and GWB on the opposite side, the combined shear capacity shall be in accordance with the provisions of Section 4.3.3.3.2.

Perforated Shear Wall Deflection



Deflection of perforated shear wall

Vmax ∑Li

$$\Delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad \text{4.3-1}$$

Bending

Vertical elongation

- Strap nail slip
- Device elongation
- Rod elongation

Apparent shear stiffness

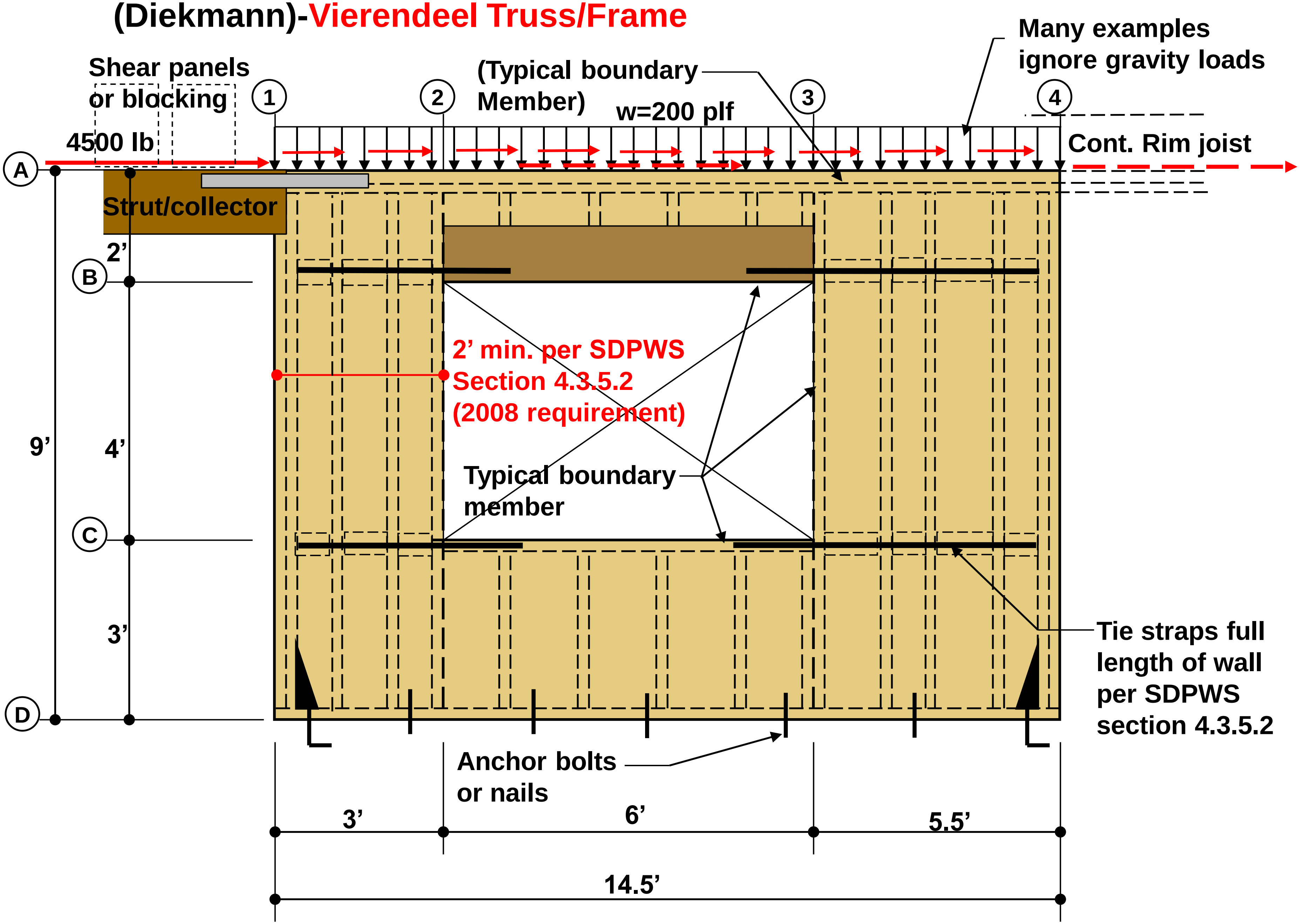
- Nail slip
- Panel shear deformation

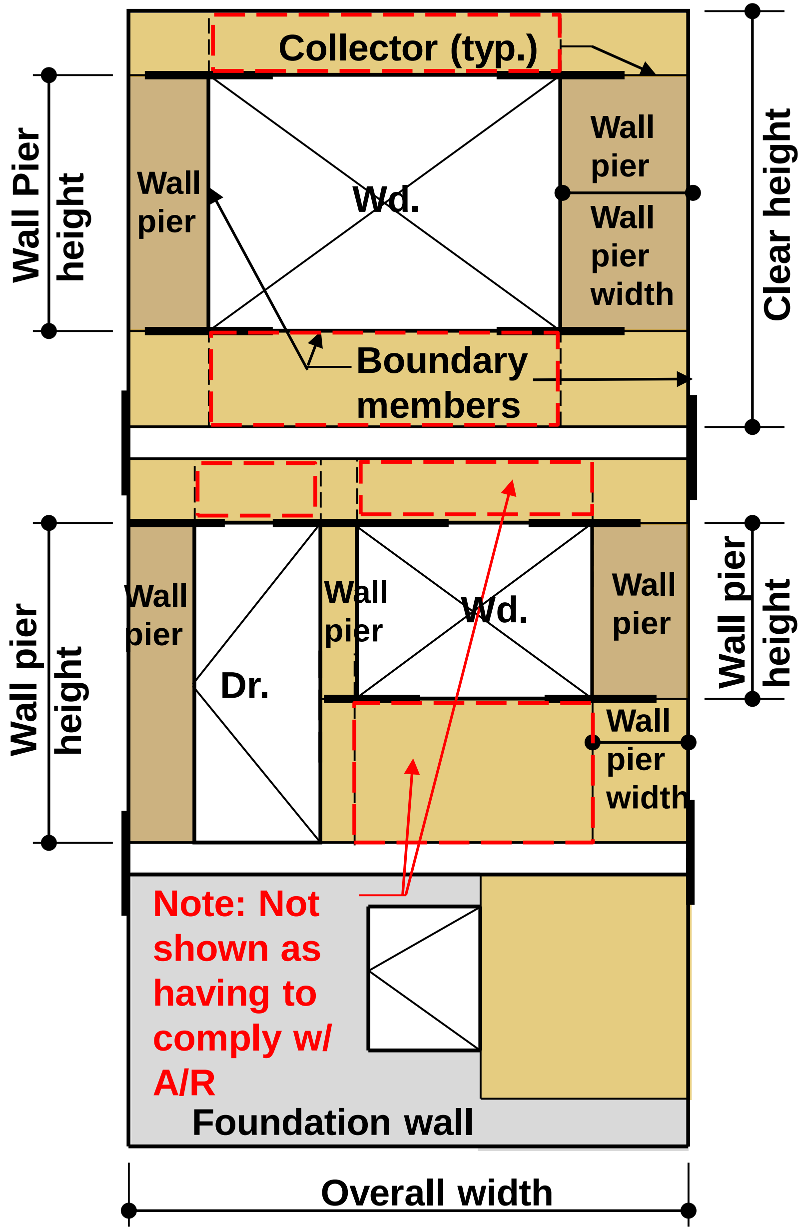
4.3.2.1

Deflection of Perforated Shear Walls: The deflection of a perforated shear wall shall be calculated in accordance with 4.3.2, where v in equation 4.3-1 is equal to vmax obtained in equation 4.3-9 and b is taken as ∑Li.

FTAO Shear Walls (See recent Testing-APA Form M410 and SR-105)

(Diekmann)-Vierendeel Truss/Frame



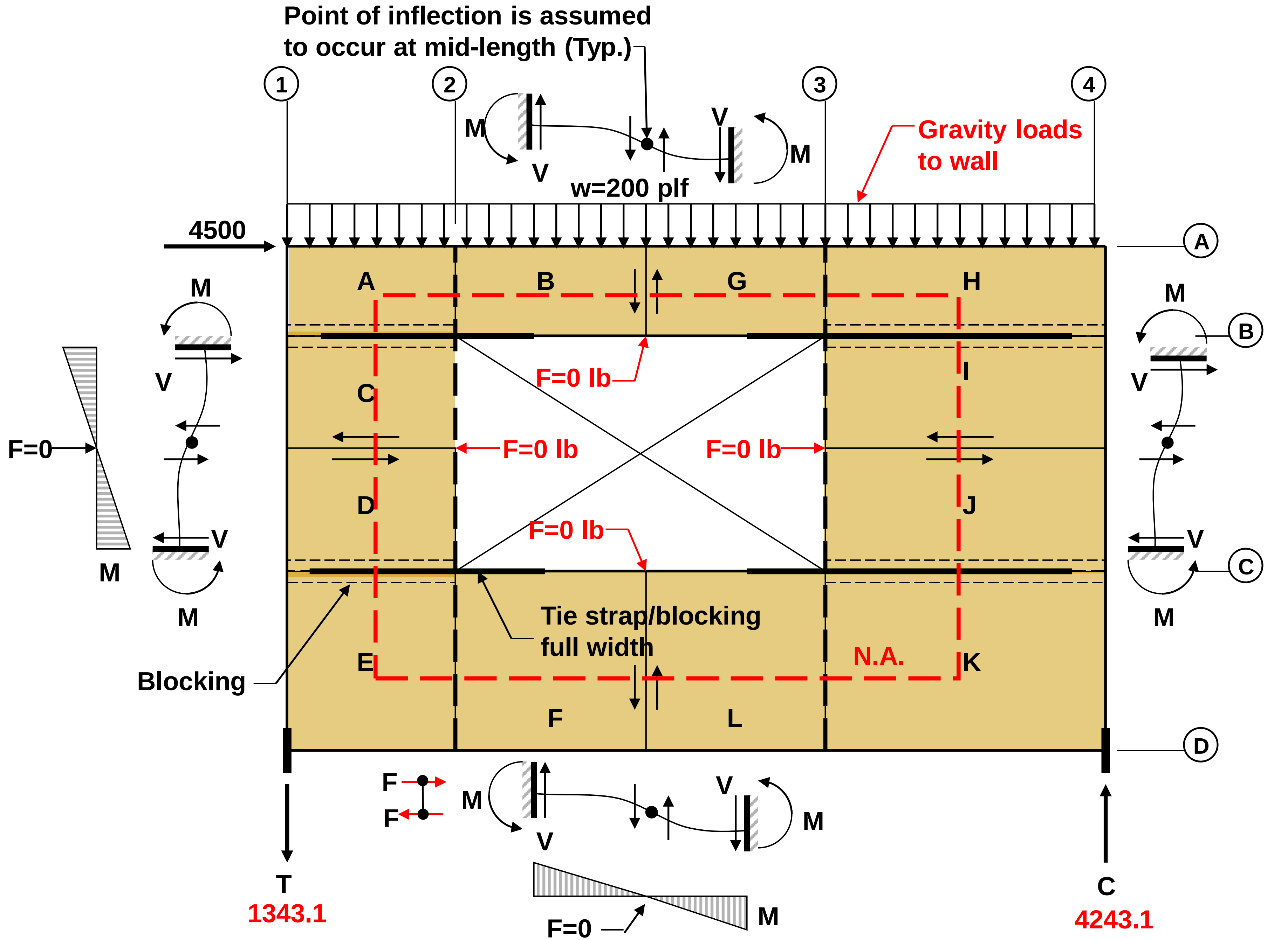


AWC SDPWS Figure 4E
(b) Force Transfer Around Opening

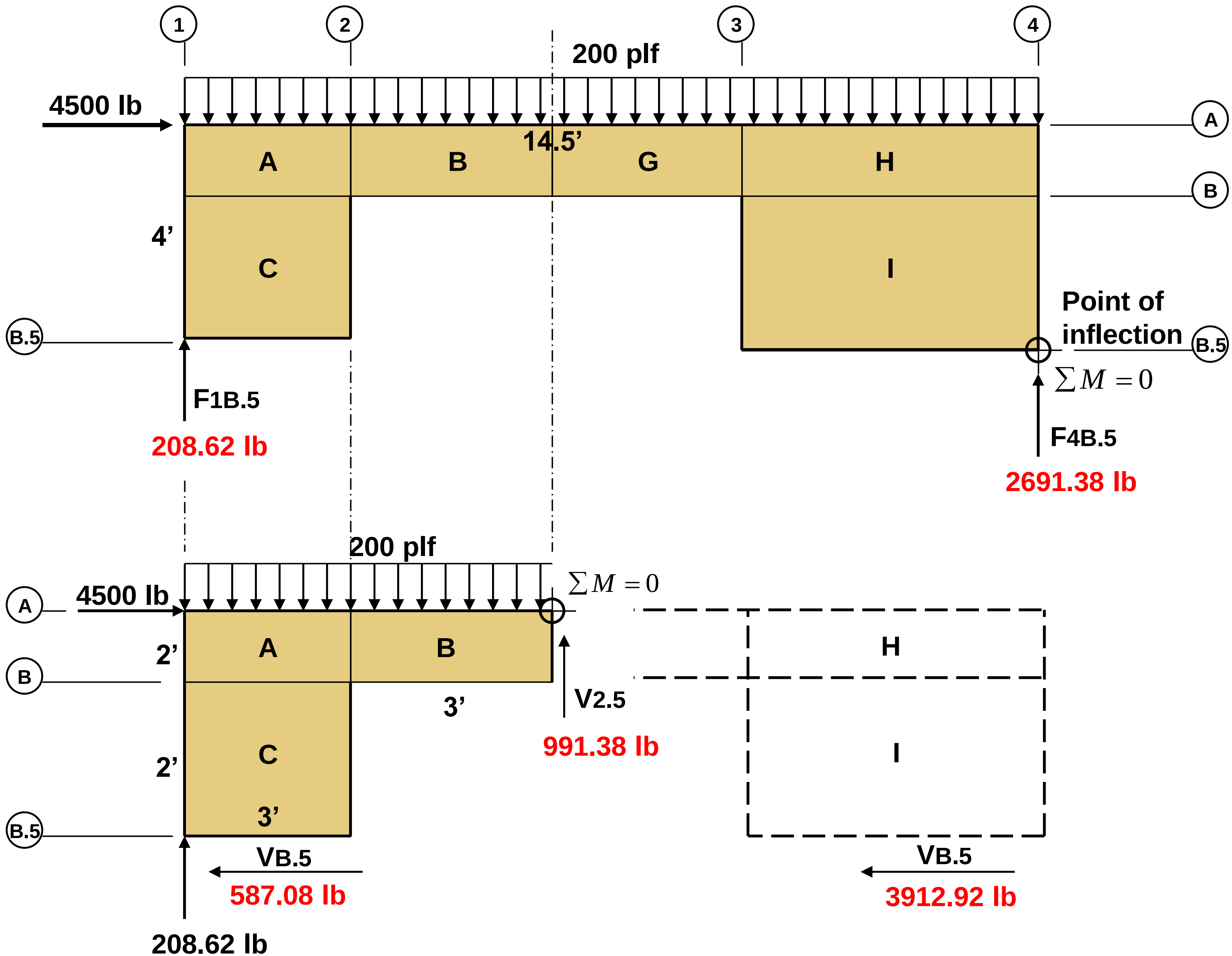
Limitations:

- The aspect ratio limitations of Table 4.3.4 shall apply to the overall wall and the pier sections on each side of the openings
- Sections exceeding 3.5:1 aspect ratio shall not be considered a part of the wall.
- Minimum pier width=2'-0".
- A full height pier section shall be located at each end of the wall.
- Where a horizontal offset occurs, portions on each side of the offset shall be considered as separate FTAO walls.
- Collectors for shear transfer shall be provided through the full length of the wall.

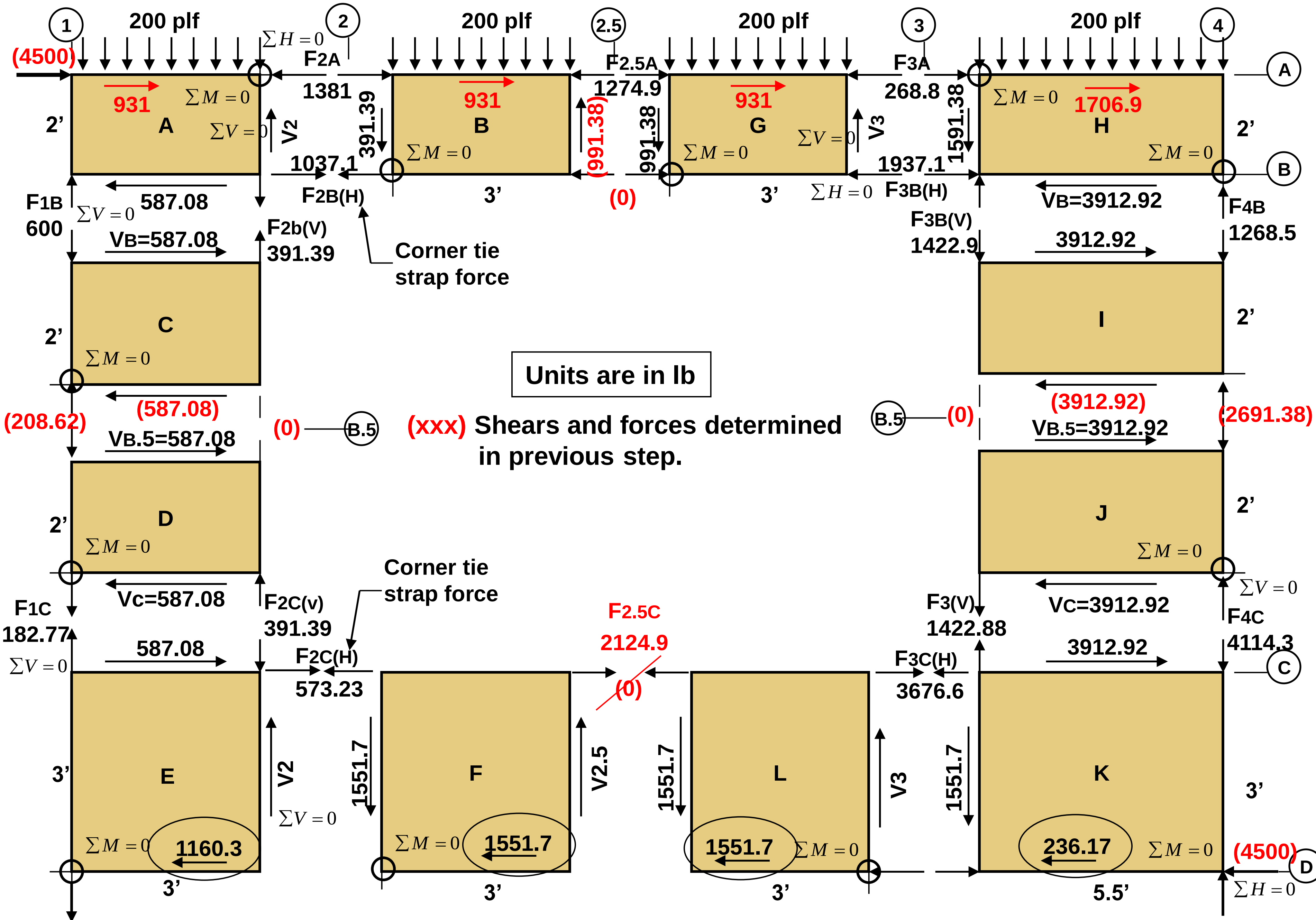
Allowable Shear Wall Aspect Ratios For FTAO Shear Walls



Force Transfer Methodology (Diekmann)-Vierendeel Truss/Frame



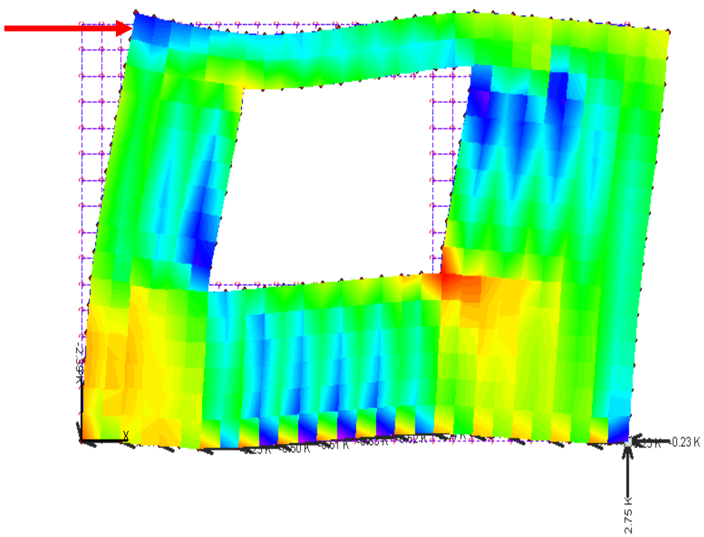
Free-body of Upper Half and Upper Left Section



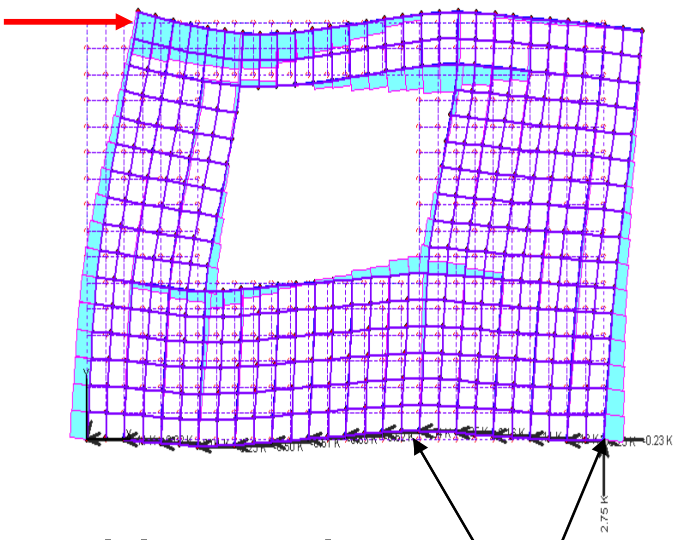
Resultant Forces on Wall Segments

FEA Results

Notice inflection point and frame like action



Shear Stresses



Axial Force Diagram

Model was based on anchor bolts at 2’ o.c. and hold downs at each end.

Advancements in FTAO Shear Wall Analysis

(See recent Testing-APA Form M410)

2015 SEAOC CONVENTION PROCEEDINGS

Advancements in Force Transfer Around Openings for Wood Framed Shear Walls

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Abstract

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

Asymmetric piers, multiple openings, and C-shaped sheathing were investigated and rational design methodologies in accordance with the International Building Code (and California Building Code) have been created. A new *APA System Report, SR-105: Design for Force Transfer Around Openings*, is under development and will guide users on the application of this new methodology to provide a structurally sound and efficient shear wall system. This paper provides insight to the creation of the System Report, examples of its application, and how to best utilize this method in wood framed construction. This research was supported in part by funds provided by the USDA Forest Products Laboratory, which is acknowledged and greatly appreciated by the project team.

Introduction

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

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

Shear Wall Design Challenges

Proper design of wood shear walls is a critical component of the lateral force resisting system. The California Building Code (based on the 2012 International Building Code) refers the engineer to the Special Design Provisions for Wind and Seismic (SDPWS-08 Section 4.3.5) for three design methods: individual full-height wall segments (segmented), force-transfer shear walls (FTAO), and perforated shear walls (perforated). Each method has benefits and challenges. Most structural engineers use the first approach, segmented, where feasible. Segmented design utilizes wood structural panels (WSP, being either plywood or Oriented Strand Board [OSB]) in full-height segments without any penetrations included (Figure 1). The WSPs transfer the shear, anchor

1

- SEAOC Convention 2015 Proceedings

- Basis of APA System Report SR-105 (in development)

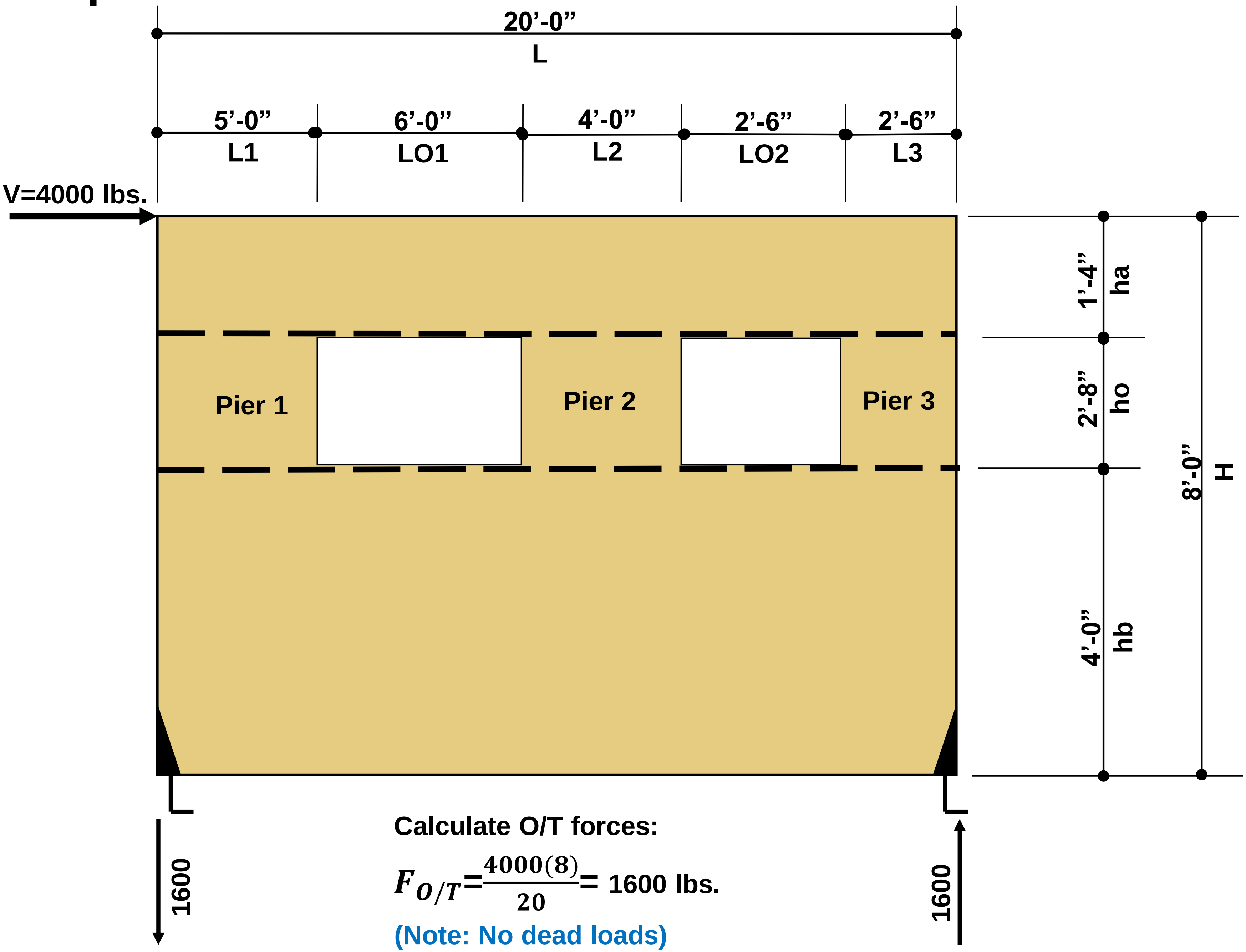
Advancements in Force Transfer Around Openings for Wood Framed Shear Walls

APA System Report SR-105
(Report pending)

Refine rational design methodologies to match test results

- **Used test results from full-scale wall configurations**
- **Analytical results from a computer model**
- **Allows asymmetric piers and multiple openings.**

Example Problem



Calculate O/T forces:

$$F_{O/T} = \frac{4000(8)}{20} = 1600 \text{ lbs.}$$

(Note: No dead loads)

Calculate:

- Aspect ratios of pier sections

$$Pier\ 1 = \frac{h}{d} = \frac{2.67}{5} = 0.534 < 2{:}1$$

$$Pier\ 2 = \frac{2.67}{4} = 0.668 < 2{:}1$$

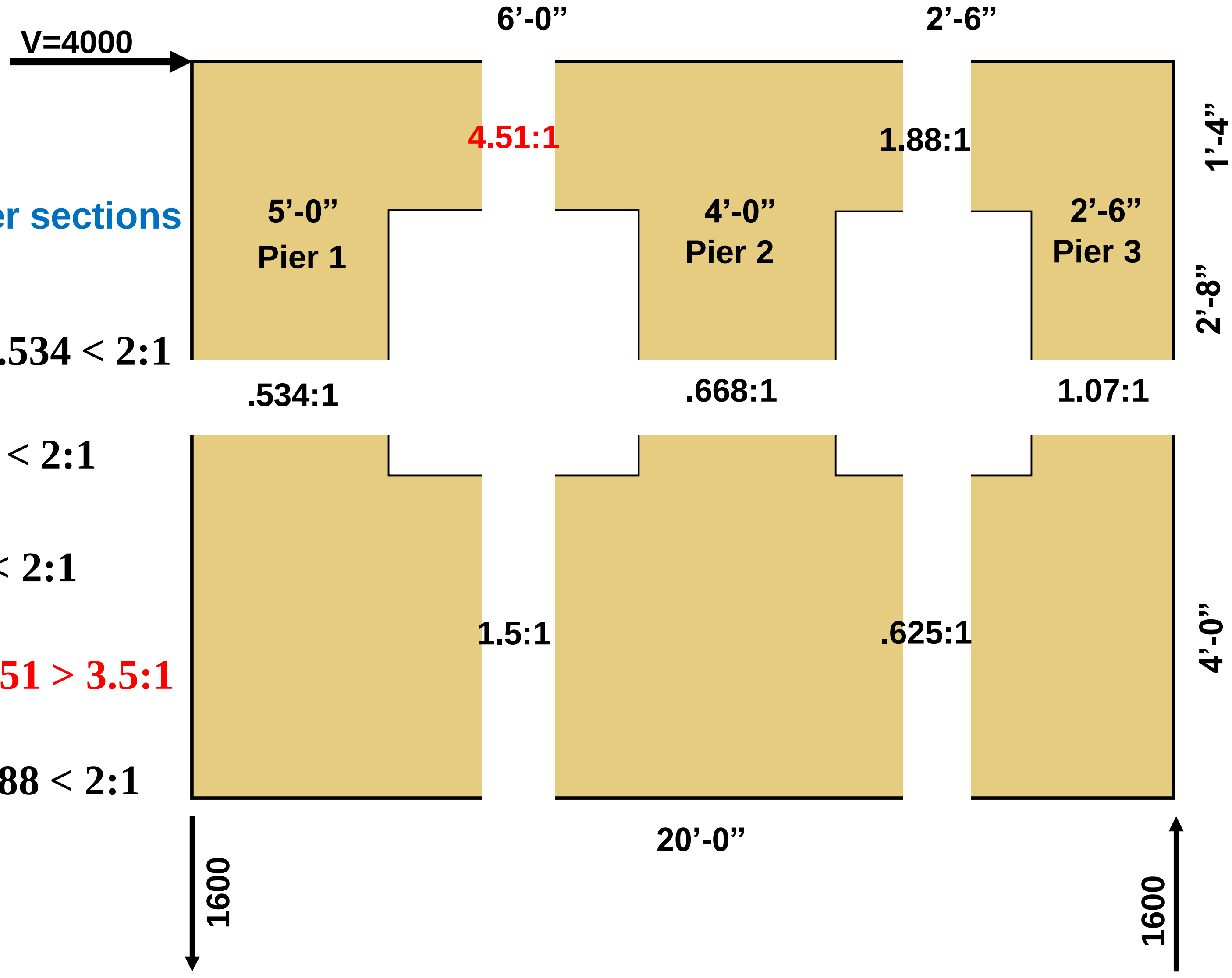
$$Pier\ 3 = \frac{2.67}{2.5} = 1.07 < 2{:}1$$

$$Header\ 1 = \frac{6}{1.33} = 4.51 > 3.5{:}1$$

$$Header\ 2 = \frac{2.5}{1.33} = 1.88 < 2{:}1$$

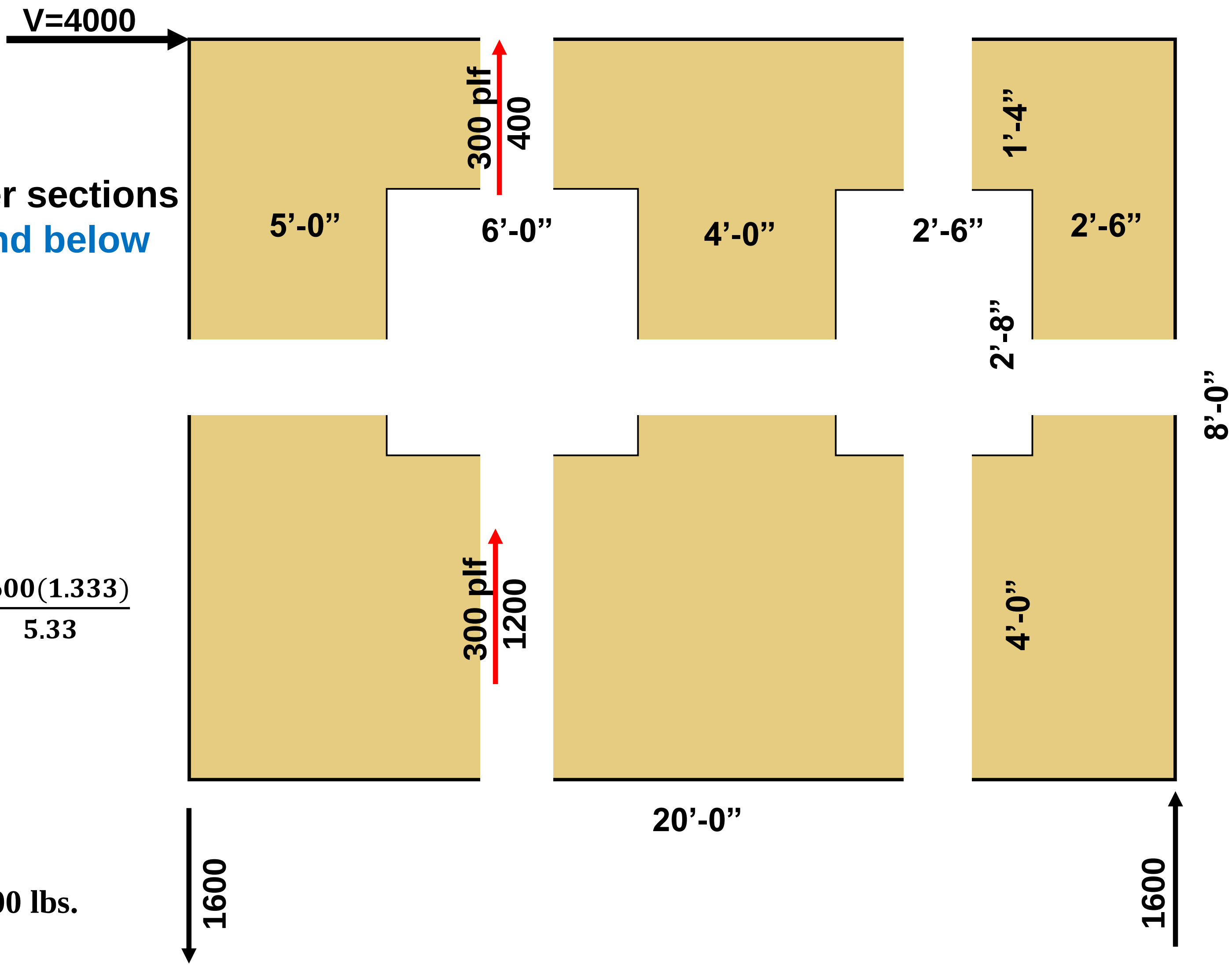
$$Sill\ 1 = \frac{6}{4} = 1.5 < 2{:}1$$

$$Sill\ 2 = \frac{2.5}{4} = 0.625 < 2{:}1$$



Calculate:

- **Aspect ratios of pier sections**
- **Unit shear above and below opening**



$$Vhdr1 = \frac{F_{o/t}(h_a)}{(h_a+h_b)} = \frac{1600(1.333)}{5.33}$$
$$= 400 \text{ lbs.}$$

$$\text{vhdr2} = \frac{400}{1.33} = 300 \text{ plf}$$

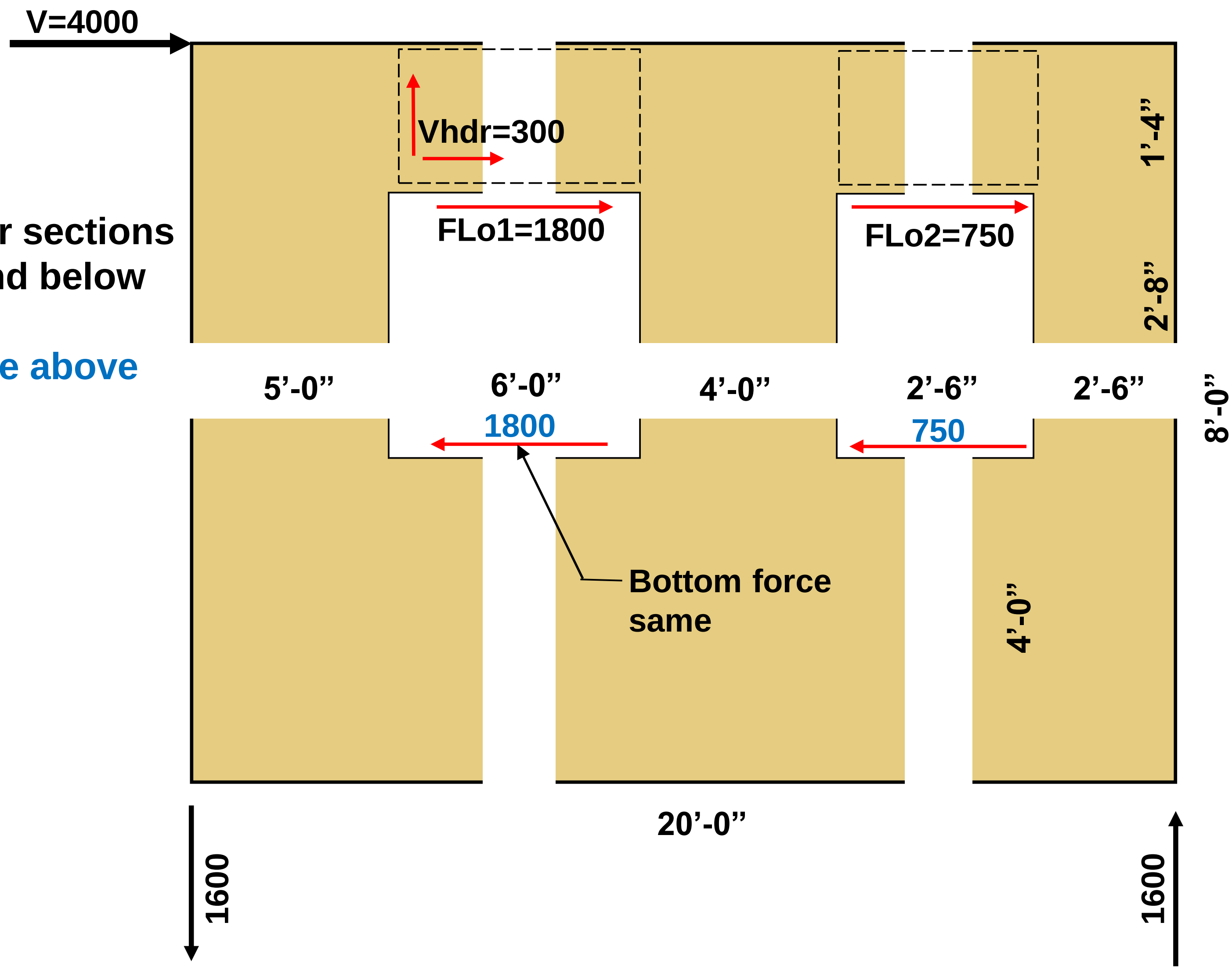
$$Vsill1 = \frac{1600(4)}{5.33} = 1200 \text{ lbs.}$$

$$\text{vsill2} = \frac{1200}{4} = 300 \text{ plf}$$

Method assumes the unit shear above and below opening are equivalent

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening



V*hdr* = 300 plf

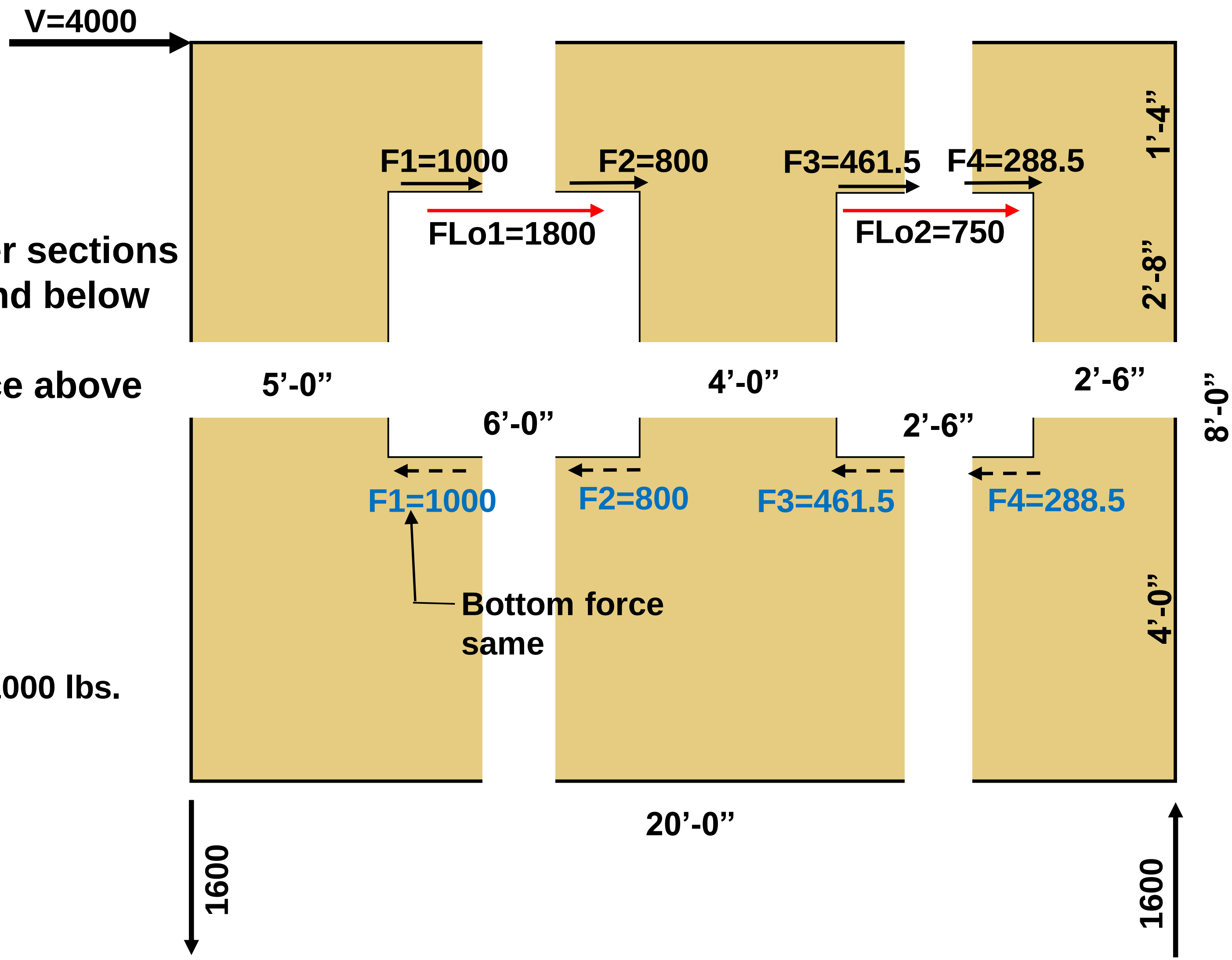
F*Lo1* = 300(6) = 1800 lbs.

F*Lo2* = 300(2.5) = 750 lbs.

The total boundary force above and below the opening = the unit shear above and below the opening multiplied by the opening length

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces



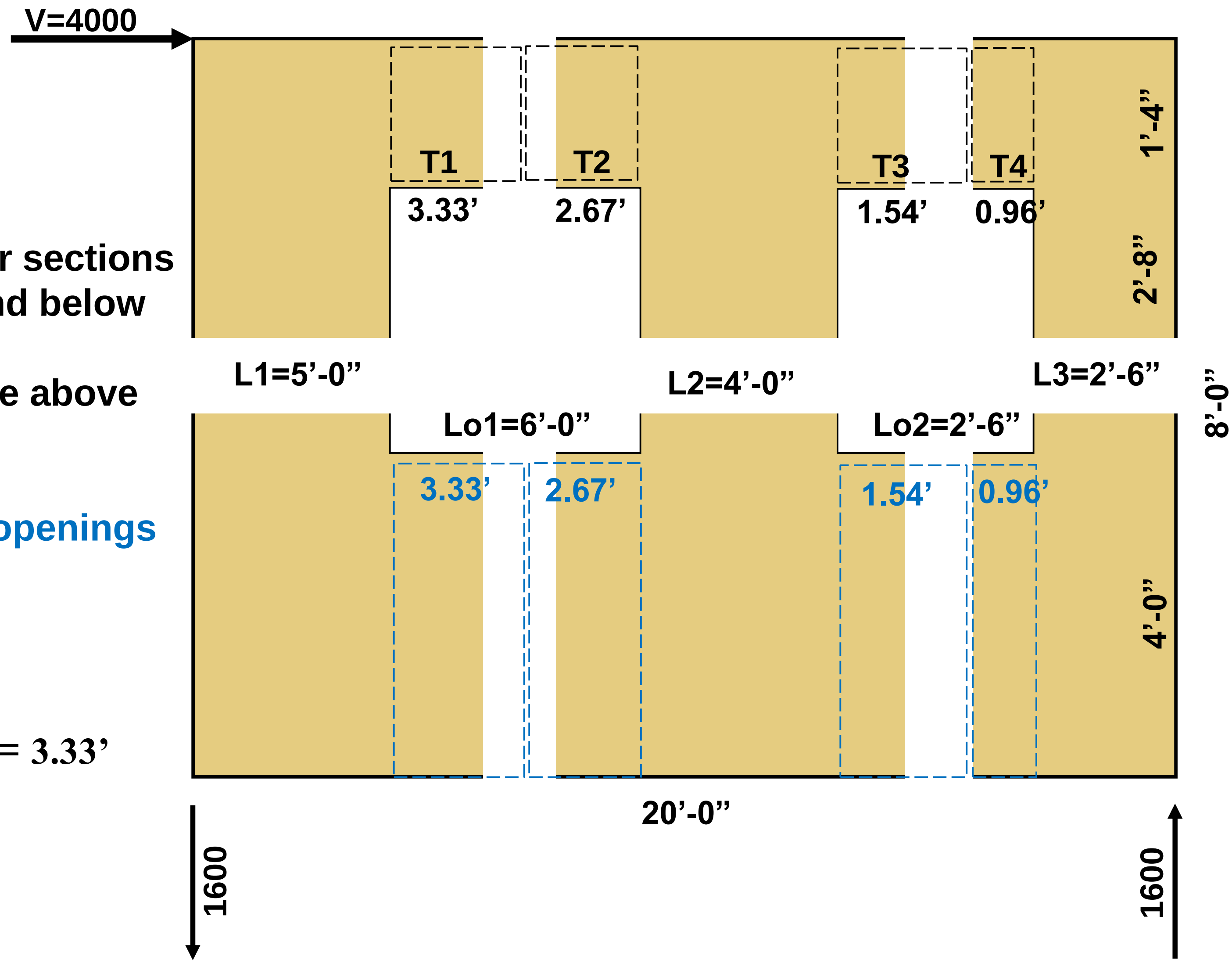
$$F1 = \frac{FLo1(L1)}{(L1+L2)} = \frac{1800(5)}{(5+4)} = 1000 \text{ lbs.}$$

$$F2 = \frac{1800(4)}{(5+4)} = 800 \text{ lbs.}$$

$$F3 = \frac{750(4)}{(4+2.5)} = 461.50 \text{ lbs.}$$

$$F4 = \frac{750(2.5)}{(4+2.5)} = 288.5 \text{ lbs.}$$

Corner forces are based on the boundary forces above and below the opening and only the pier sections adjacent to that opening



Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings

$$T1 = \frac{L1(Lo1)}{(L1+L2)} = \frac{5(6)}{(5+4)} = 3.33'$$

$$T2 = \frac{4(6)}{(5+4)} = 2.67'$$

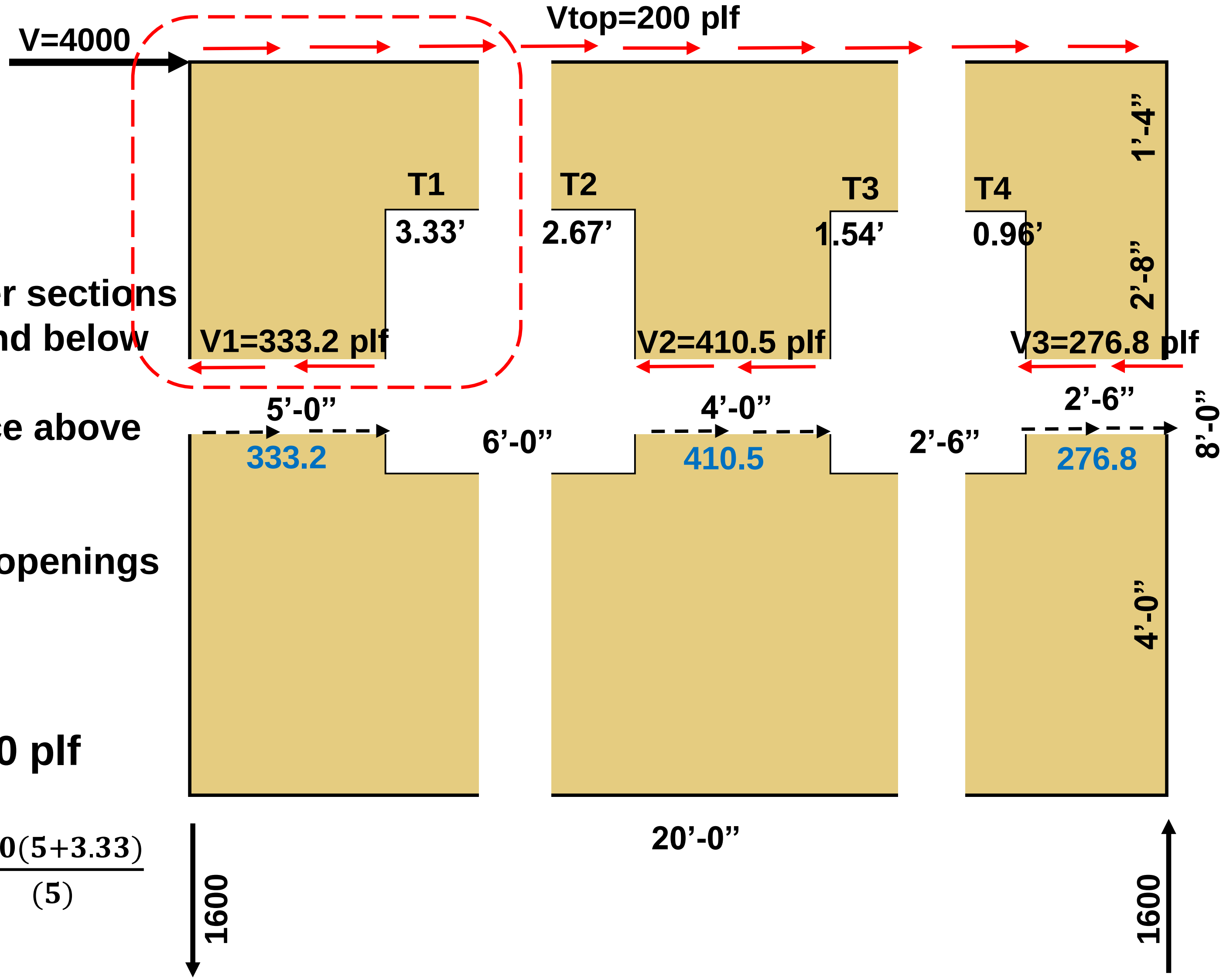
$$T3 = \frac{4(2.5)}{(4+2.5)} = 1.54'$$

$$T4 = \frac{2.5(2.5)}{(4+2.5)} = 0.96'$$

The tributary length of the opening is the ratio of the pier length multiplied by the opening length divided by the sum of the lengths of the piers on each side of the opening

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings
- Unit shear in piers



$$Vtop = \frac{V}{L} = \frac{4000}{20} = 200 \text{ plf}$$

$$V1 = \frac{vtop(L1+T1)}{L1} = \frac{200(5+3.33)}{(5)} = 333.2 \text{ plf}$$

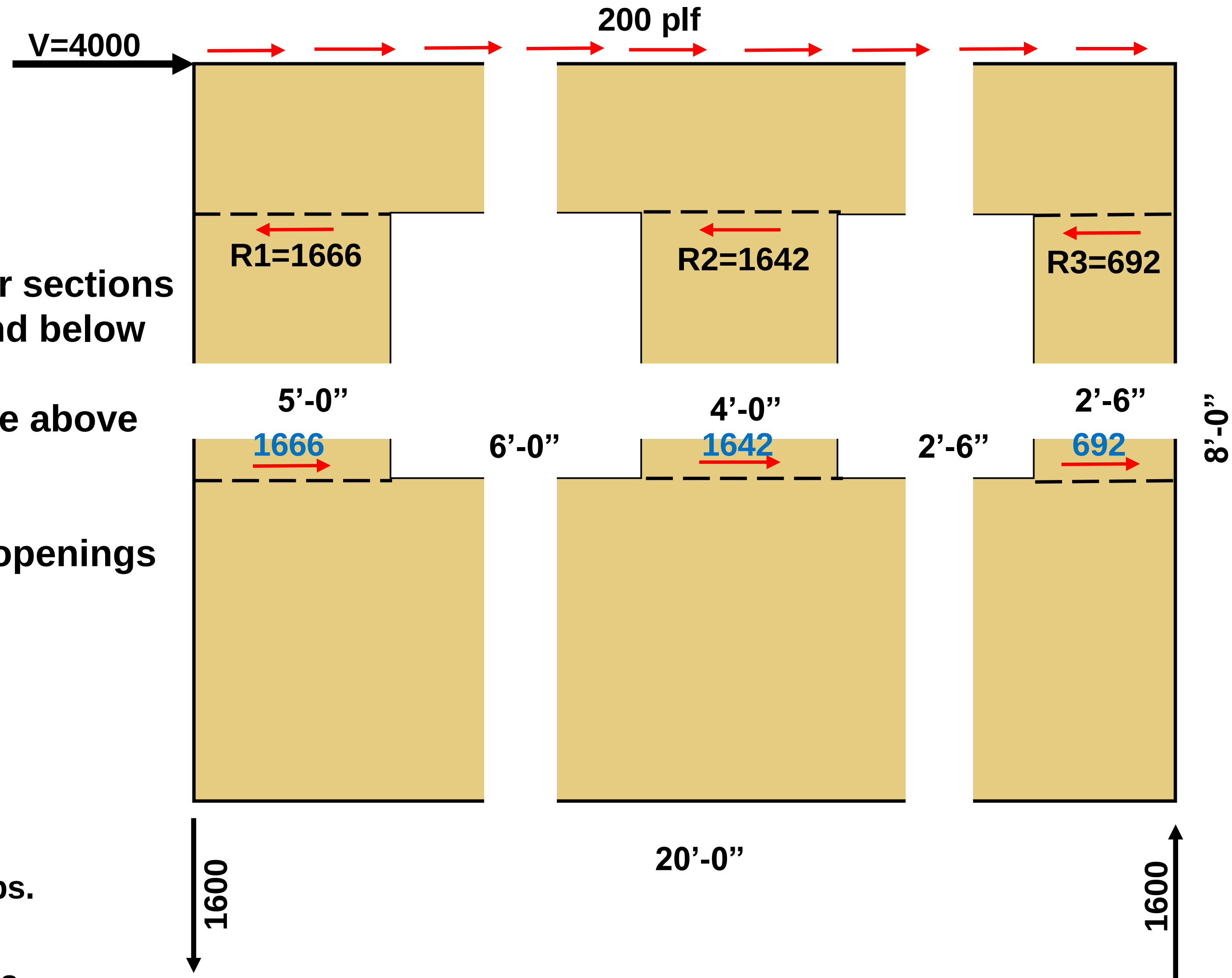
$$V2 = \frac{200(2.67+4+1.54)}{(4)} = 410.5 \text{ plf}$$

$$V3 = \frac{200(0.96+2.5)}{(2.5)} = 276.8 \text{ plf}$$

The pier shear is equal to the unit shear at the top of the wall multiplied by the pier length + the tributary length adjacent to the pier divided by the length of the pier

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings
- Unit shear in piers
- Resisting forces



R1=v1L1=333.2(5)=1666 lbs.

R2=v2L2=410.5(4)=1642 lbs.

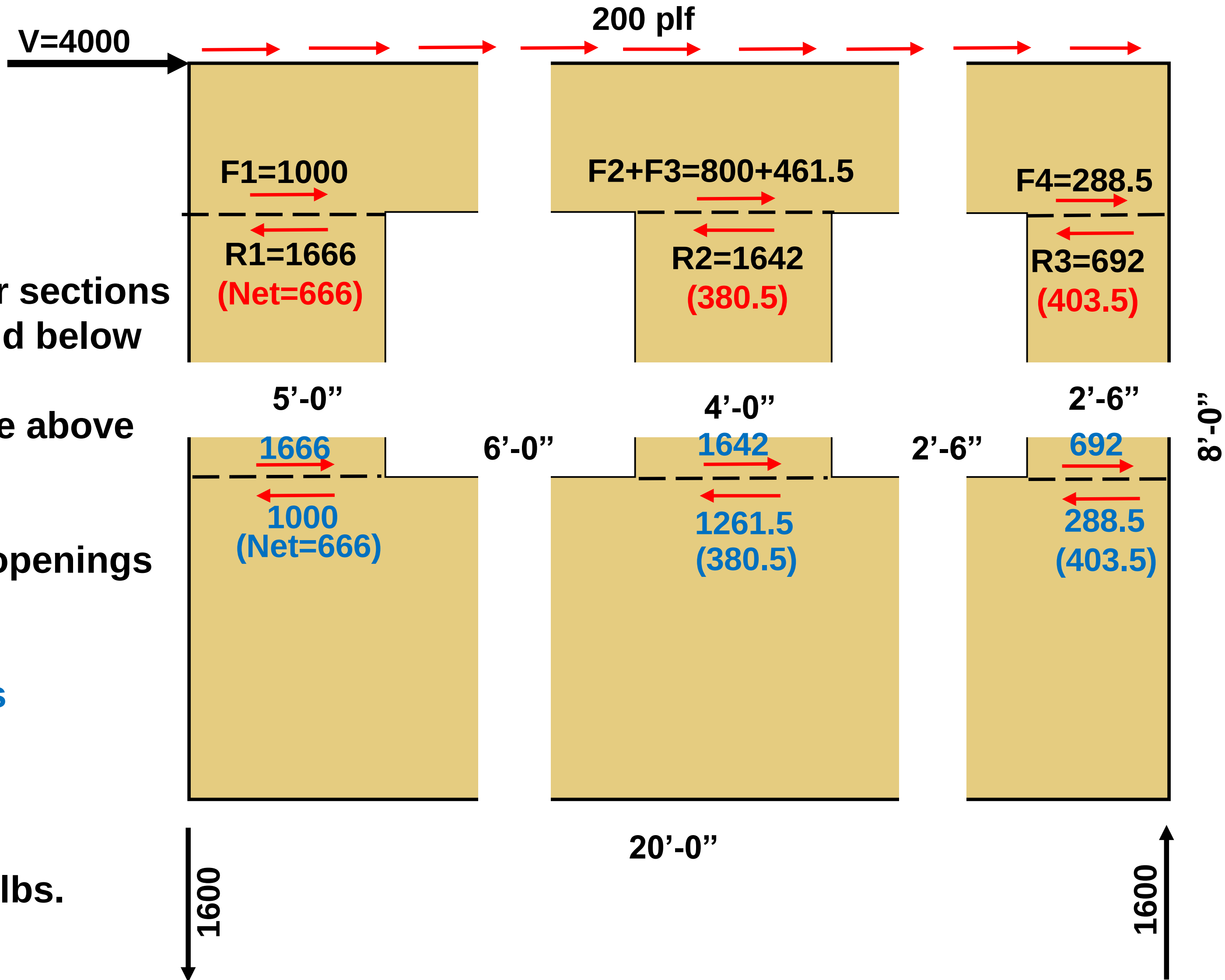
R3=v3L3=276.8(2.5)=692 lbs.

4000 lbs.

The resisting forces of the piers equal to the pier unit shear multiplied by the pier length

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings
- Unit shear in piers
- Resistance forces
- Pier collector forces



R1-F1=1666-1000=666 lbs.

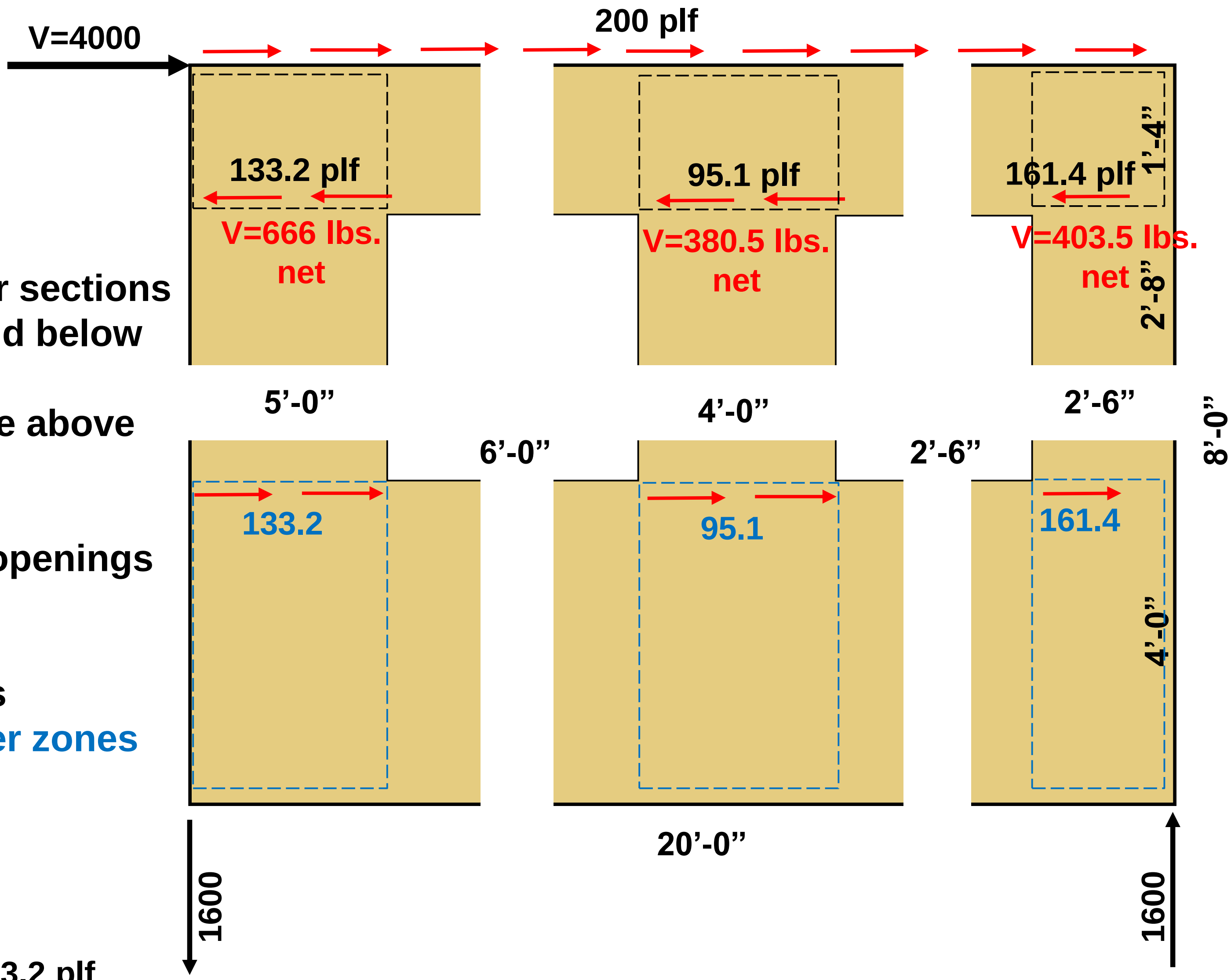
R2-F2=1642-(800+461.5)
=380.5 lbs.

R3-F4=692-288.5=403.5 lbs.

The net collector force at the corners of the Opening is equal to the pier resisting force minus the opening corner force

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings
- Unit shear in piers
- Resistance forces
- Pier collector forces
- Unit shears at corner zones



Va1=(R1-F1)/L1=666/5 = 133.2 plf

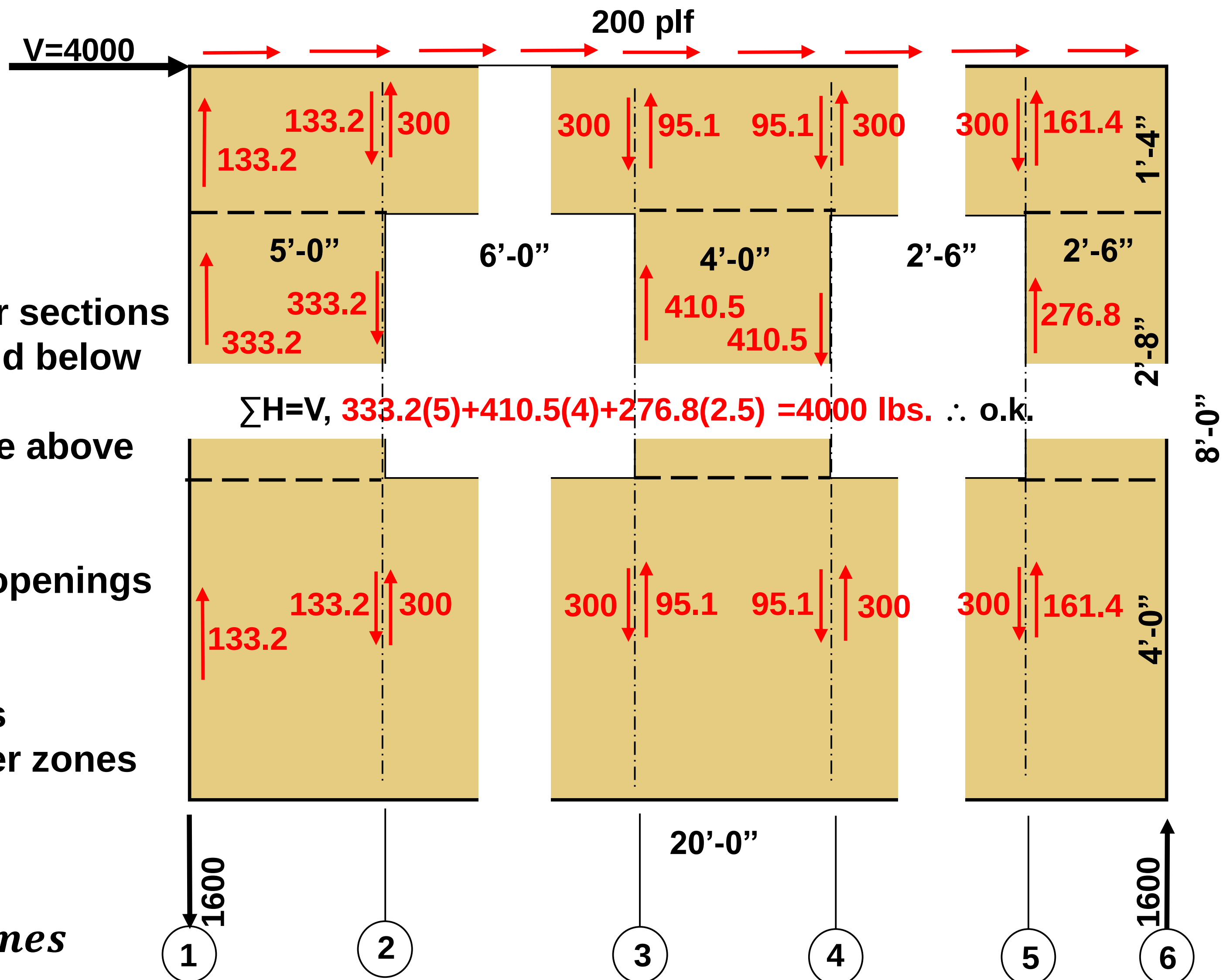
Va2=(R2-F2-F3)/L2=380.5/4 = 95.1 plf

Va3=(R3-F4)/L3=403.5/2.5 =161.4 plf

The corner zone unit shear is equal to the net collector force divided by the pier length

Calculate:

- Aspect ratios of pier sections
- Unit shear above and below opening
- Total boundary force above and below opening
- Corner forces
- Tributary length of openings
- Unit shear in piers
- Resistance forces
- Pier collector forces
- Unit shears at corner zones
- Verify design



$\Sigma V = 0$ *at all grid lines*

Grid 1=va1(ha+hb)+v1(ho)=Fo/t, 133.2(4+1.33)+333.2(2.67)=1600 ∴ o.k.

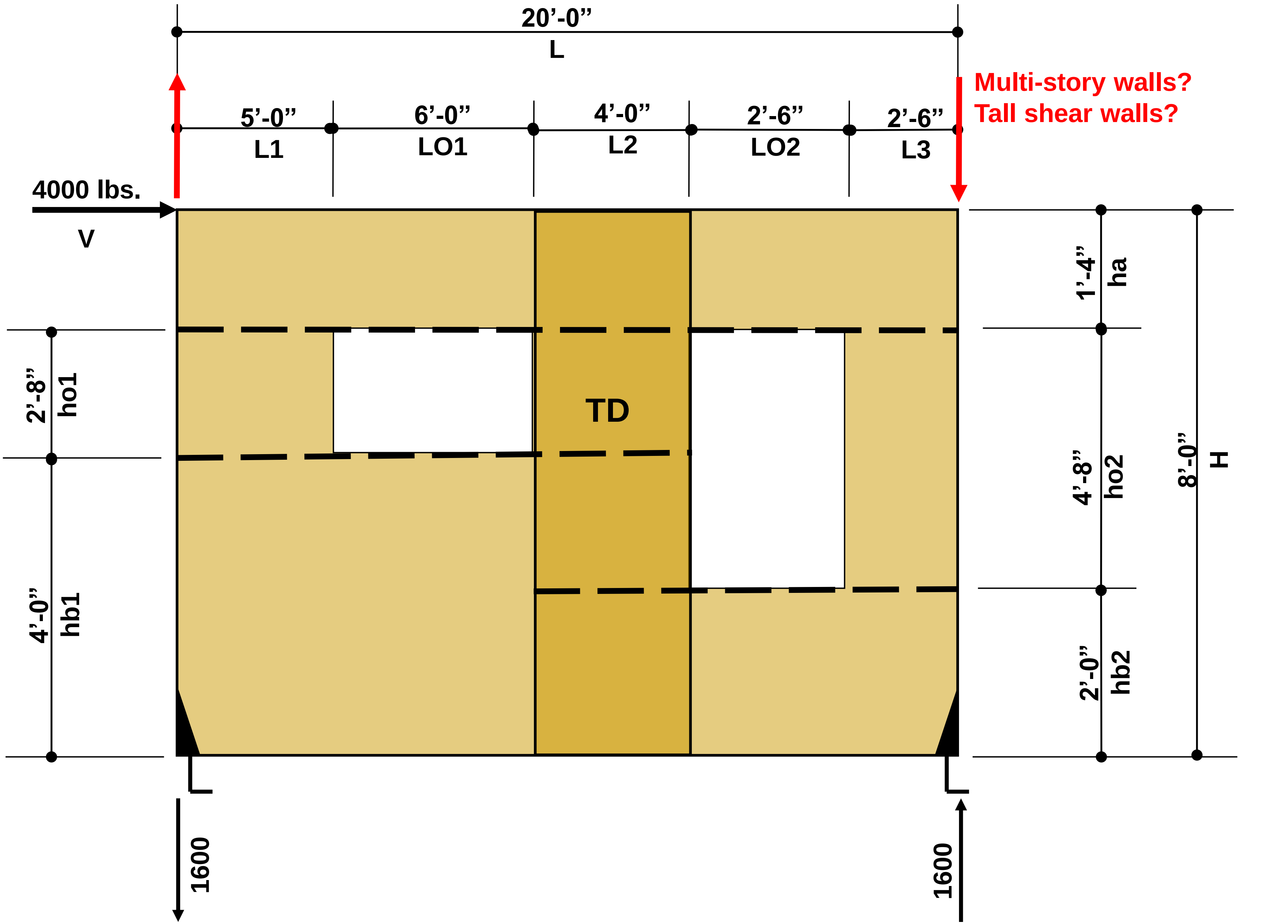
Grid 2=va(ha+hb)-va1(ha+hb)-v1(ho)=0, 300(4+1.33)-133.2(4+1.33)-333.2(2.67)=0 ∴ o.k.

Grid 3=va2(ha+hb)+v2(ho)-va(ha+hb)=0, 95.1(5.33)+410.5(2.67)-300(5.33)=0 ∴ o.k.

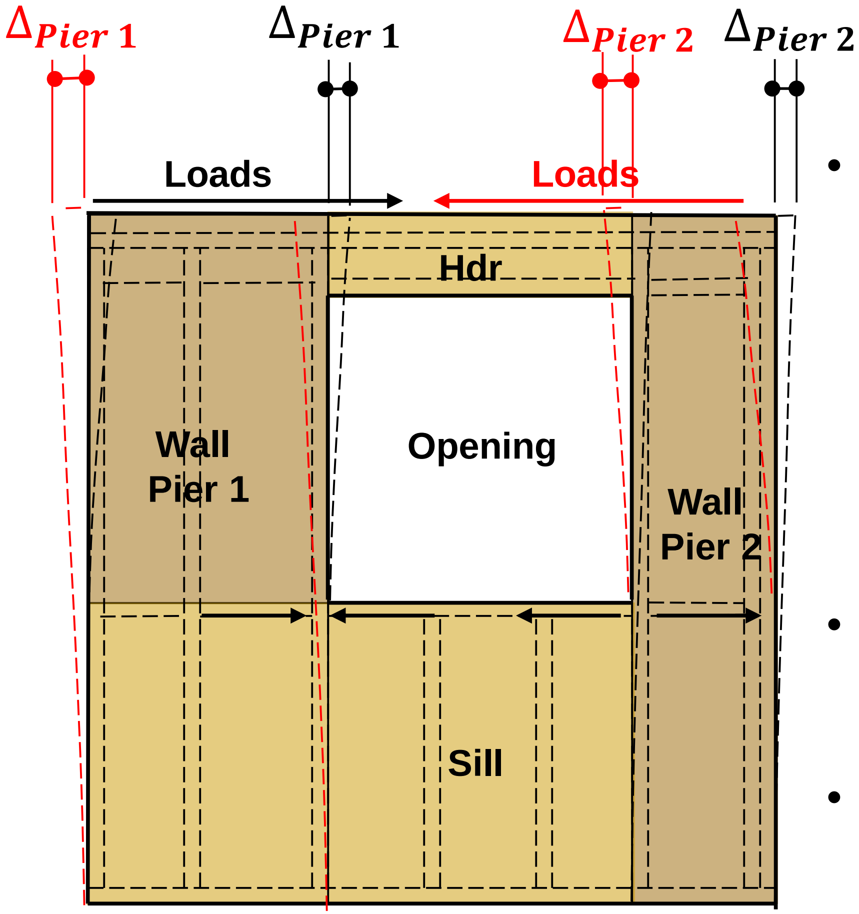
Grid 4=va2(ha+hb)+v2(ho)-va(ha+hb)=0, -95.1(5.33)-410.5(2.67)+300(5.33)=0 ∴ o.k.

Grid 5=va(ha+hb)-va3(ha+hb)-v3(ho)=0, -300(5.33)+161.4(5.33)+276.8(2.67)=0 ∴ o.k.

Grid 6=va3(ha+hb)+v3(ho)=Fo/t, 161.4(5.33)+276.8(2.67)=1600 ∴ o.k.



20’-0”
L
Multi-story walls?
Tall shear walls?
5’-0”
L1
6’-0”
LO1
4’-0”
L2
2’-6”
LO2
2’-6”
L3
4000 lbs.
V
TD
1’-4”
ha
2’-8”
ho1
4’-8”
ho2
8’-0”
H
4’-0”
hb1
2’-0”
hb2
1600
1600



- The deflection of the wall is the average of the deflection of the piers as shown **(acting both ways combined)** using the 4 term eq.

Single opening

$$\Delta_{Aver.} = \frac{(\Delta_{pier\ 1} + \Delta_{pier\ 2}) + (\Delta_{pier\ 1} + \Delta_{pier\ 2})}{4}$$

- The remainder of the terms are identical to the traditional equation.

- Deflections for a wall with multiple openings is similar.

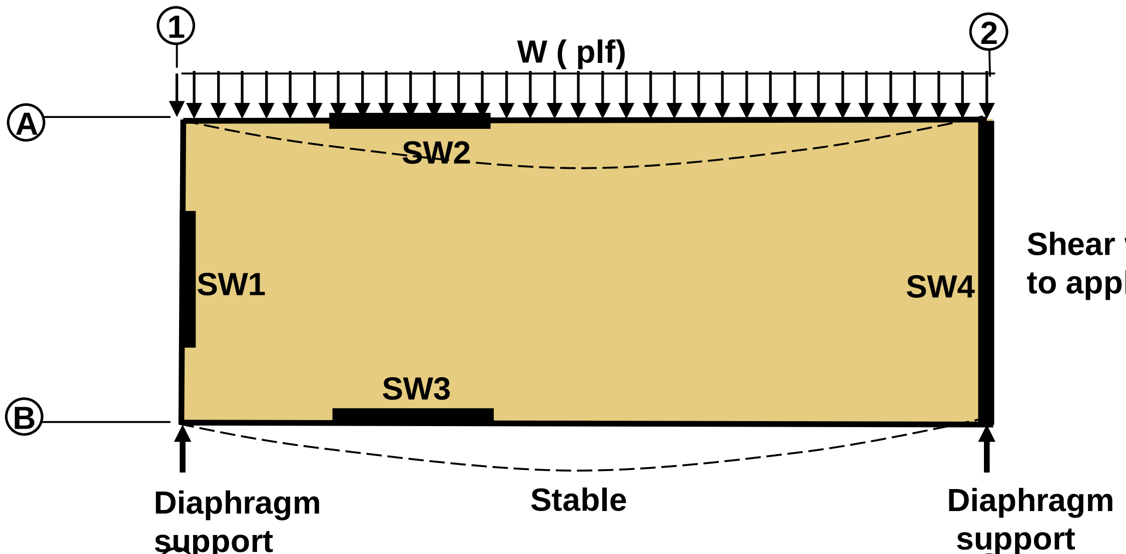
$$\Delta_{Aver.} = \frac{(\Delta_{pier\ 1} + \Delta_{pier\ 2} + \Delta_{pier\ 3}) + (\Delta_{pier\ 1} + \Delta_{pier\ 2} + \Delta_{pier\ 3})}{6} \ldots$$

Traditional 4 term deflection equation

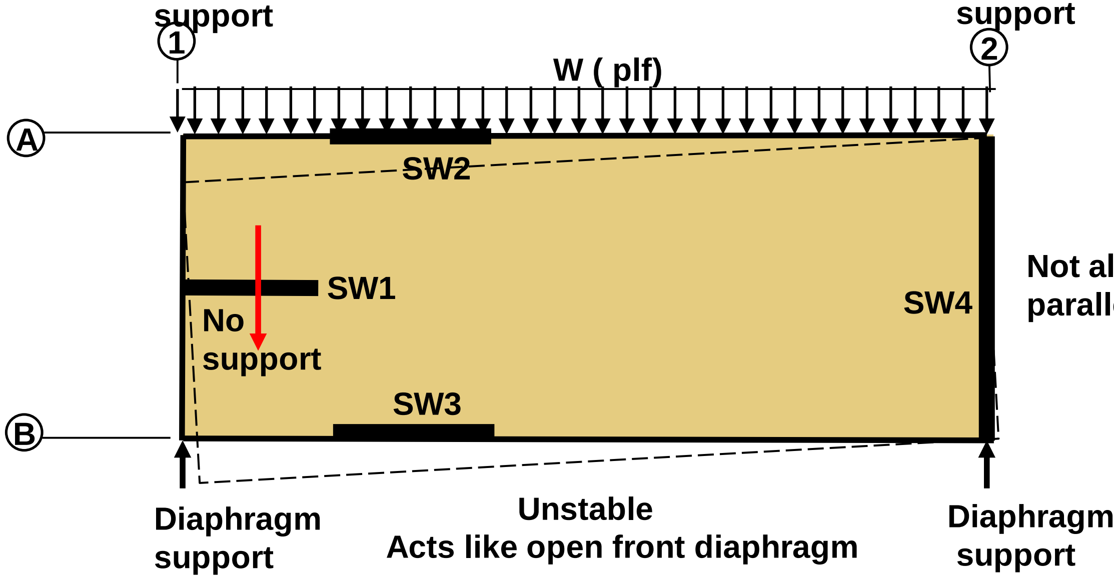
$$\Delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{G_v t_v} + 0.75he_n + \frac{h\Delta_a}{b}$$

Reference APA SR-105

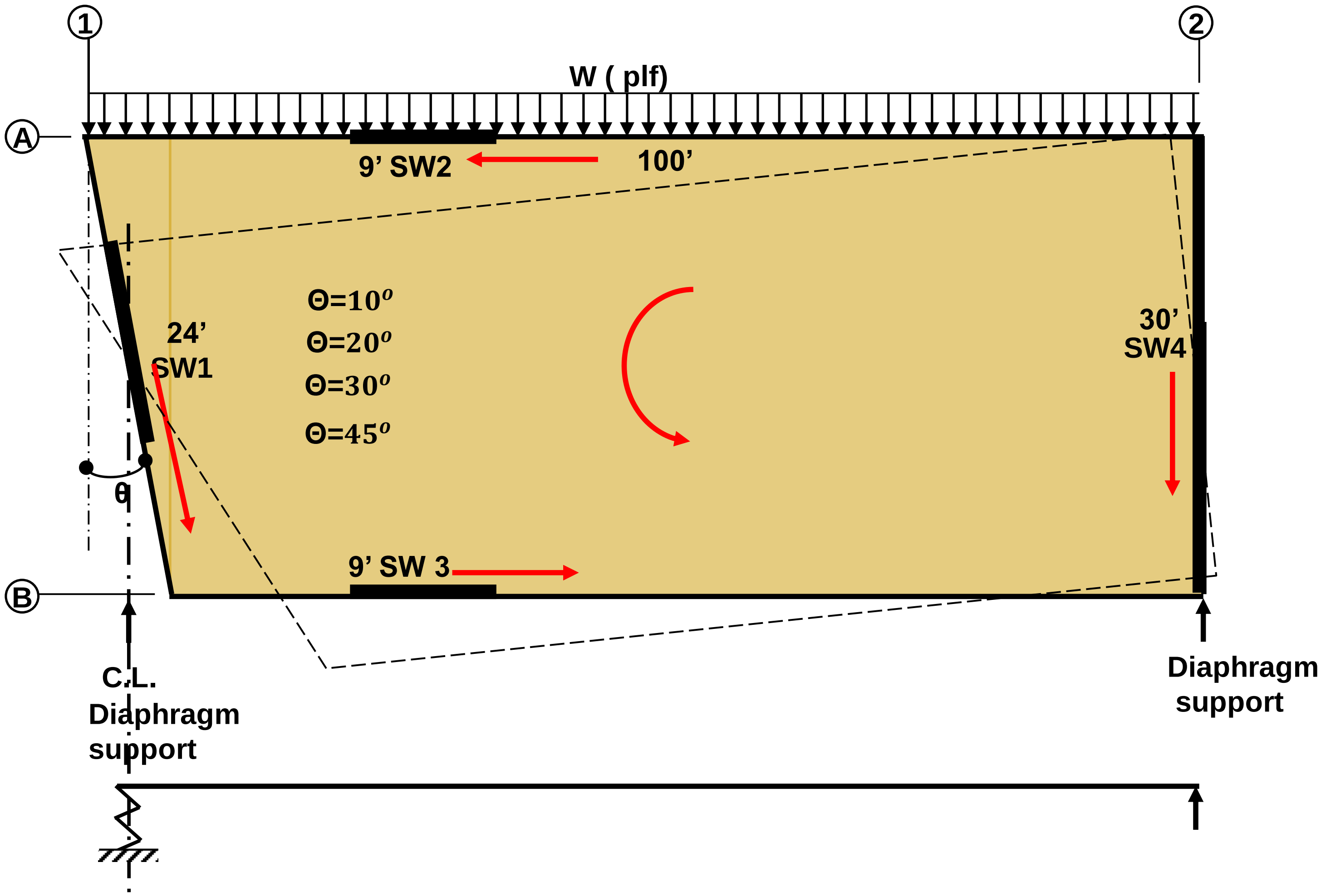
Angled Shear Walls



Shear walls are parallel to applied loads

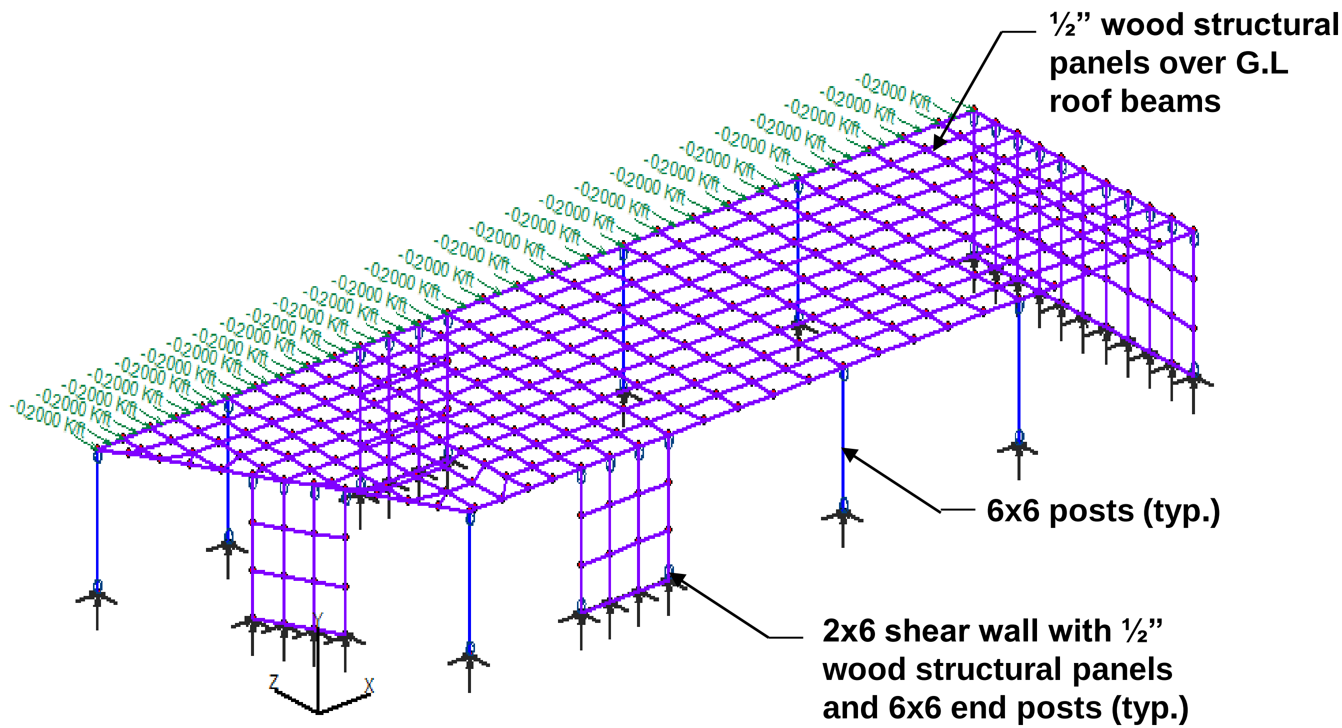


Not all shear walls are parallel to applied loads



Diaphragm with Angled Shear Wall Layout (Base Model)

Analogous to a simple span beam with a pinned end and spring supported end


½" wood structural panels over G.L roof beams
6x6 posts (typ.)
2x6 shear wall with ½" wood structural panels and 6x6 end posts (typ.)
-0.2000 K/ft
Y
Z
X

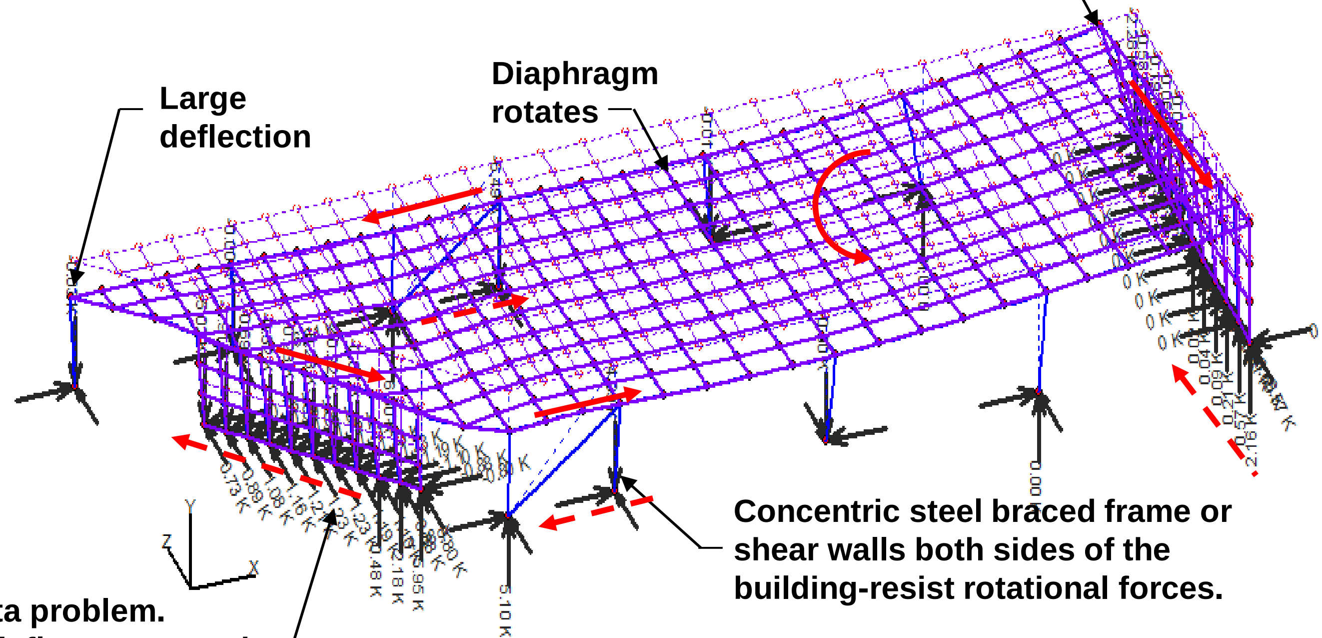
FEA Model

Diaphragm with Angle Shear Wall-Horizontal Type 5 Irregularity Transverse Loading

Wall warps and translates

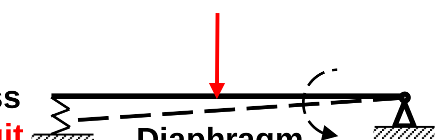
Diaphragm rotates

Large deflection



Concentric steel braced frame or shear walls both sides of the building-resist rotational forces.

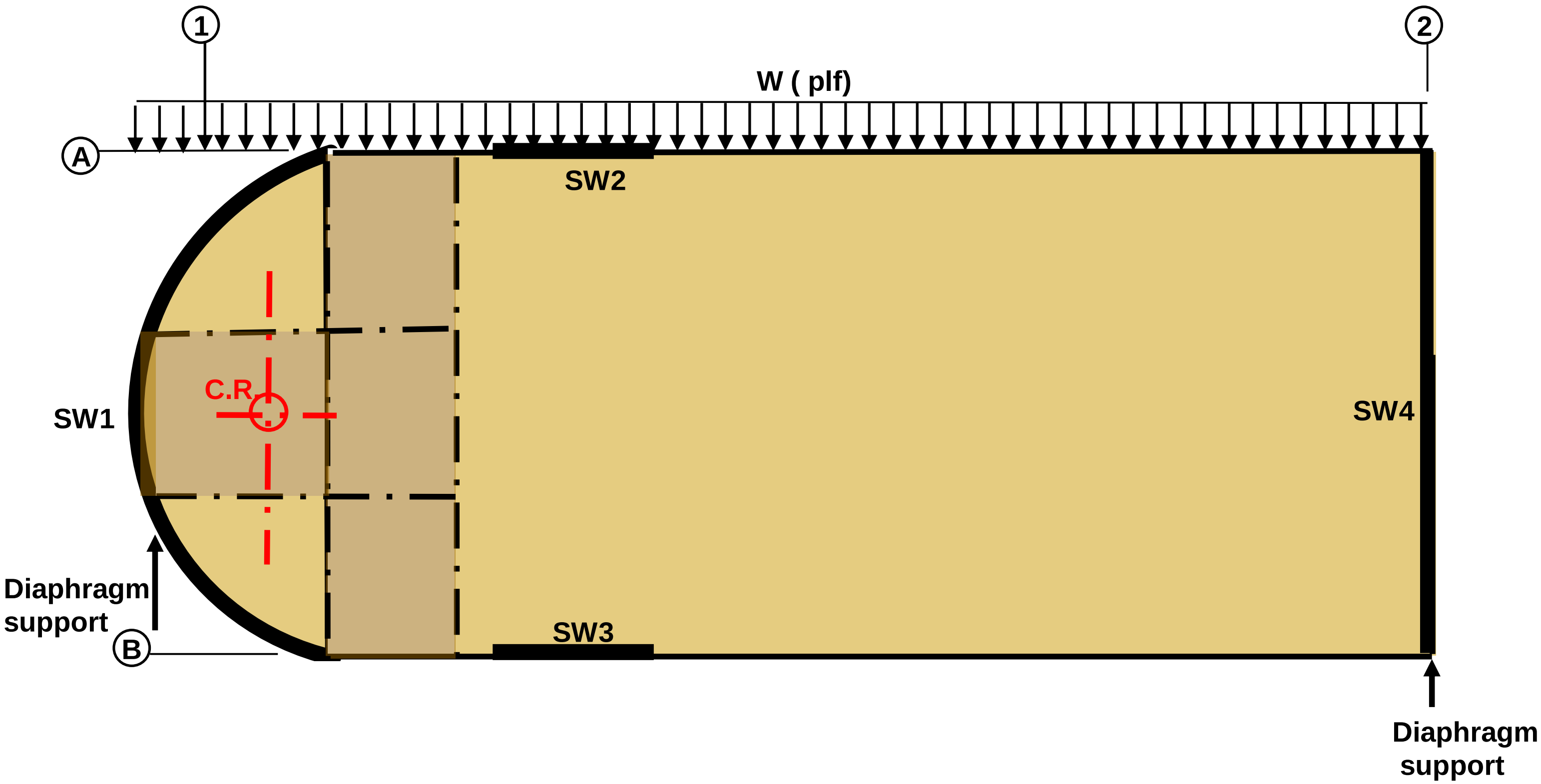
P-delta problem.
Wall deflects too much out-of-plane, which causes rotation in the diaphragm
(Acts like open front diaphragm-shift forces to side walls)



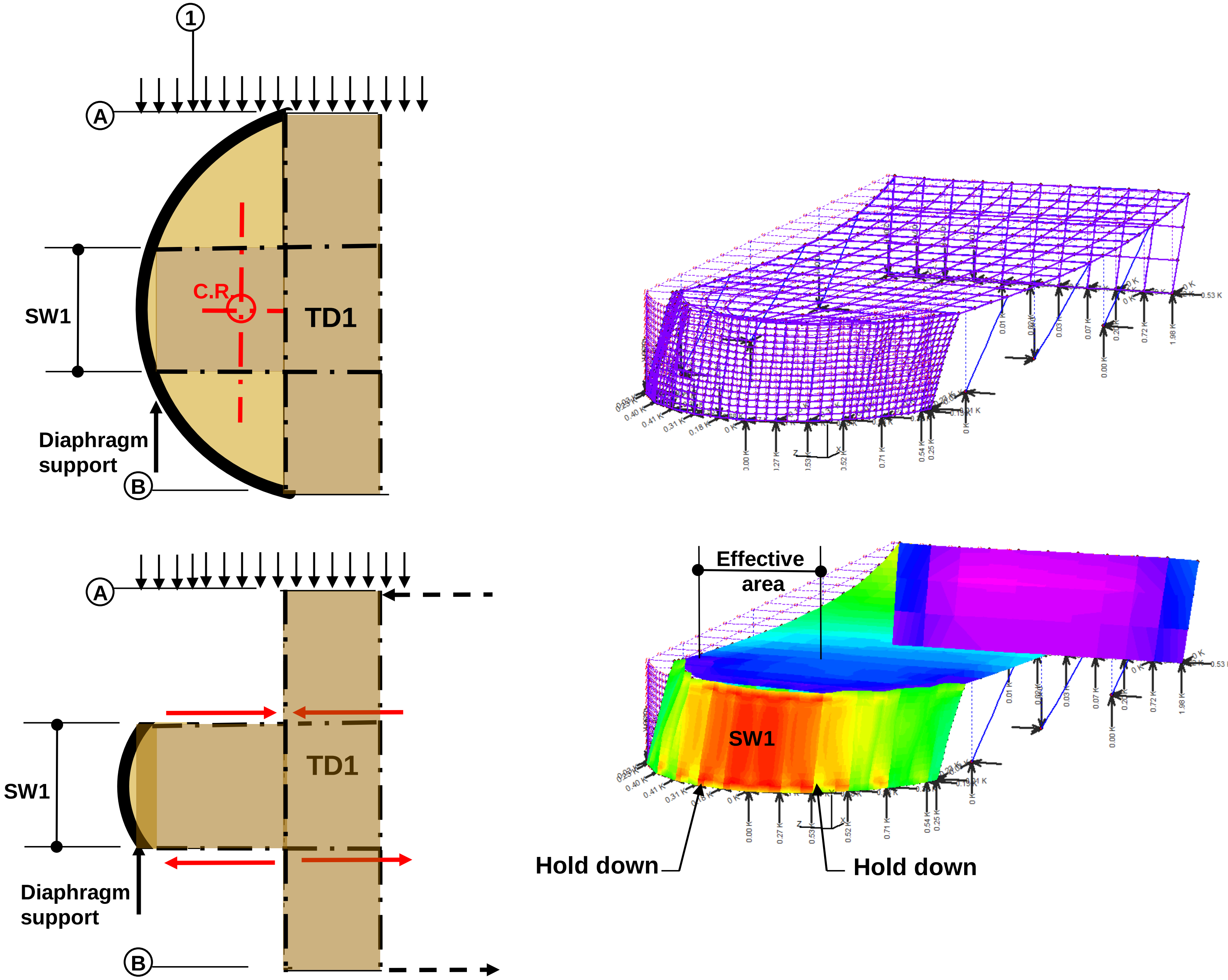
Structural Behavior

Structure is stable

Circular Shear Walls



Example- Circular Shear Wall Layout


1
A
B
SW1
C.R
TD1
Diaphragm support
A
B
SW1
TD1
Diaphragm support
Effective area
SW1
Hold down
Hold down

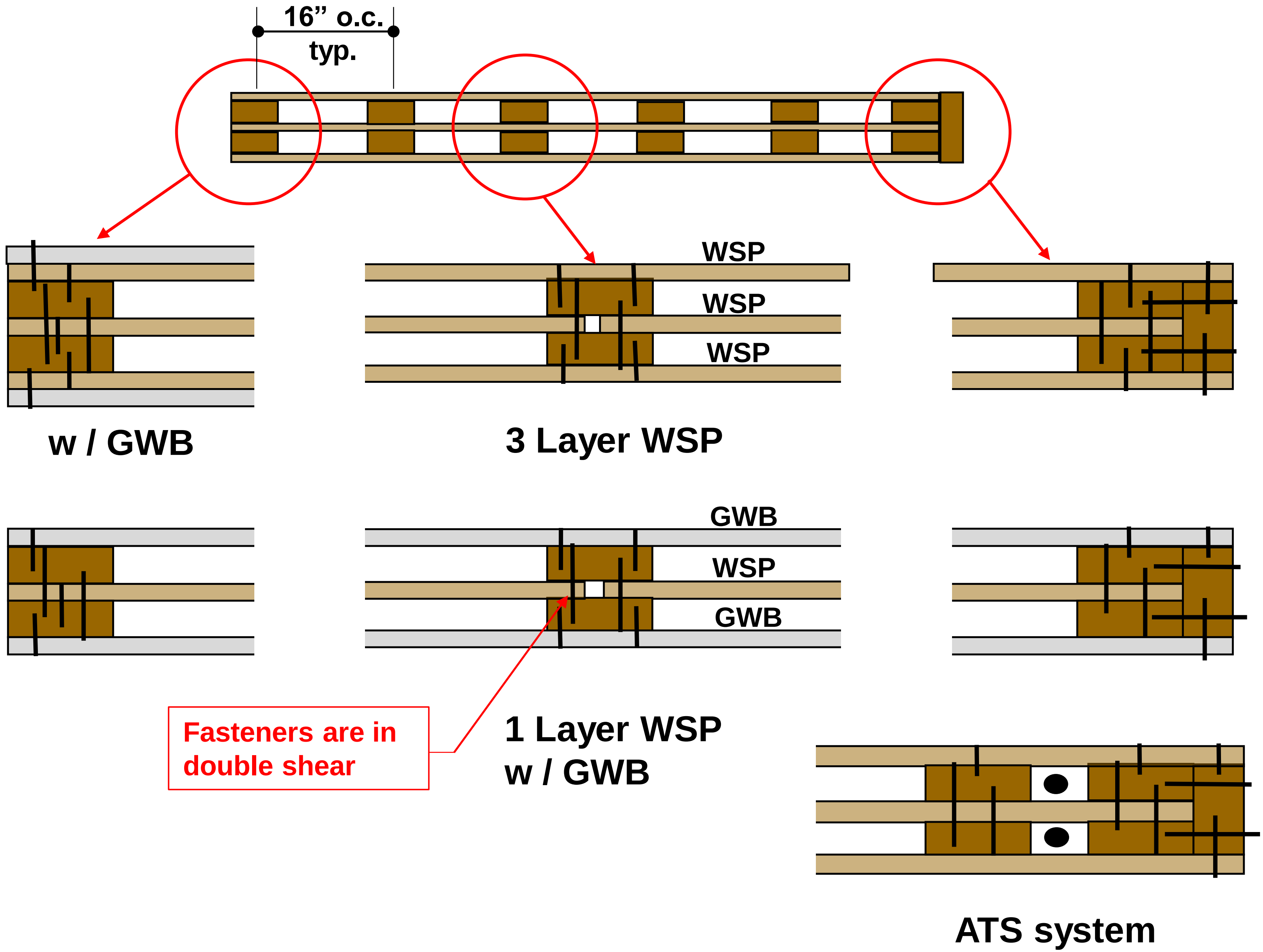
Mid-Ply Shear Walls

Source: nees.org

- **Mid-ply walls:**
  - **Carry high shear demand**
  - **Reduce torsional effects**

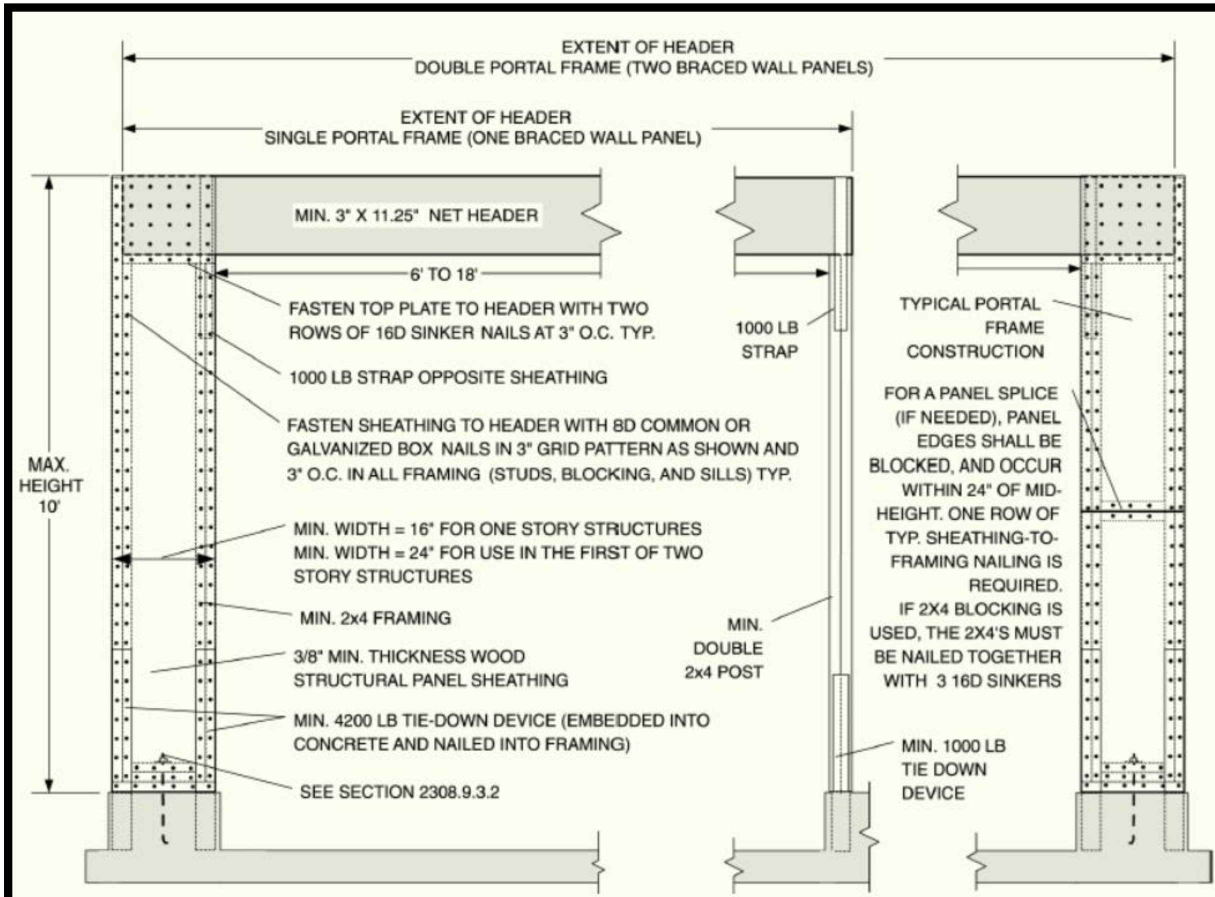
- **Test Results:**
  - **Mid-ply walls 2.4 to 2.8 x stiffer than standard shear walls monotonic loading.**
  - **Mid-ply walls 1.8 to 2.2 x stiffer than standard shear walls cyclic loading.**
  - **Better energy dissipaters, 3 to 5x better than standard shear walls**

Typical Mid-Ply Shear Walls



Prescriptive / Proprietary Portal Frames

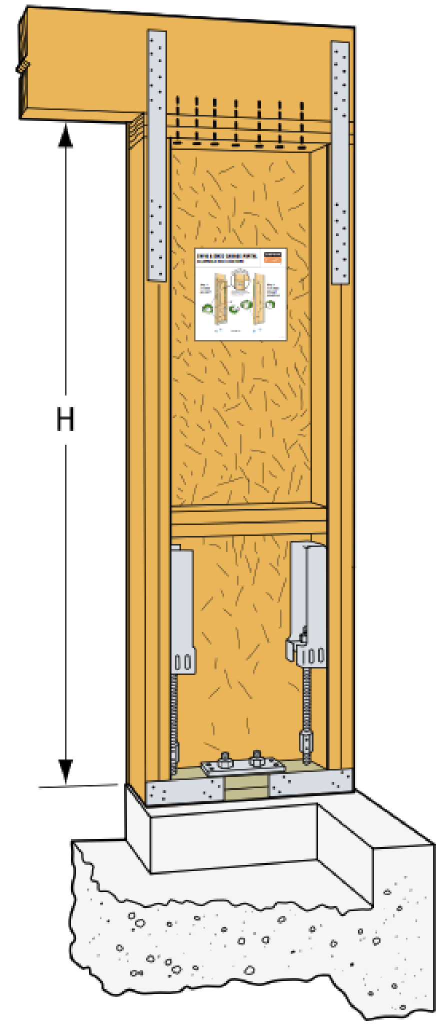
IBC 2308.9.3.2
Prescriptive Code Portal Frames



For SI: 1 foot = 304.8 mm; 1 inch = 25.4 mm; 1 pound = 4.448 N.

FIGURE 2308.9.3.2 ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

Proprietary Portal Frames



Source: strongtie.com

Pre-engineered Proprietary Shear Walls

Pre-engineered Narrow Wall Section Proprietary

<u>**Considerations:**</u>

Large Hold-down forces

Larger deflections

Larger foundations

Pre-engineered Proprietary Truss Walls

Truss Walls

Truss Frames

Hybrid Wood/Steel Proprietary Systems

Source: hardyframe.com

Presentation Topics

- Shear Wall Types
- **Shear wall Anchorage**
- SDPWS Code Requirements
- Complete Load Paths
- Offset Shear Walls and Relevant Code Requirements
- Mid-rise Diaphragms and Tall Shear Walls

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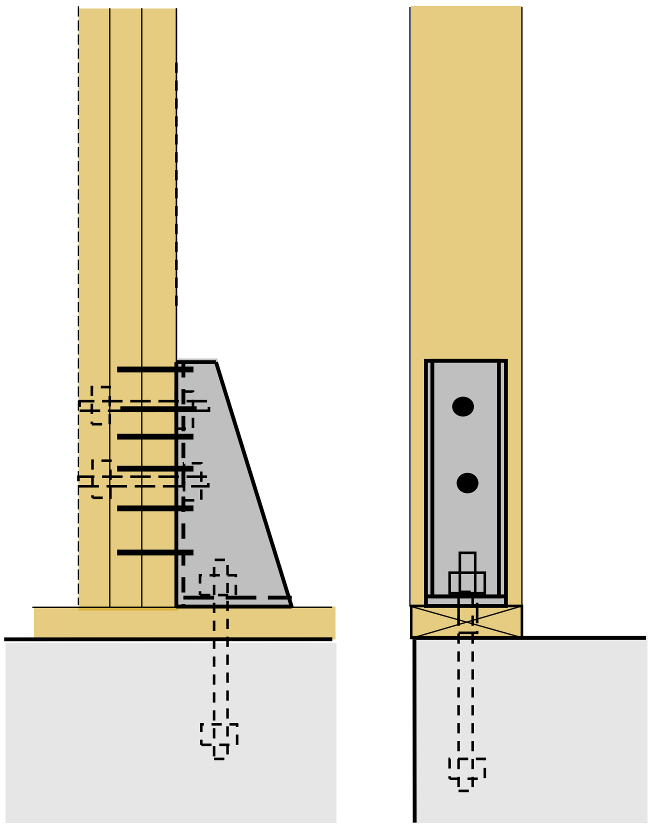
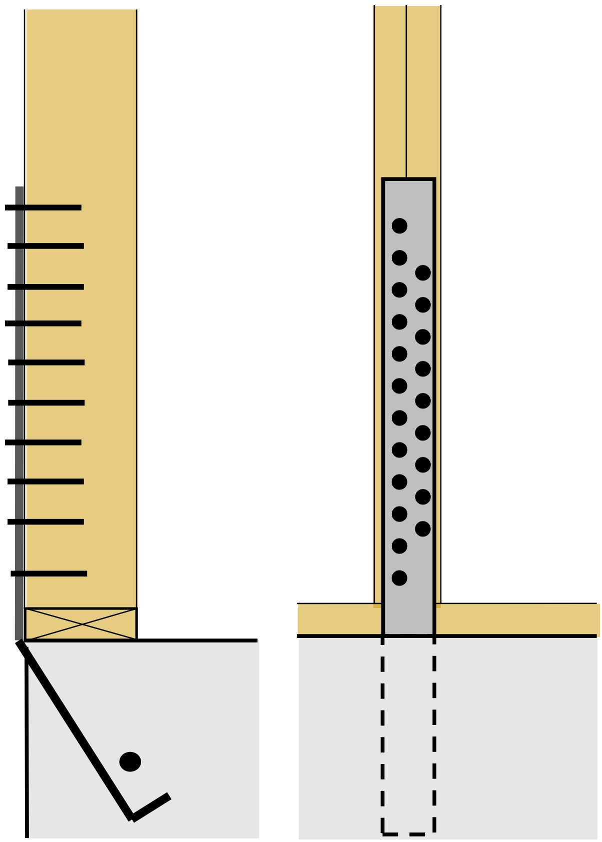
Togawa Smith Martin

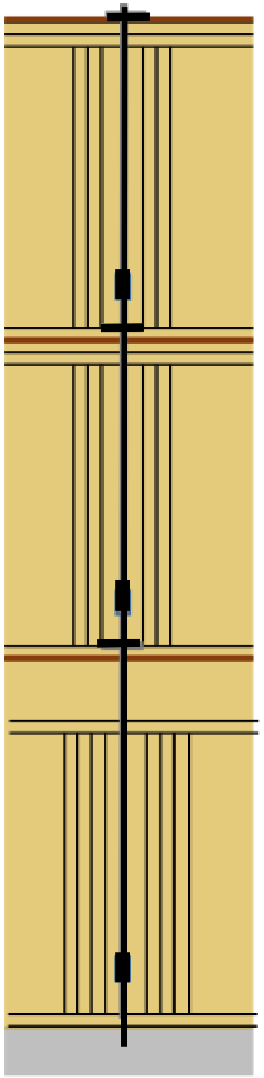
Hold Down Anchors Systems



Standard Hold Down
(Bucket Style)

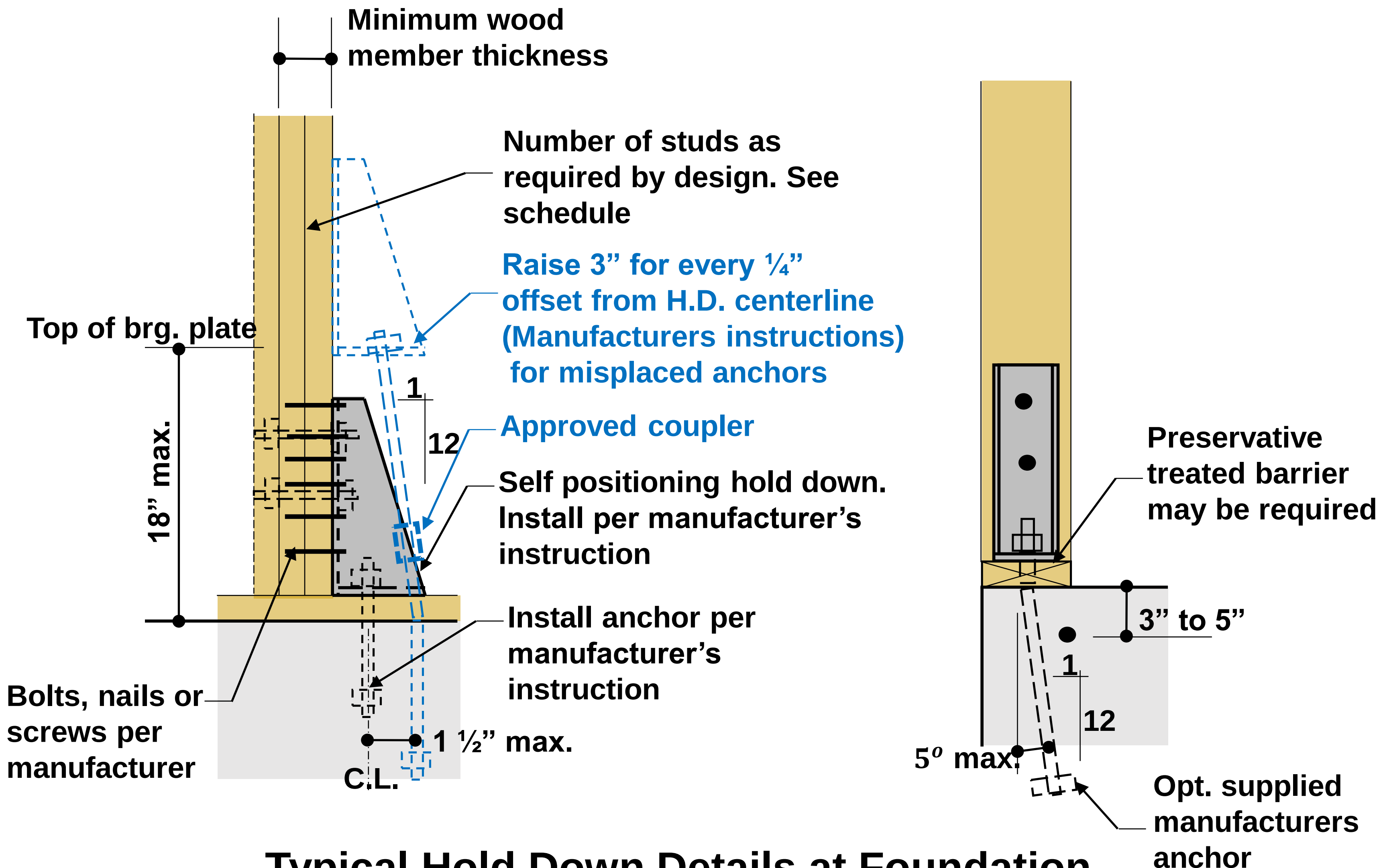


Strap Hold Down



Continuous Rod
Automatic Tensioning System
ATS

Bucket Style Hold Downs



Typical Hold Down Details at Foundation

Standard (discrete) Hold Downs

Multiple Hold Downs at Corners

Source: strongtie.com

Source: DartDesignInc.com

Courtesy of Willdan Engineering

Hold downs installed at an angle

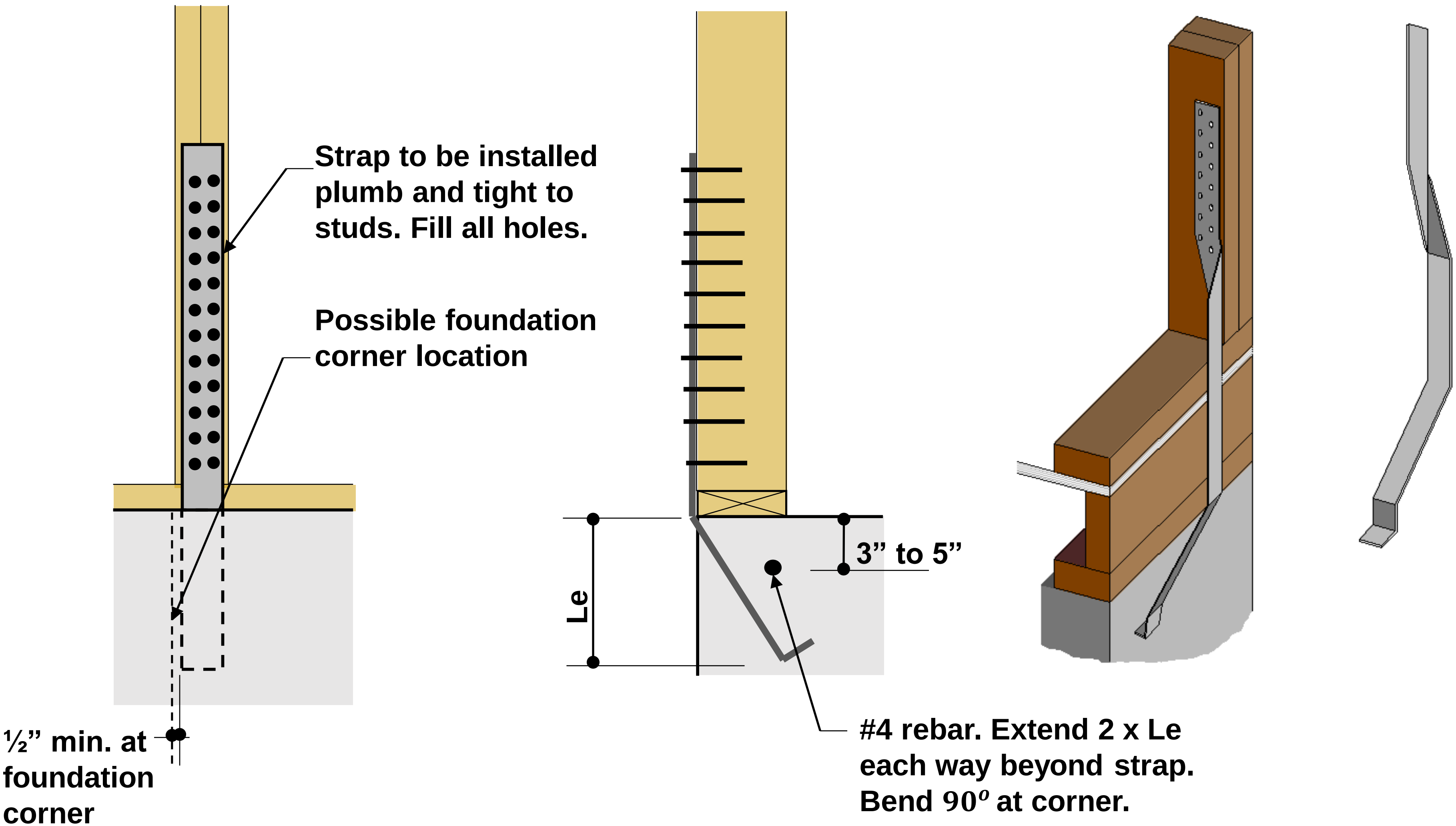
Large knots

Field Installation Issues

Courtesy of Willdan Engineering

Field Installation Issues

Strap Hold Down Installations



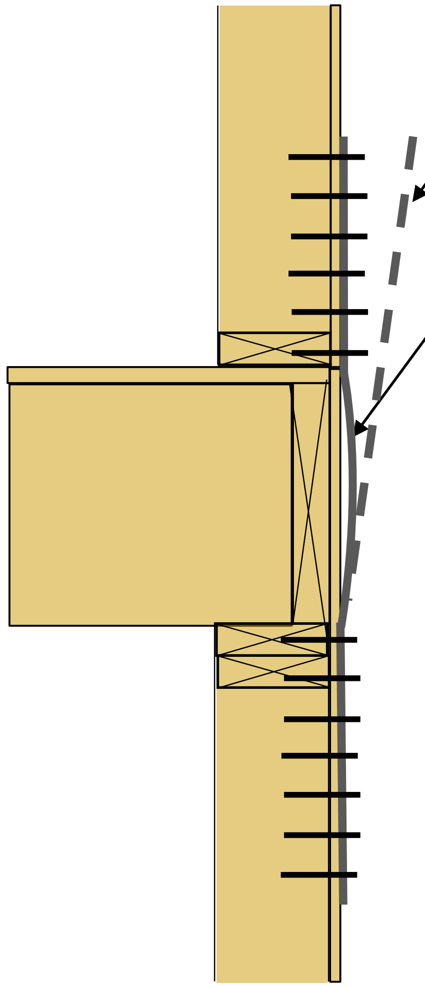
Field Installation Issues

Loose anchor strap-stabbed into foundation-no vibration

Slack in hold down strap (bent)
Often stabbed into foundation
without vibration.

Field Installation Issues

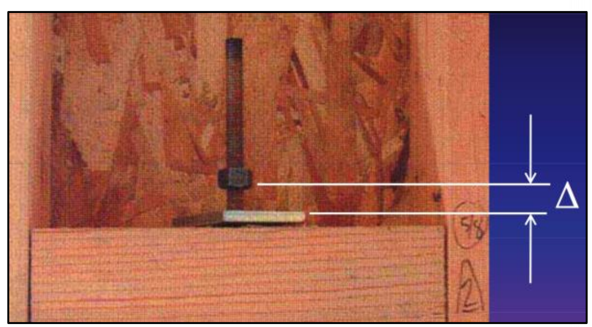
Field Installation Issues

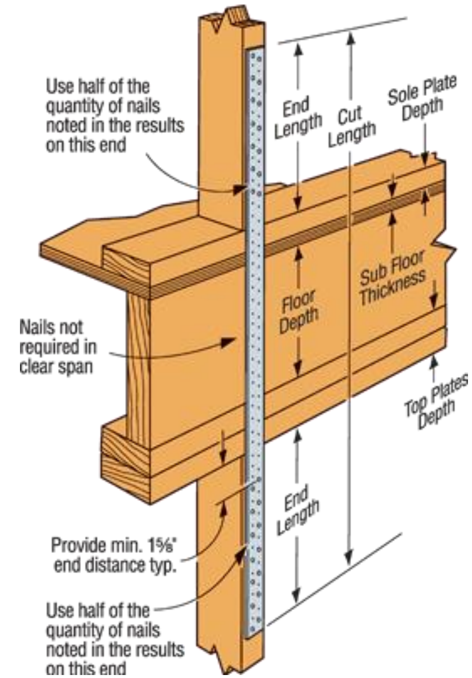


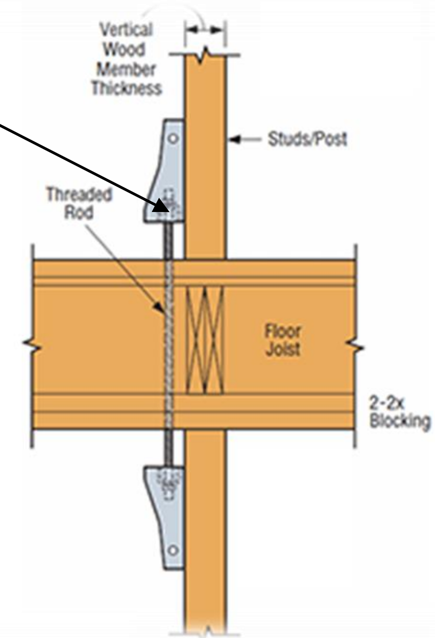
Nail after shrinkage occurs.

Slack in strap (bent) due to vertical shrinkage of framing.

Loose bolted connection due to vertical shrinkage of framing. Tighten after shrinkage







Typical Hold Down Details at Floor

Standard (discrete) Hold Downs

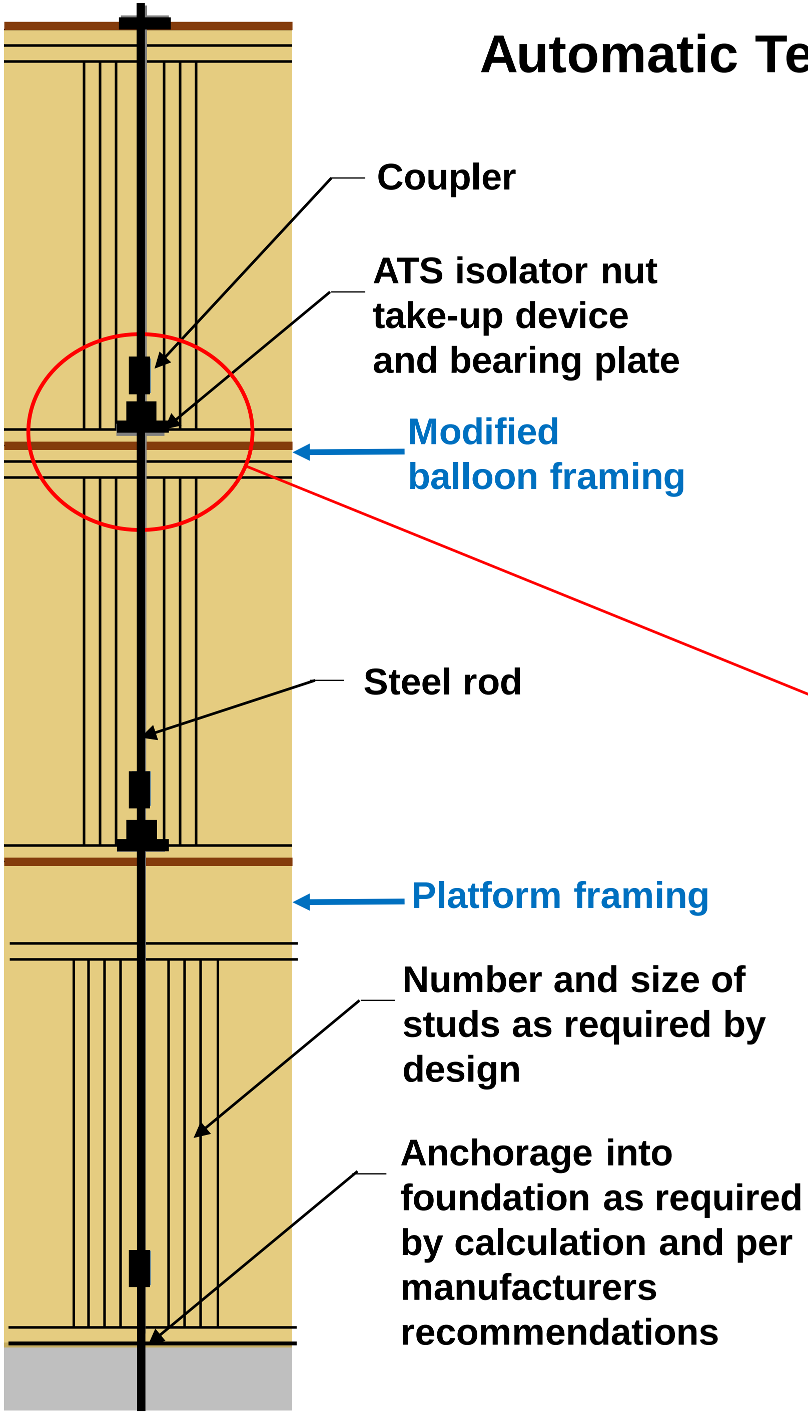
Continuous Rod Tie Downs with Shrinkage Compensation Devices

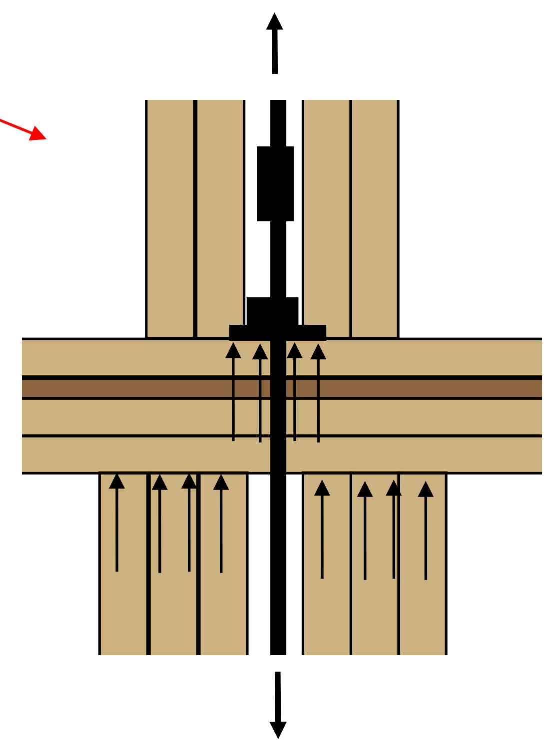
Automatic Tensioning Systems/Devises

Source: strongtie.com

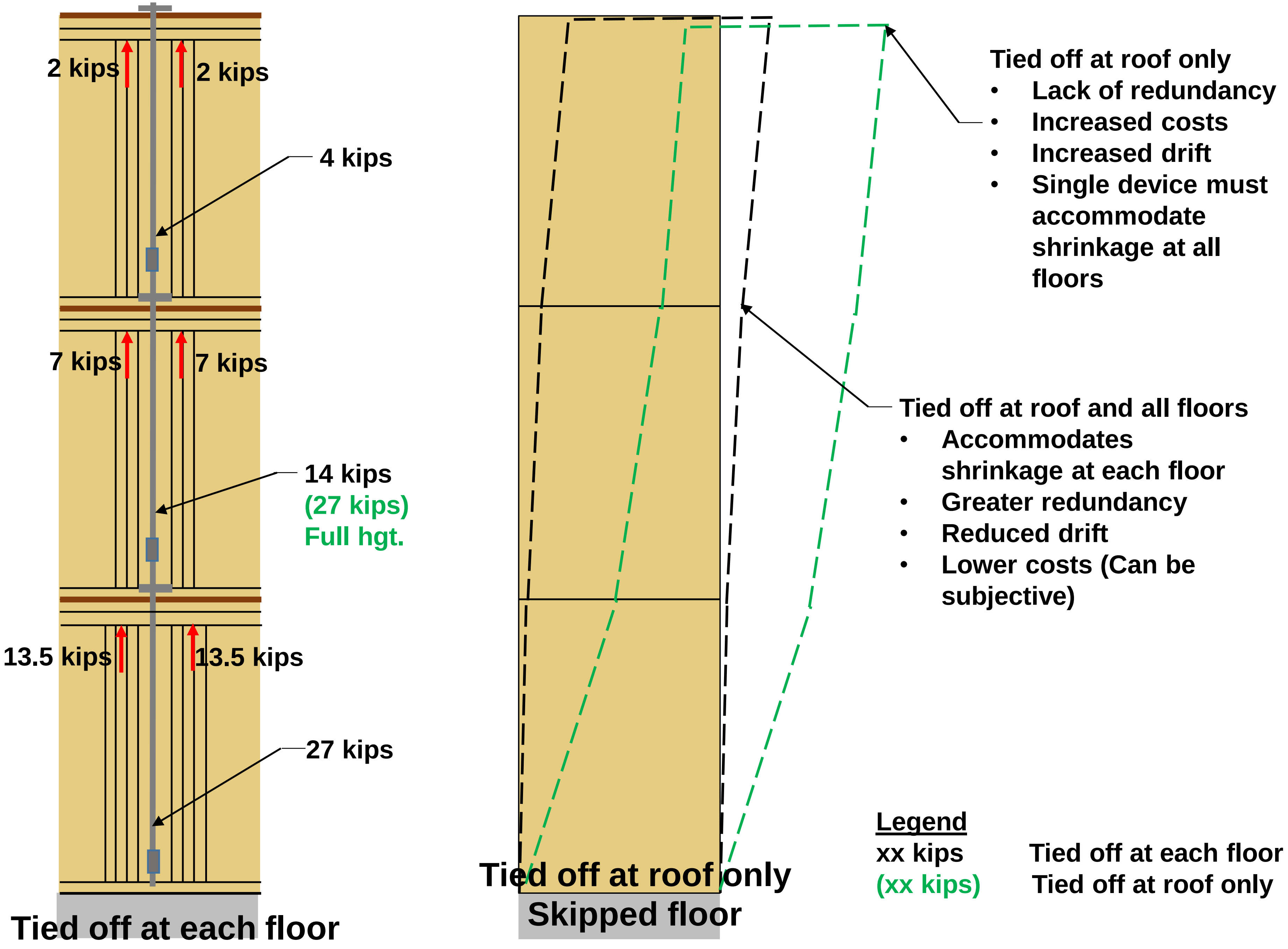
Automatic Tensioning Systems/Devises

- Restraints are required at roof and each floor level to get best results
- Software programs are available for design
- Must be installed per manufacturers recommendations





Automatic Tensioning Systems/Devises



Presentation Topics

- Shear Wall Types
- Shear wall Anchorage
- **SDPWS Code Requirements**
- Complete Load Paths
- Offset Shear Walls and Relevant Code Requirements
- Mid-rise Diaphragms and Tall Shear Walls

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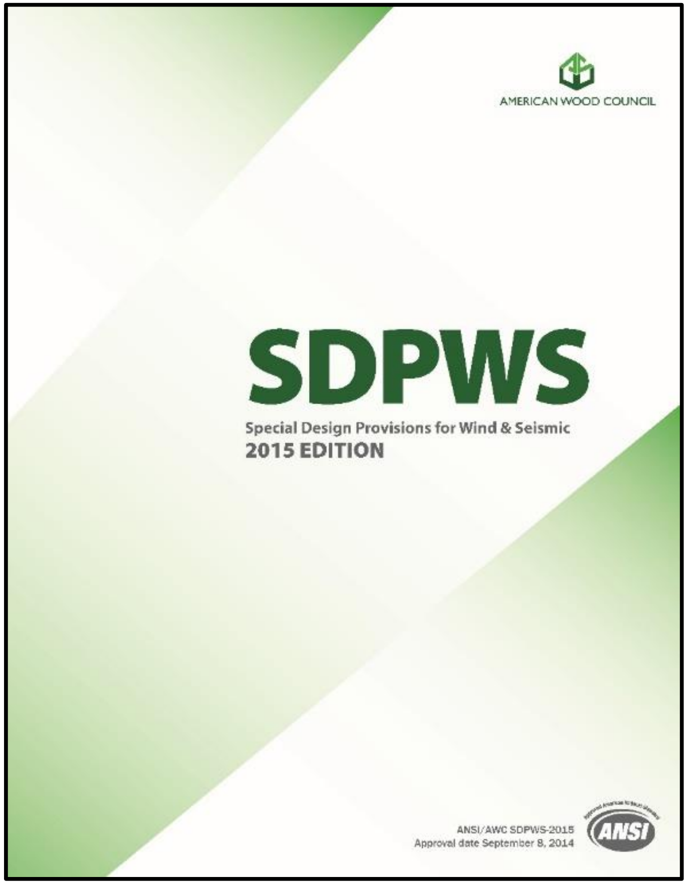
Shear Wall Capacity and Load Distribution



2015 SDPWS

Allowable values shown in tables are nominal shear values

<u>Design values:</u>

LRFD (Strength) = 0.8 x Vn
ASD = Vn/2

Shear Wall Capacity-Wood Based Panels

Blocked

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

Wood Based Panels[4]

Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration In Framing Member or Blocking (in.)	Fastener Type & Size	A Seismic: Panel Edge Fastener Spacing (in.) 6: Vs (plf)	6: Ga (kips/in.) OSB	6: Ga (kips/in.) PLY	4: Vs (plf)	4: Ga (kips/in.) OSB	4: Ga (kips/in.) PLY	3: Vs (plf)	3: Ga (kips/in.) OSB	3: Ga (kips/in.) PLY	2: Vs (plf)	2: Ga (kips/in.) OSB	2: Ga (kips/in.) PLY	B Wind: Panel Edge Fastener Spacing (in.) 6: Vw (plf)	4: Vw (plf)	3: Vw (plf)	2: Vw (plf)
Wood[4,5] Structural Panels- Structural 1	5/16	1-1/4	Nail (common or Galvanized box) 6d	400	13	10	600	18	13	780	23	16	1020	35	22	560	840	1090	1430
	3/8	1-3/8	8d	460	19	14	720	24	17	920	30	20	1220	43	24	645	1010	1290	1710
	7/16			510	16	13	790	21	16	1010	27	19	1340	40	24	715	1105	1415	1875
	15/32			560	14	11	860	18	14	1100	24	17	1460	37	23	785	1205	1540	2045
	15/32	1-1/2	10d	680	22	16	1020	29	20	1330	36	22	1740	51	28	950	1430	1860	2435

Increased 40%*

Use Table 4.3B for WSP over ½" or 5/8" GWB or Gyp-sheathing Board
Use Table 4.3C for Gypsum and Portland Cement Plaster
Use Table 4.3D for Lumber Shear Walls

*40% Increase based on reducing the load factor from 2.8, which was originally used to develop shear capacities down to 2.0 for wind resistance. (2006 IBC commentary)

Shear Wall Capacity-Wood Based Panels

Unblocked

Table 4.3.3.3.2 Unblocked Shear Wall Adjustment Factor, C_{ub}

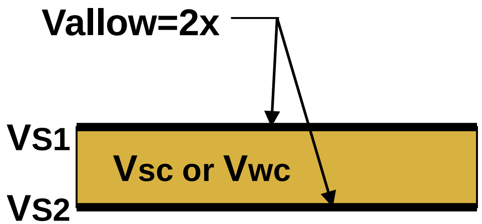
Supported Edges	Intermediate Framing	Stud Spacing (in.)			
		12	16	20	24
6	6	1.0	0.8	0.6	0.5
6	12	0.8	0.6	0.5	0.4

$v_{ub} = v_b C_{ub}$ Eq. 4.3-2

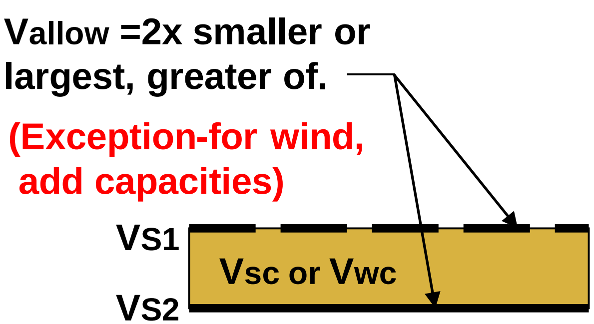
v_{ub} = Nominal unit shear value for unblocked shear wall

v_b = Use nominal shear values from Table 4.3A for blocked WSP shear walls with stud spacing at 24" o.c. and 6" o.c nailing

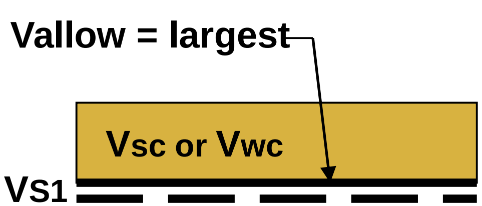
Summing Shear Capacities-4.3.3.3



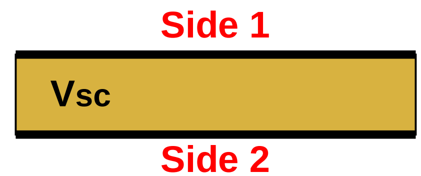
Same material, construction and same capacities



Dissimilar materials



Dissimilar materials (Not cumulative)



Same material, construction on both sides, different capacities-Seismic

The shear wall deflection shall be calculated using the combined apparent shear wall shear stiffness, Gac and the combined nominal unit shear capacity, vsc, using the following equations:

$$v_{SC} = K_{min} G_{ac}$$ Combined apparent shear wall shear capacity (seismic)

$$G_{ac} = G_{a1} + G_{a2}$$

$$K_{min} = \frac{v_{S1}}{G_{a1}} \text{ or } \frac{v_{S2}}{G_{a2}}$$ Minimum of

Where:

Gac= Combined apparent shear wall shear stiffness

Ga1= Apparent shear wall shear stiffness side 1 (Ga2 side 2)-from SDPW Tables 4A-4D

vS1=Nominal unit shear capacity of side 1 (vS2 side 2)

Vsc = combined nominal unit shear capacity seismic
Vwc = combined nominal unit shear capacity wind
vS1=Nominal unit shear capacity of side 1 (vS2 side 2)

2 layers WSP 1 side of wall

APA

APA Report T2014L-21

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

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

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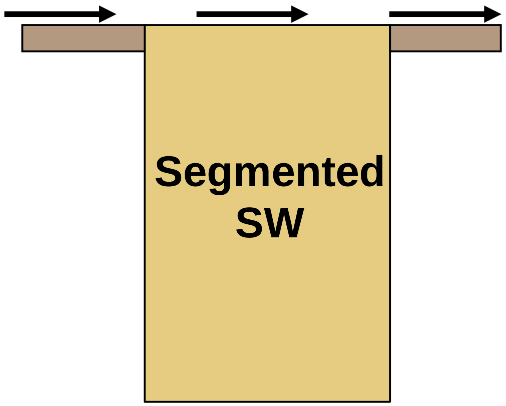
When two layers of WSP sheathing are applied to the same side of a conventional wood-frame wall, the shear capacity for a single-sided shear wall of the same sheathing-type, thickness, and attachment may be doubled provided that the wall assembly meets all of the following requirements:

- **Panel joints between layers shall be staggered**
- **Framing members located where two panels abut shall be a minimum of 3 x framing.**
- **Special sheathing attachment requirements**
- **Special retrofit construction requirements**
- **Double 2x end studs, if used, shall be stitch-nailed together based on the uplift capacity of the double-sheathed shear wall.**

Shear Wall A/R Adjustment Factors-SDPWS Section 4.3.4

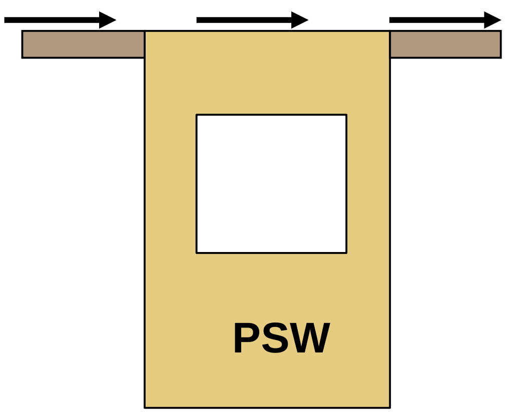


<u>**Capacity Adjustments**</u>**-Wind and seismic**

- **WSP's with A/R > 2:1 multiply Vn x (1.25-0.125h/b) per Section 4.3.4.2.**
- **Struct. Fiberboard with A/R > 1:1 multiply Vn x (1.09-0.09h/b) per Section 4.3.4.2.**

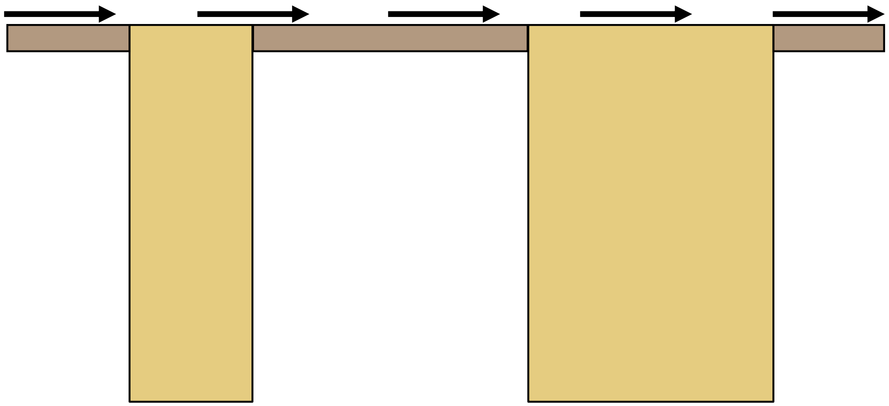
Justification-Based on tests:

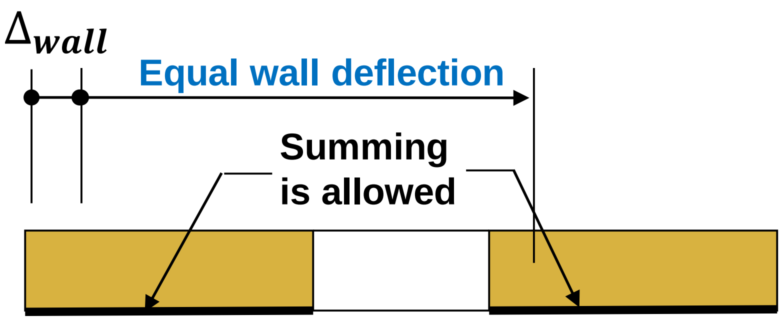
- **Accounts for reduced unit shear capacity in high aspect ratio walls due to loss of stiffness as A/R increases.**



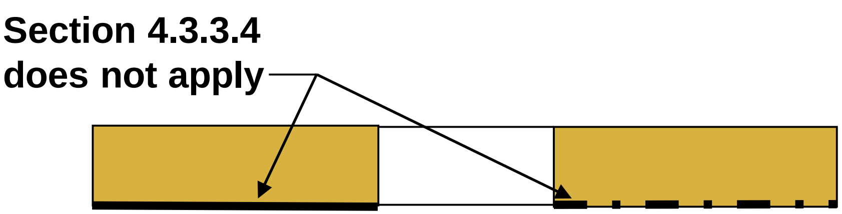
- **Segments with an aspect ratio > 2:1 shall be multiplied by 2b/h for the purposes of determining Li and ∑Li. The provisions of Section 4.3.4.2 and the exceptions to Section 4.3.3.4.1 (equal deflection) shall not apply to perforated shear wall segments.**

Distribution of lateral forces to In-line Shear Walls





Same materials in same wall line



Dissimilar materials in same wall line

Section 4.3.4.2-Limitation of section

- **All walls are of same materials and construction**
- **Shear distribution to individual SW's shall provide same calculated deflection in each shear wall (Default Method) per 2015 SDPWS)**

Exception:

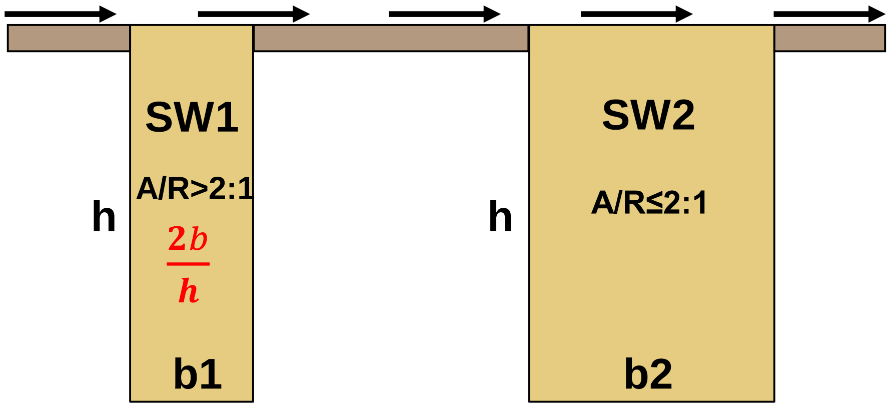
1. **Where Vn of all WSP shear walls having A/R>2:1 are multiplied by 2b/h, shear distribution to individual full-height wall segments is permitted to be taken as proportional to design shear capacities of individual full height wall segments. (Traditional Method-by length)**

   Where multiplied by 2b/h the Vn need not be reduced additionally by Aspect Ratio Factor (WSP)= 1.25-0.125h/b per Section 4.3.4.2.

2. **Where Vn x 0.1+0.9b/h of all structural fiberboard shear walls with A/R>1:1, shear distribution to individual full-height wall segments shall be permitted to be taken as proportional to design shear capacities. (Traditional Method-by length)**

   Where multiplied by Vn x 0.1+0.9b/h, the nominal shear capacities need not be reduced additionally by Section 4.3.4.2

Distribution of lateral forces to In-line Shear Walls



Section 4.3.3.4-Limitation of section

- All walls are of same materials and construction

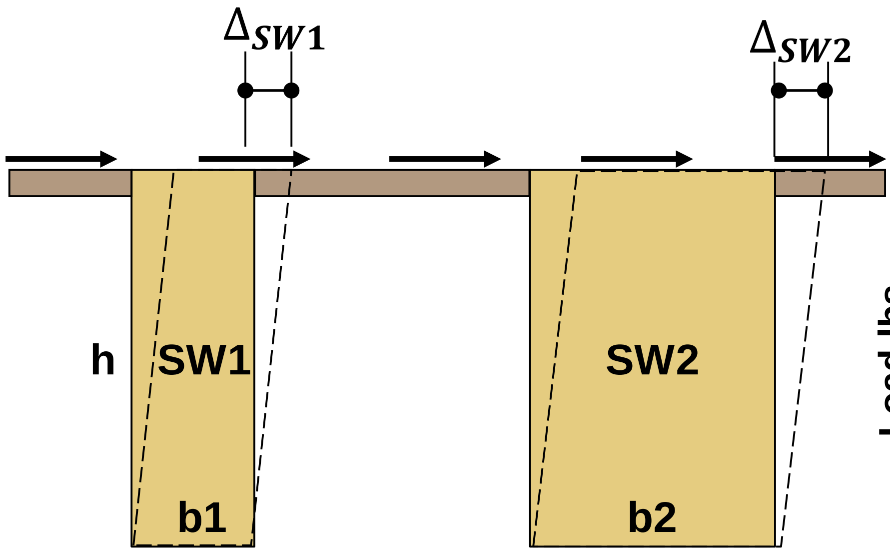
Method 1-Simplified Approach
Traditional Method

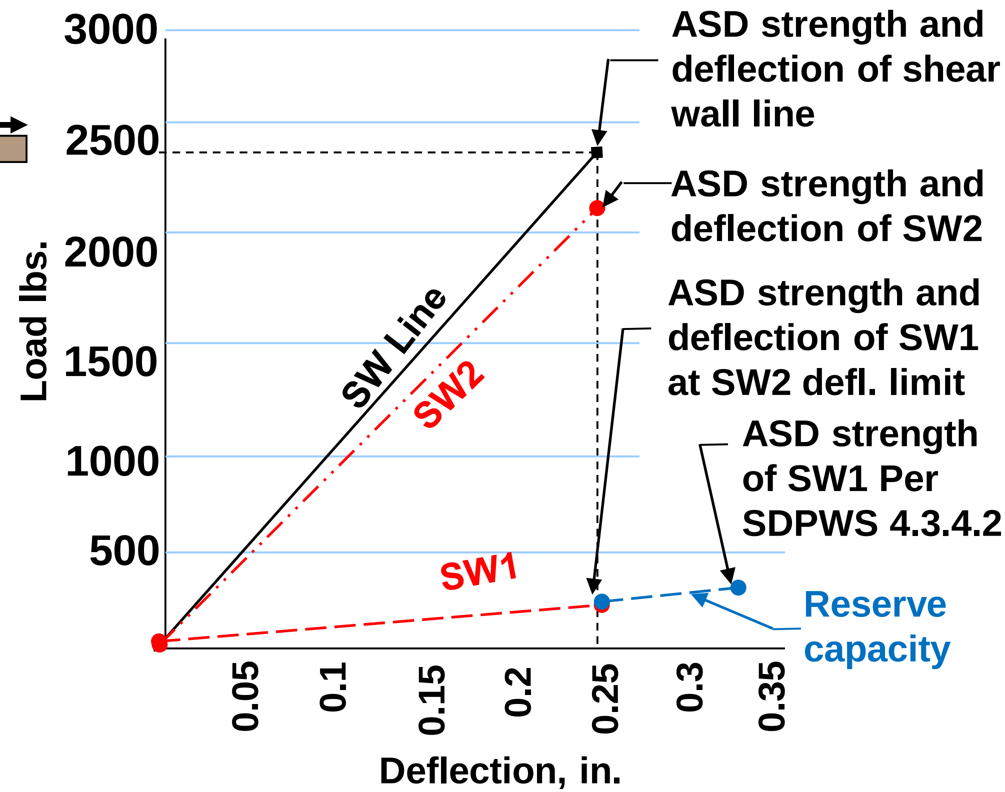
Example Calculation per Commentary (assumes same unit shear capacity)

SW1 and SW2

- Find nominal shear capacity
- Check aspect ratio-adjust per SDPWS 4.3.4.1 exception
- Aspect ratios > 2:1 use reduction factor $\frac{2b}{h}$ (further reductions in 4.3.4.2 are not required)
- Convert to ASD unit shear capacity
- Sum design strengths = $v_{sw1} \times b_1 + v_{sw2} \times b_2$ for line strength
- Will produce similar results to equal deflection method

Distribution of lateral forces to In-line Shear Walls





Method 2-Equal Deflection

Example Calculation per Commentary (assumes same unit shear capacity)

Method of Analysis

SW1 and SW2

- Find nominal unit shear capacity
- Check aspect ratio-adjust per SDPWS 4.3.4.2
- Aspect ratios > 2:1 use reduction factor 1.25-0.125h/b per 4.3.4.2
- Determine Ga and EA values
- Determine ASD unit shear capacity
- Calculate deflection of larger SW (SW2)

- Determine unit shear in smaller SW (SW1) that will produce same deflection

$$v_{sw1} = \frac{\Delta_{sw2}}{\dfrac{8h^3}{EAb_{sw1}} + \dfrac{h}{1000G_a} + \dfrac{h^2}{kb_{sw1}}}$$

- Sum design strengths = vsw1 x b1 + vsw2 x b2 for line strength

Construction Requirements

4.3.7.1 - 3x Stud: Requirements:

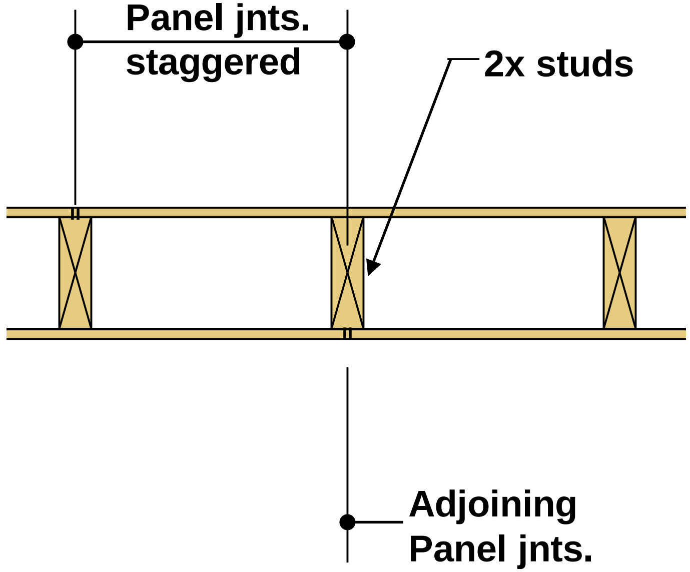
3x's at <u>adjoining panels</u> required when:

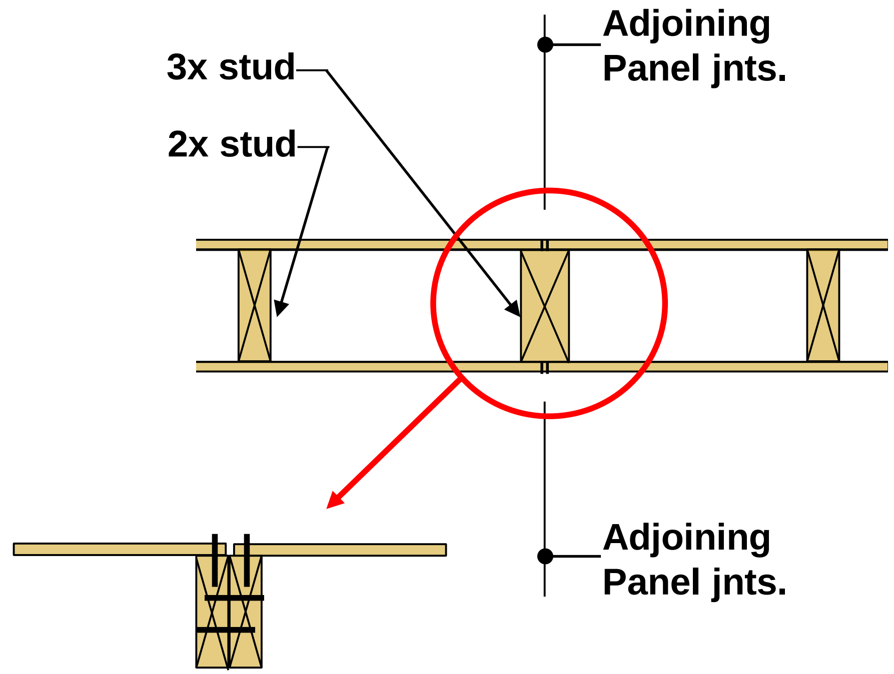
- **Nails are spaced 2" o.c.**
- **10d nails are spaced 3" o.c. and have penetration >1.5"**
- **Nominal unit shear capacity on either side of shear wall > 700 plf (SDC D-F)**

4.3.6.3.1 Adhesives:

Adhesive attachment of shear wall sheathing shall not be used alone, or in combination with mechanical fasteners.

Exception: Approved adhesive attachment systems shall be permitted for wind and seismic design in Seismic Design Categories A, B, and C where R = 1.5 and Ω_o = 2.5, unless other values are approved. Not permitted in SDC D.





Optional joint

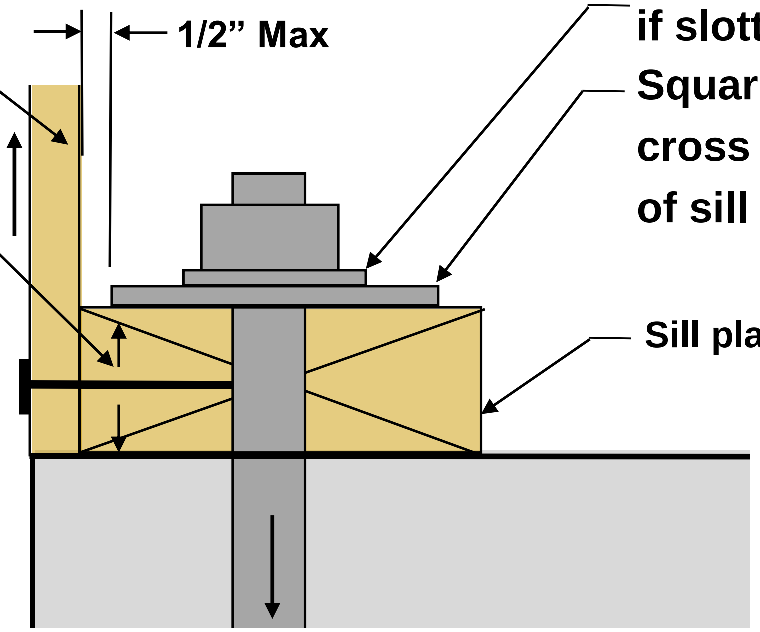
Panel joints Staggered

Panel joints Aligned

Footnote:

6. Where panels are applied on both faces of a shear wall and nail spacing is less than 6" on center on either side, panel joints shall be offset to fall on different framing members. Alternatively, the width of the nailed face of framing members shall be 3" nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.

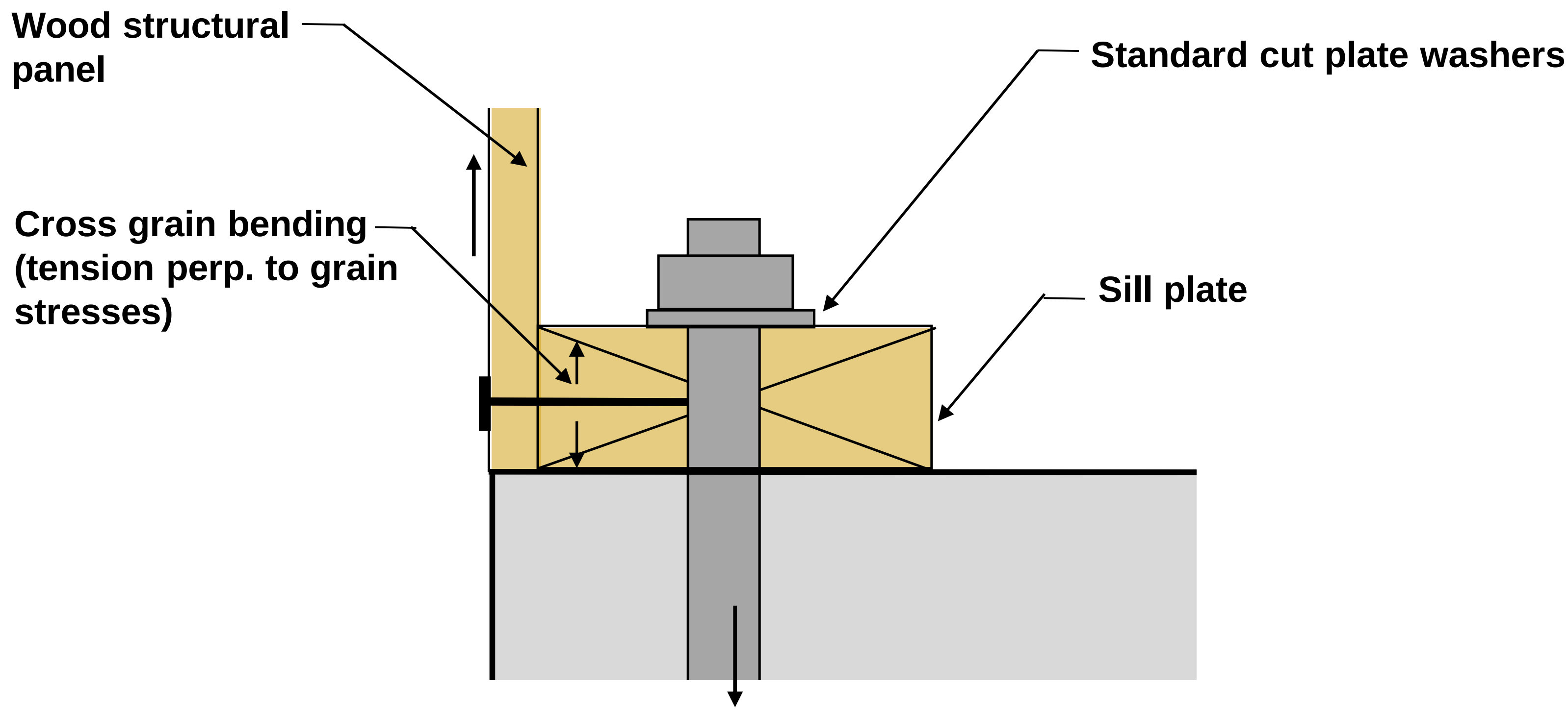
Shear Wall Anchorage



Square Plate Washers

4.3.6.4.3 Anchor Bolts:

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



Cut Washers

Standard cut washers

- Permitted to be used where anchor bolts are designed to resist shear only and the following requirements are met:
    - a) The shear wall is designed segmented wall with required uplift anchorage at shear wall ends sized to resist overturning <u>neglecting</u> DL stabilizing moment.
    - b) Shear wall aspect ratio, h:b, does not exceed 2:1.
    - c) The <u>nominal</u> unit shear capacity of the shear wall does not exceed 980 plf for seismic or 1370 plf for wind.

Presentation Topics

- Shear Wall Types
- Shear wall Anchorage
- SDPWS Code Requirements
- **Complete Load Paths**
- Offset Shear Walls and Relevant Code Requirements
- Mid-rise Diaphragms and Tall Shear Walls

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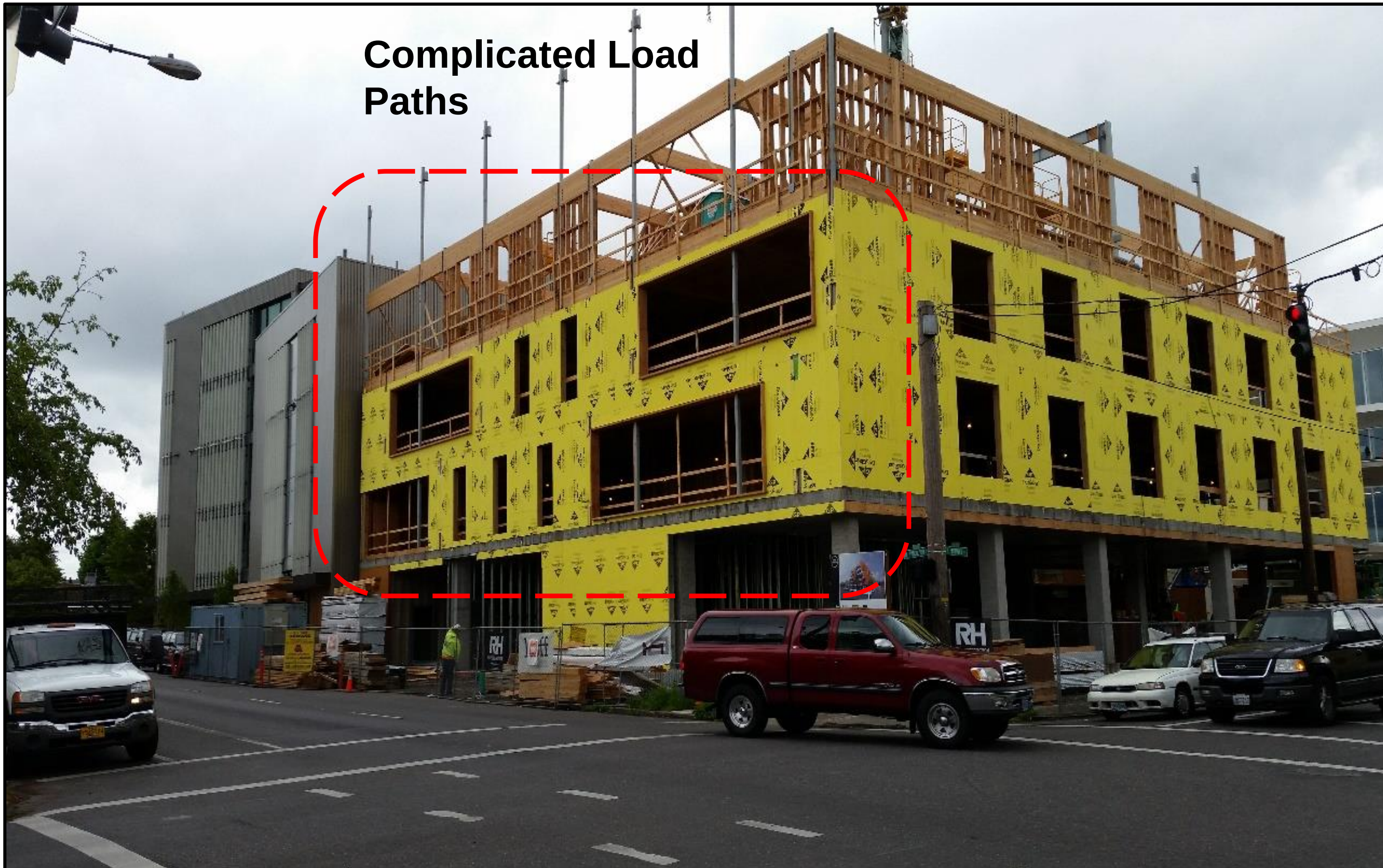
Detailing for Continuous Load Paths

Karuna I
Holst Architecture

Continuous Stacked In-line Load Paths

Photo: Terry Malone

Detailing for Continuous Load Paths

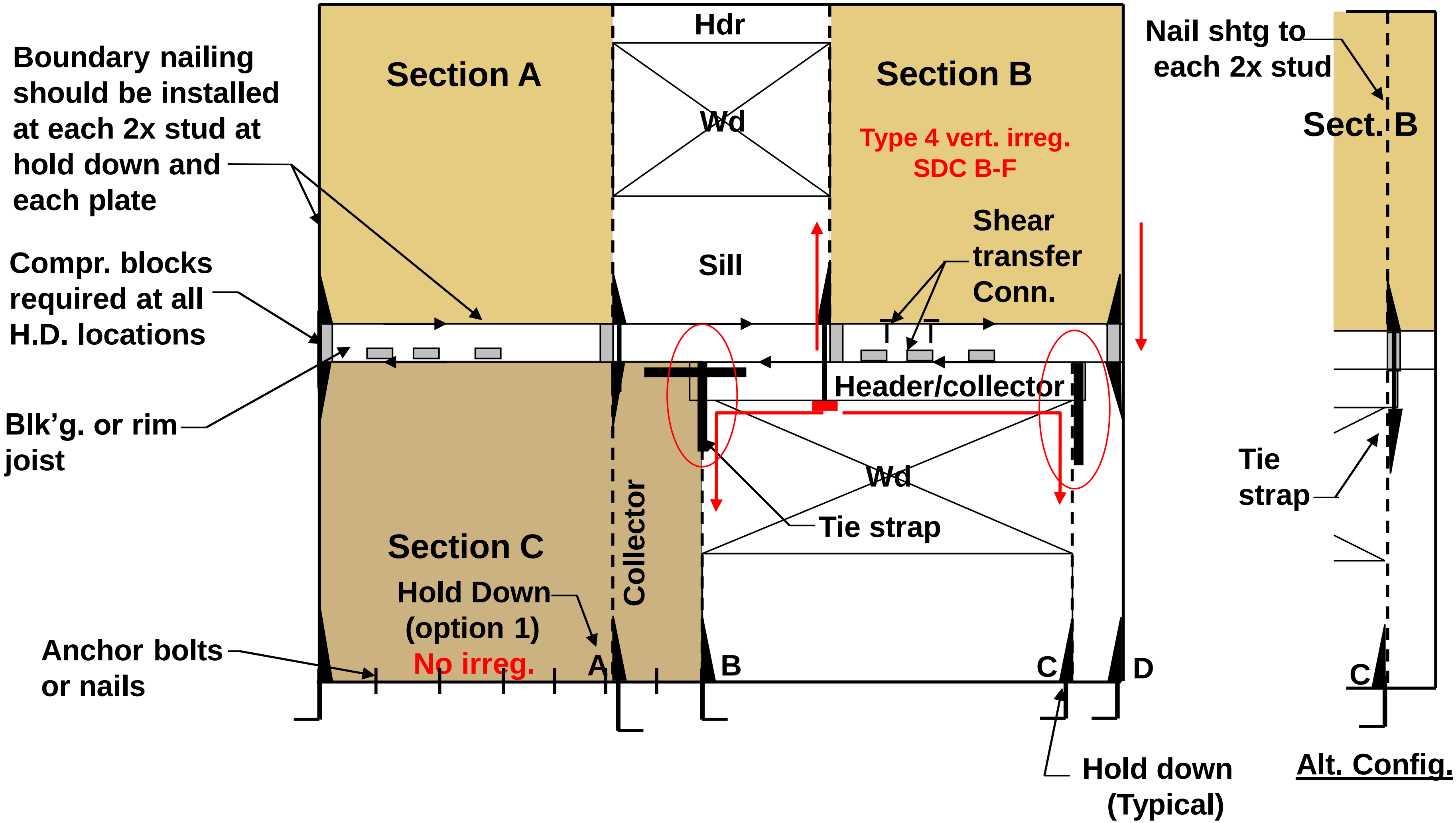


Karuna I
Holst Architecture

Dis-continuous Load Paths

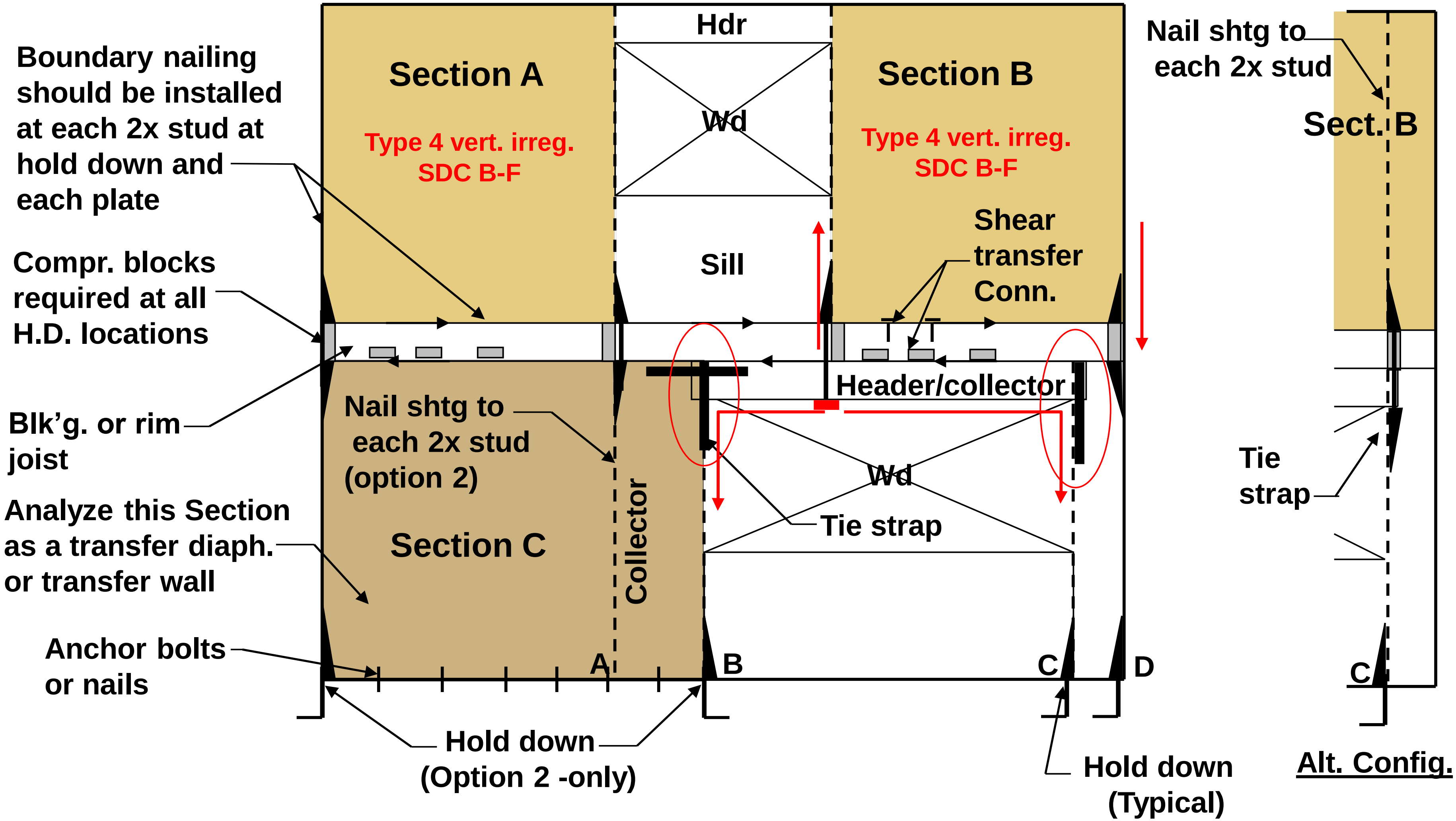
Photo: Terry Malone

In-plane Offset Segmented Shear Walls

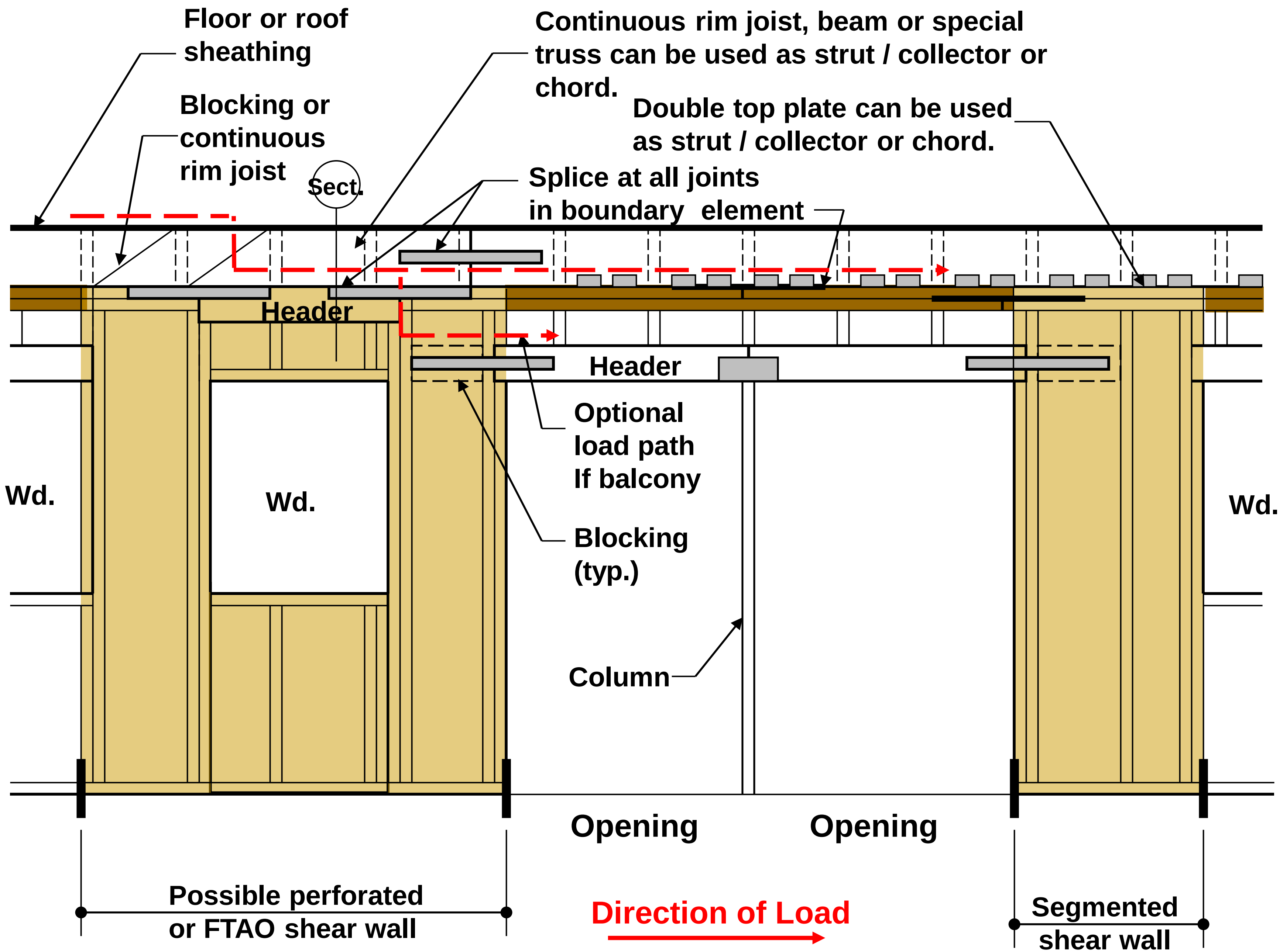


- Type 4 Vertical Irregularity, in-plane offset
- ASCE 7-10 12.3.3.3 Elements supporting discontinuous walls SDC B-F
- ASCE 7-10 12.3.3.4 25% increase in Fpx SDC D-F (connections)

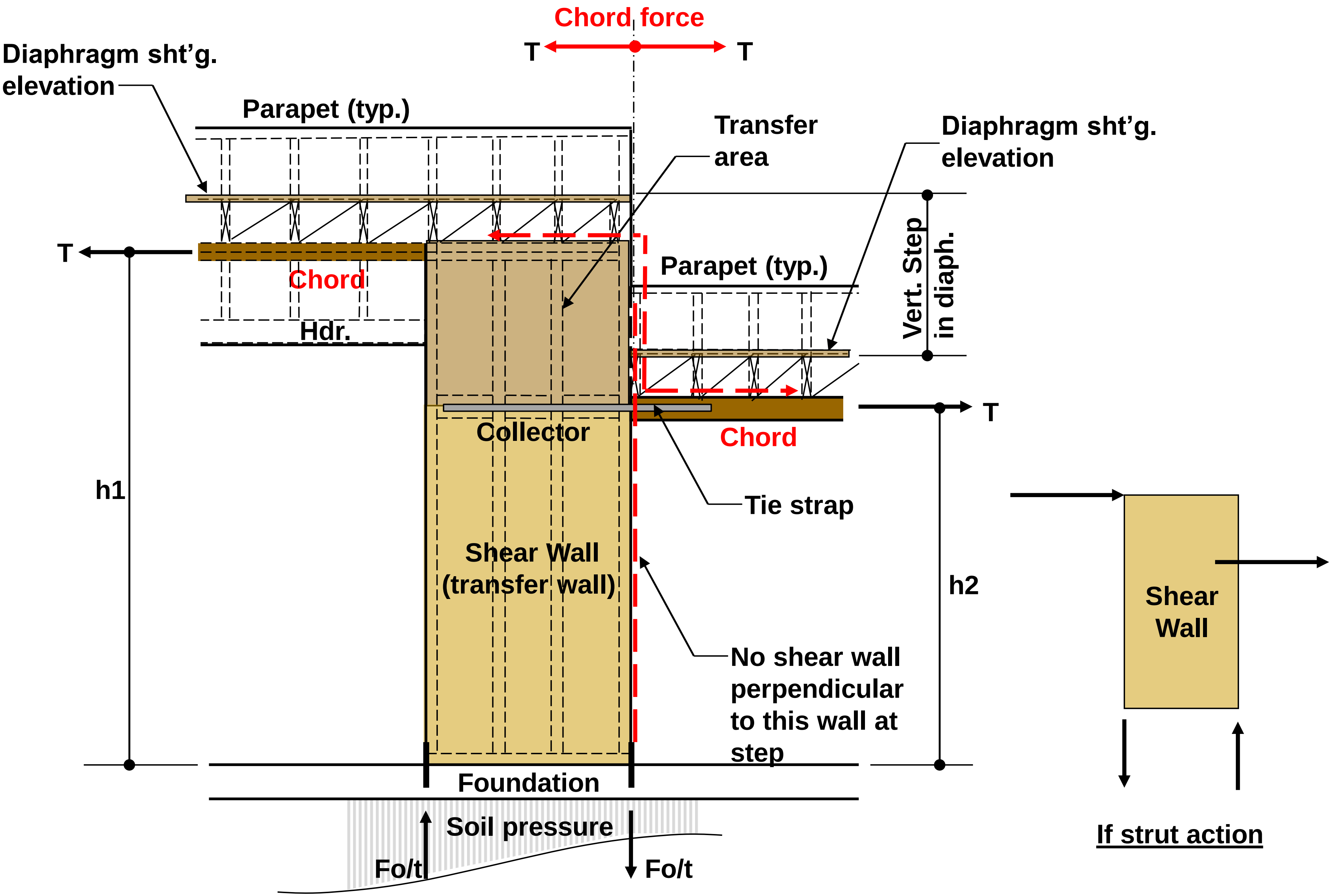
In-plane Offset Segmented Shear Walls



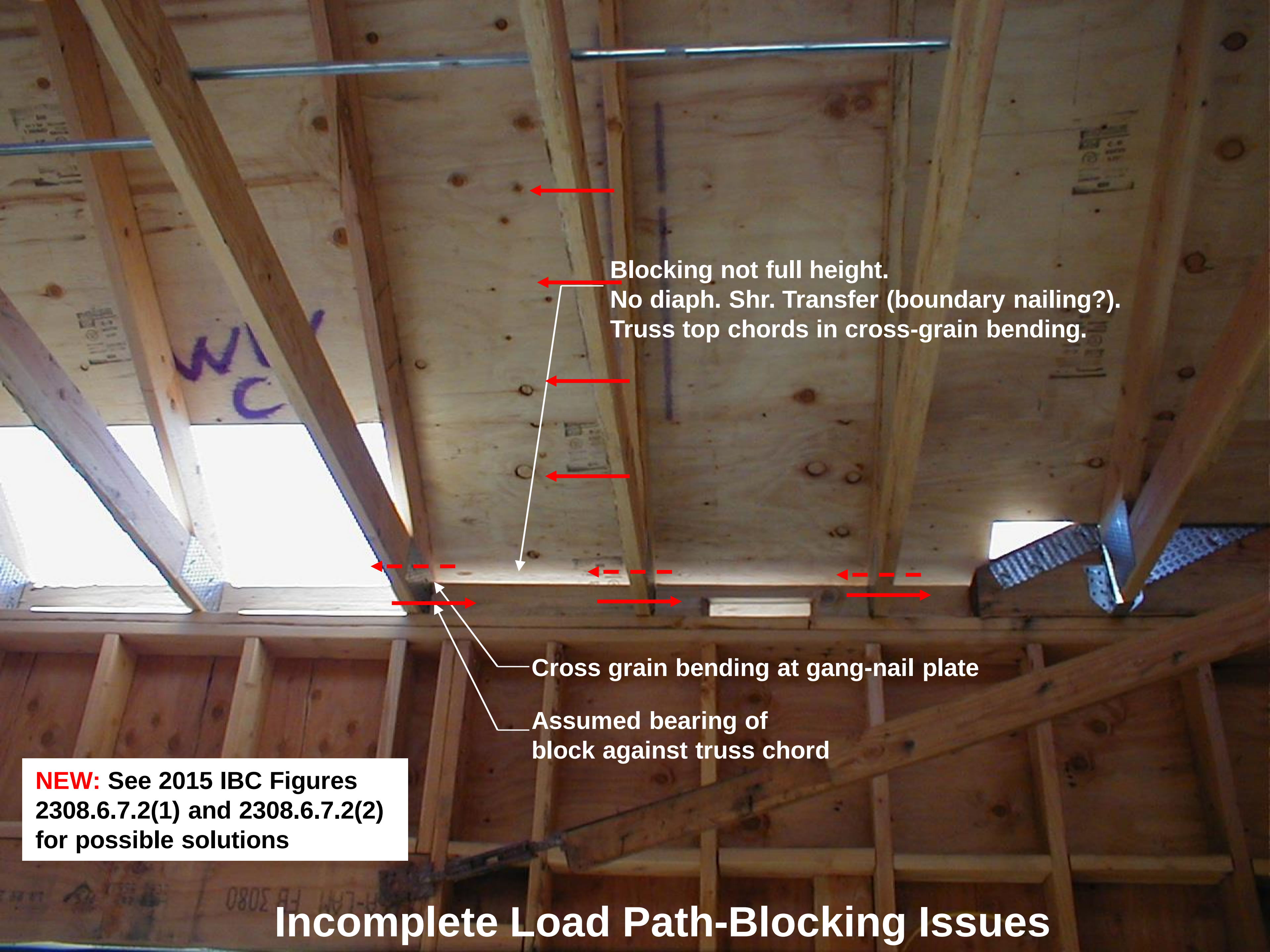
- Type 4 Vertical Irregularity, in-plane offset
- ASCE 7-10 12.3.3.3 Elements supporting discontinuous walls SDC B-F
- ASCE 7-10 12.3.3.4 25% increase in Fpx SDC D-F (connections)



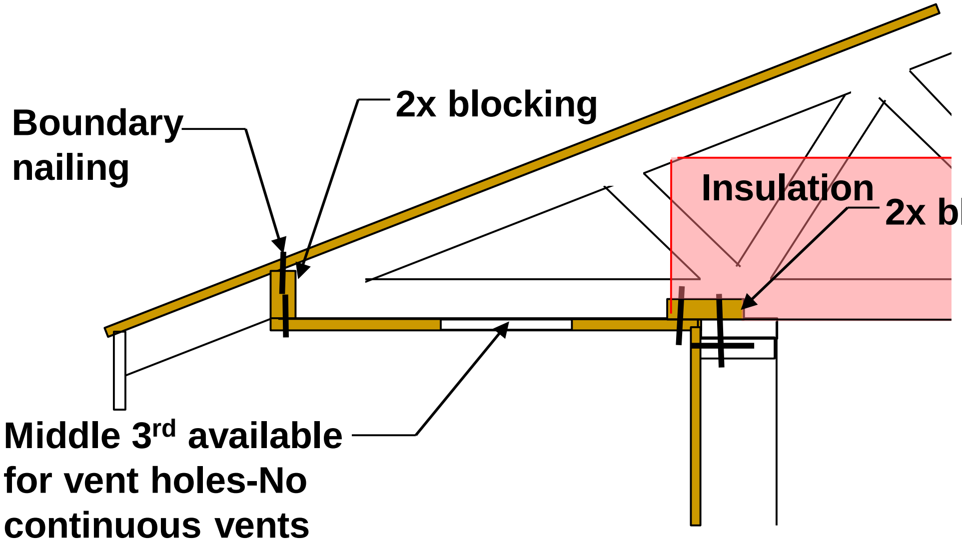
Examples of Drag Struts, Collectors and chords at Exterior Boundaries



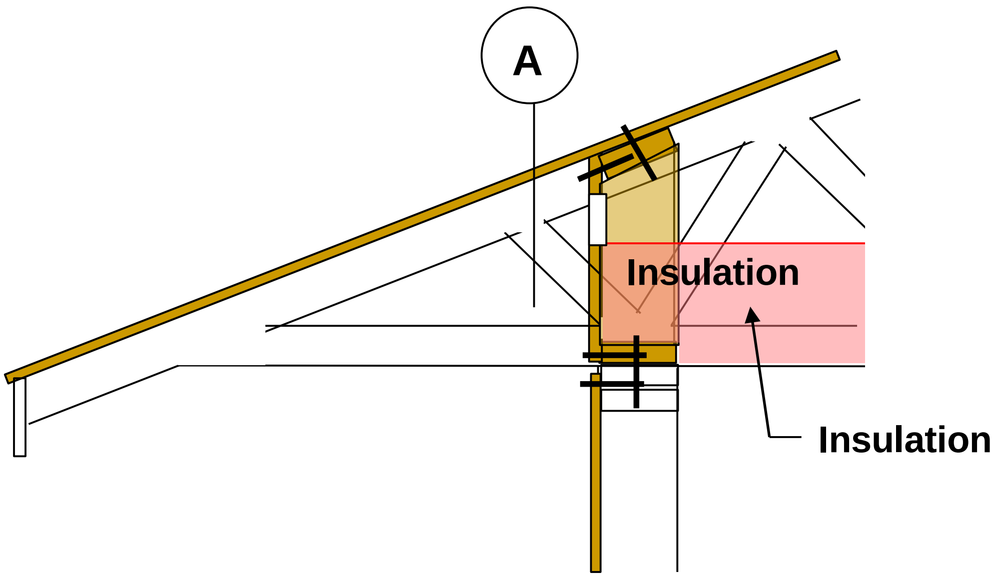
Complete Load Path to Foundation
Roof at Different Elevations-Chord Forces


Blocking not full height.
No diaph. Shr. Transfer (boundary nailing?).
Truss top chords in cross-grain bending.
Cross grain bending at gang-nail plate
Assumed bearing of
block against truss chord
NEW: See 2015 IBC Figures 2308.6.7.2(1) and 2308.6.7.2(2) for possible solutions
Incomplete Load Path-Blocking Issues

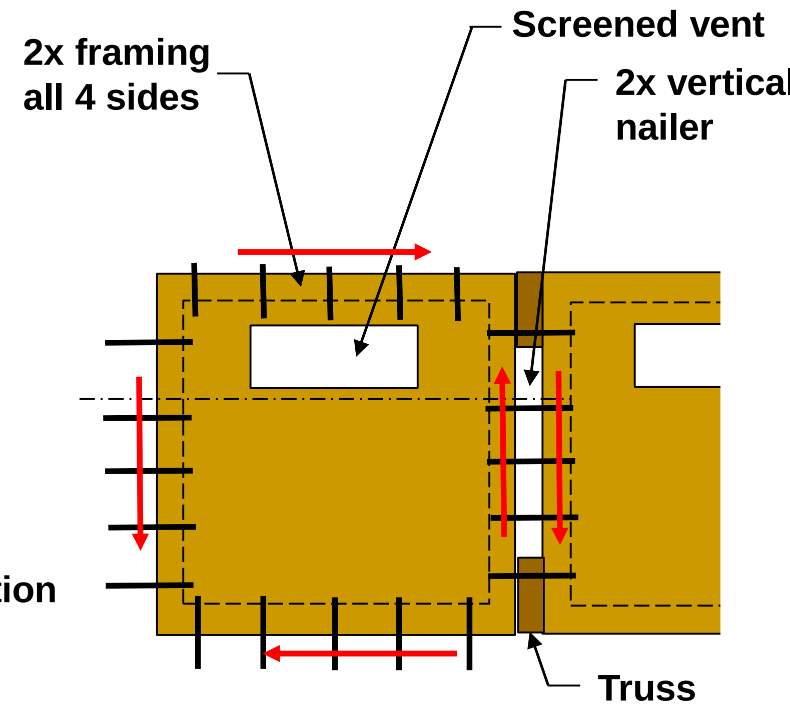
Venting Options



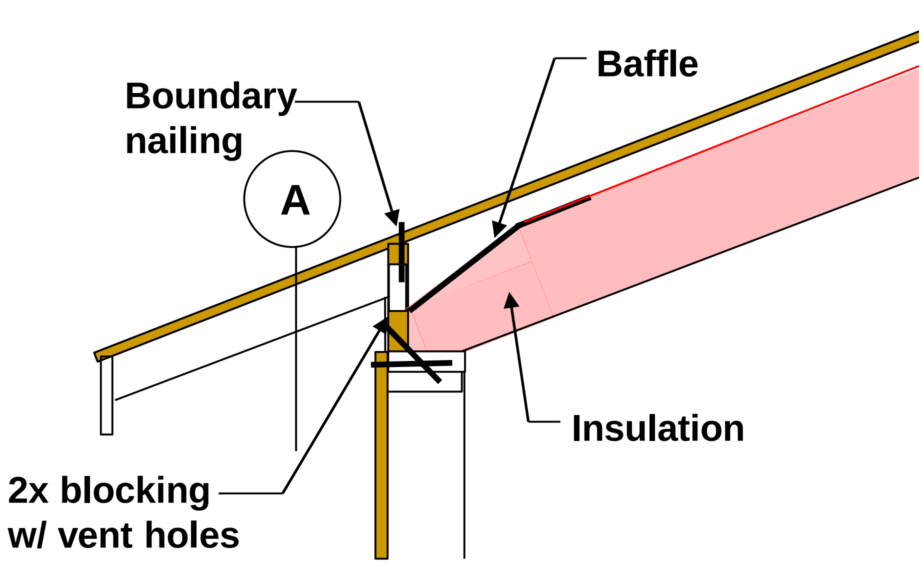
2015 IBC Figure 2308.6.7.2(1)

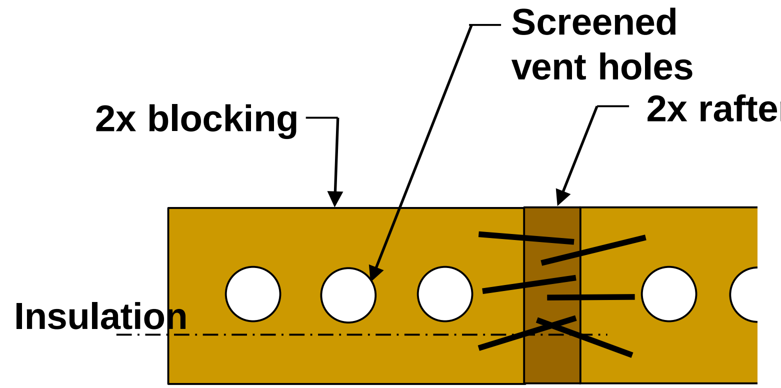


2015 IBC Figure 2308.6.7.2(2)

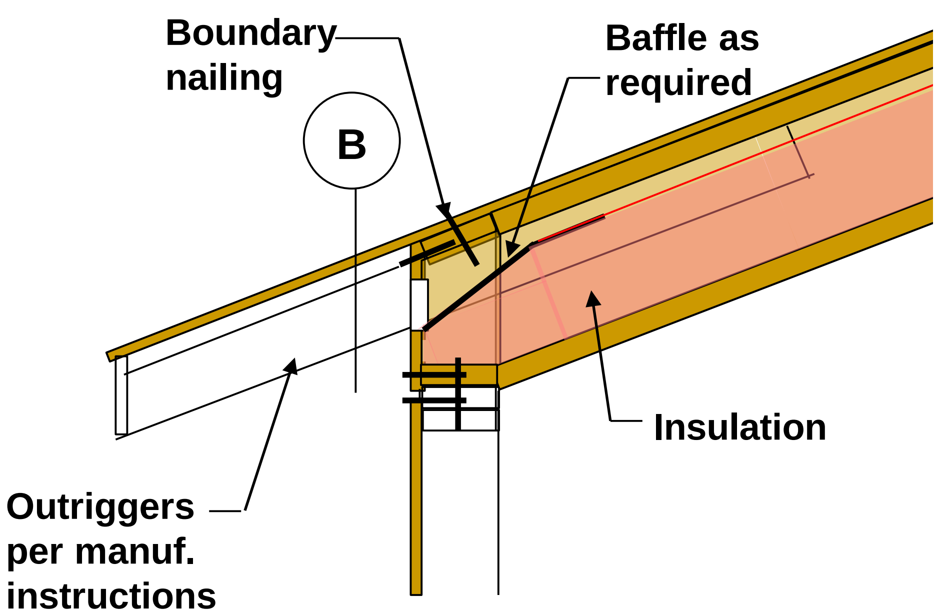


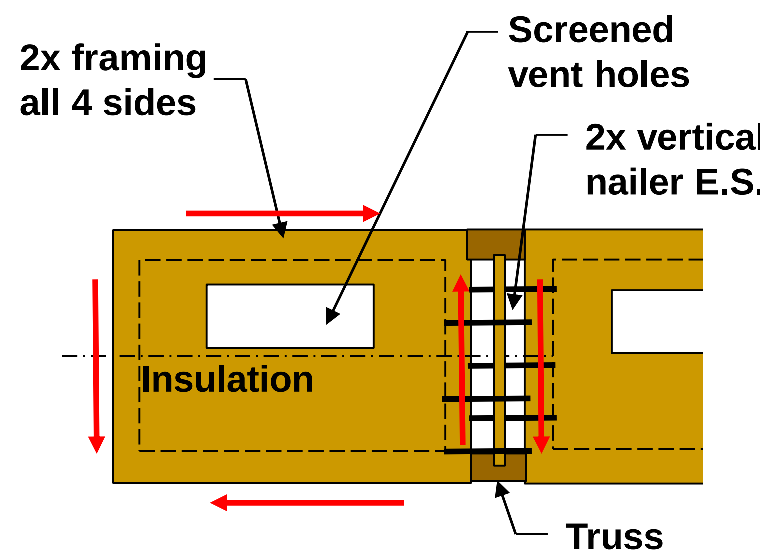
A Typical shear panel





A **Typical Solid Blocking**





B **Typical shear panel**

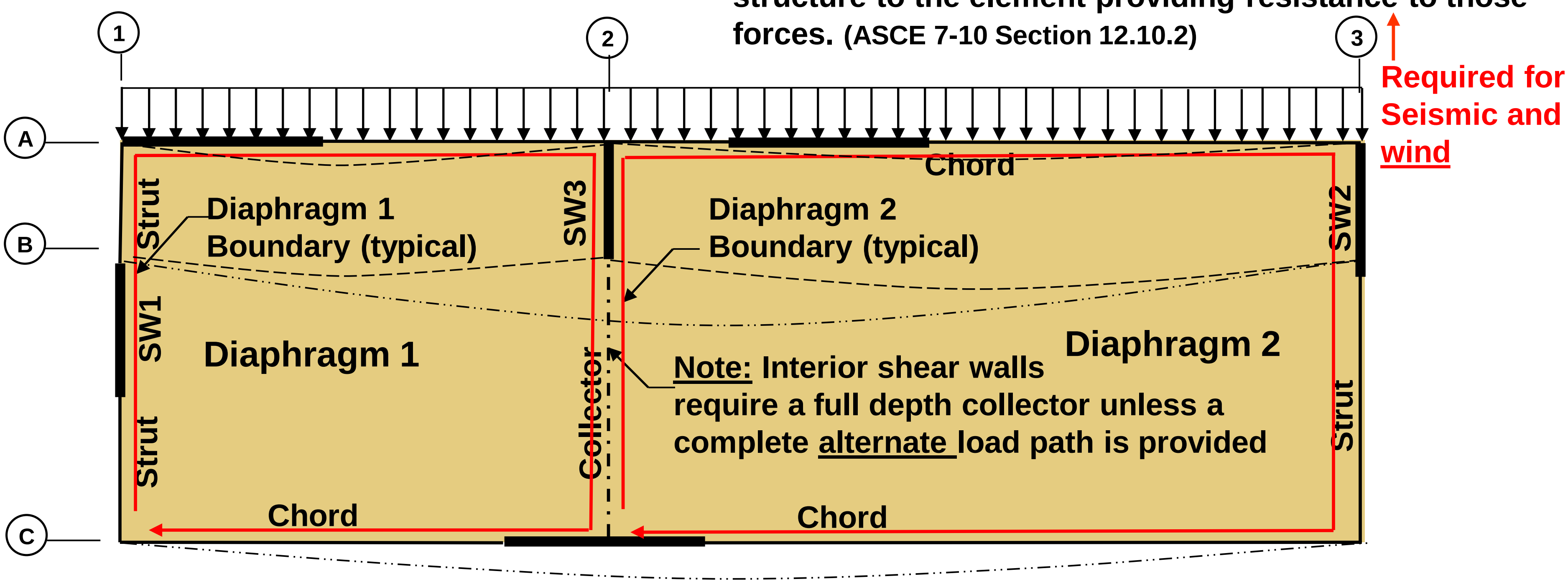
Venting Options

Diaphragm Boundary Elements and Interior Shear Walls

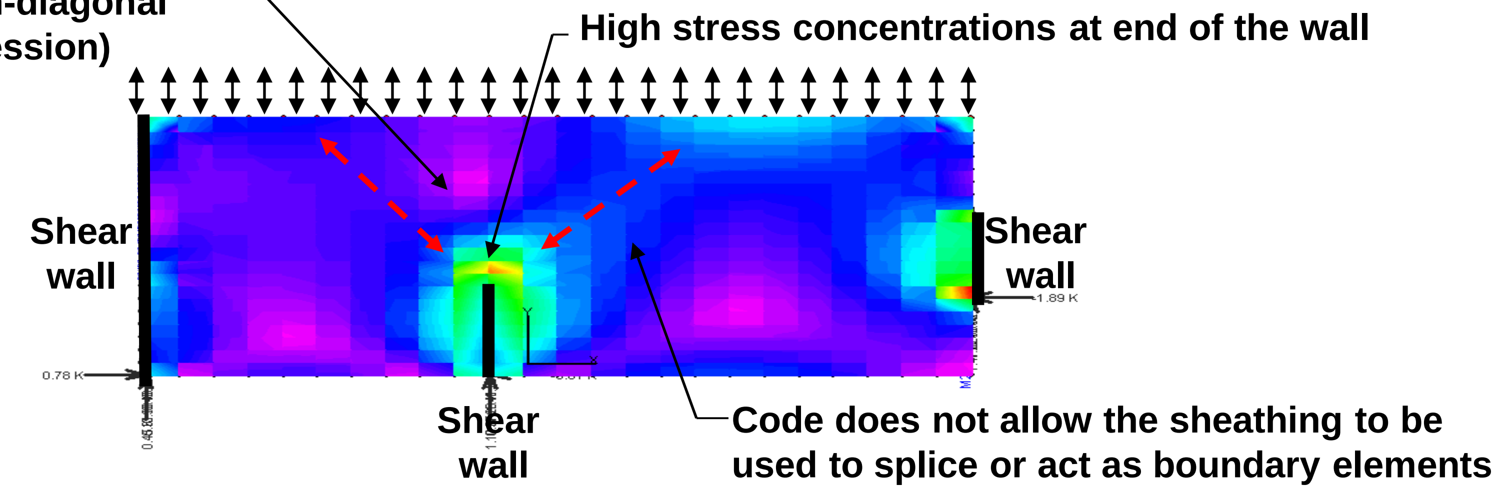
Fundamental Principles:
A shear wall is a location where diaphragm forces are resisted (supported), and therefore defines a diaphragm boundary location.

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

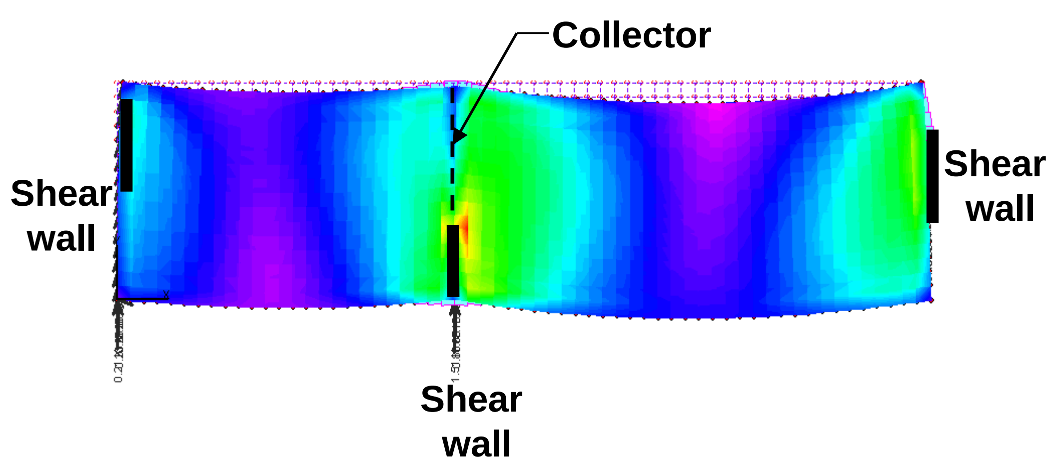
- **Diaphragm Boundary Elements:**
  - **Chords, drag struts, collectors, Shear walls, frames**
  - **Boundary member locations:**
    - **Diaphragm and shear wall perimeters**
    - **Interior openings**
    - **Areas of discontinuity**
    - **Re-entrant corners.**
- **Diaphragm and shear wall sheathing shall not be used to act as or splice boundary elements. (SDPWS 4.1.4)**
- **Collector elements shall be provided that are capable of transferring forces originating in other portions of the structure to the element providing resistance to those forces. (ASCE 7-10 Section 12.10.2)**



Note:
Diaphragm sections act as notched beams (shear distribution-diagonal tension or compression)



Shear Distribution if No Collector



Shear Distribution if Continuous Collector

Diagram Boundary Elements and Interior Shear Walls

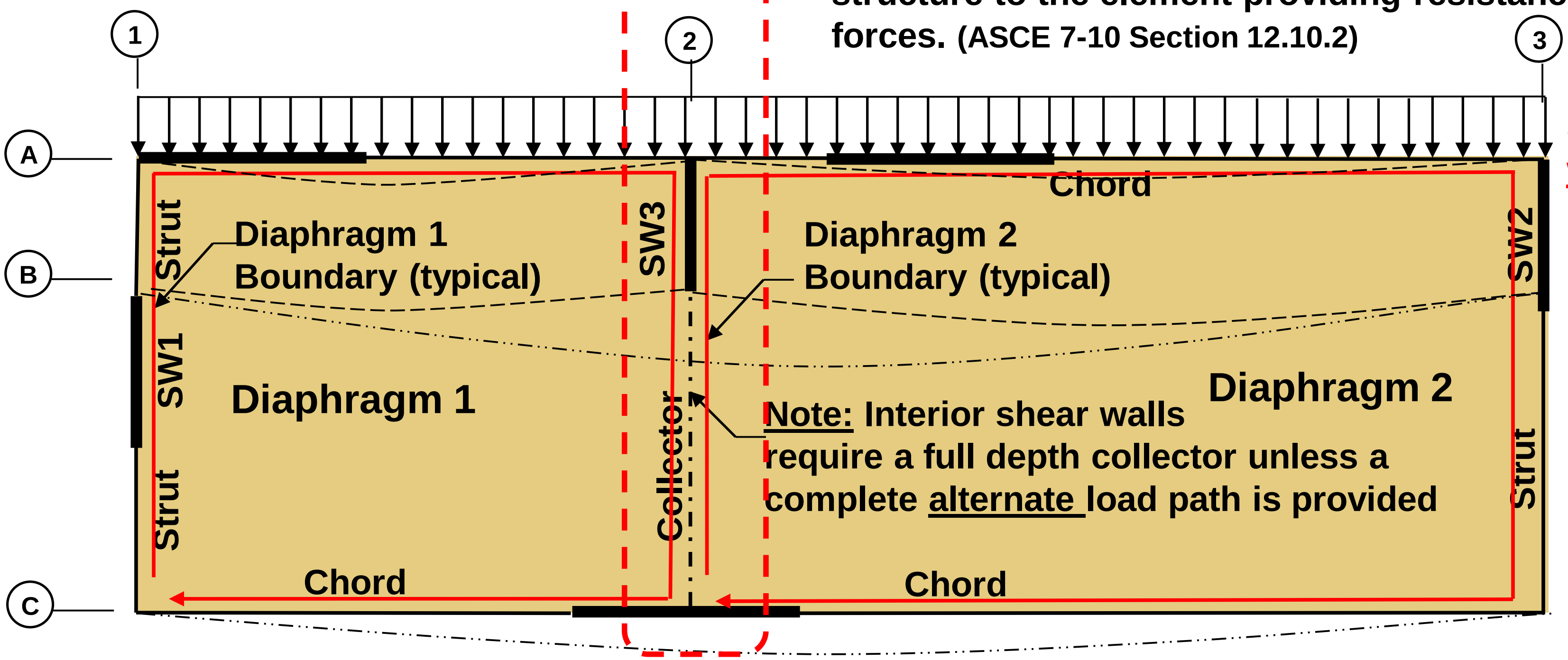
<u>Fundamental Principles:</u>
A shear wall is a location where diaphragm forces are resisted (supported), and therefore defines a diaphragm boundary location.

<u>Note:</u> All edges of a diaphragm shall be supported by a boundary element. (ASCE 7-10 Section 11.2)

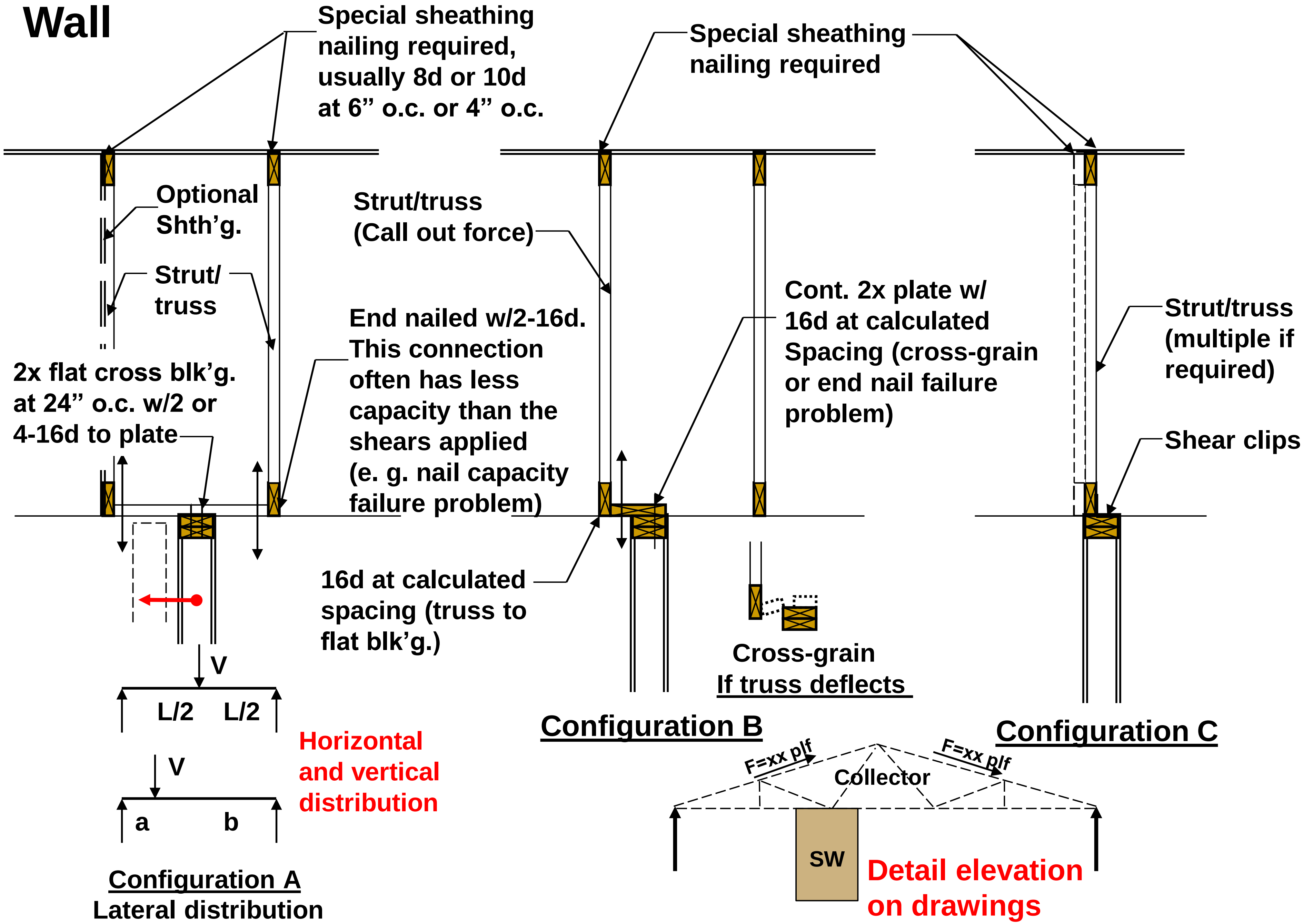
- **<u>Diaphragm Boundary Elements:</u>**
  - **Chords, drag struts, collectors, Shear walls, frames**
  - **Boundary member locations:**
    - **Diaphragm and shear wall perimeters**
    - **Interior openings**
    - **Areas of discontinuity**
    - **Re-entrant corners.**
- **Diaphragm and shear wall sheathing shall not be used to act as or splice boundary elements. (SDPWS 4.1.4)**
- **Collector elements shall be provided that are capable of transferring forces originating in other portions of the structure to the element providing resistance to those forces. (ASCE 7-10 Section 12.10.2)**

Required for Seismic and <u>wind</u>

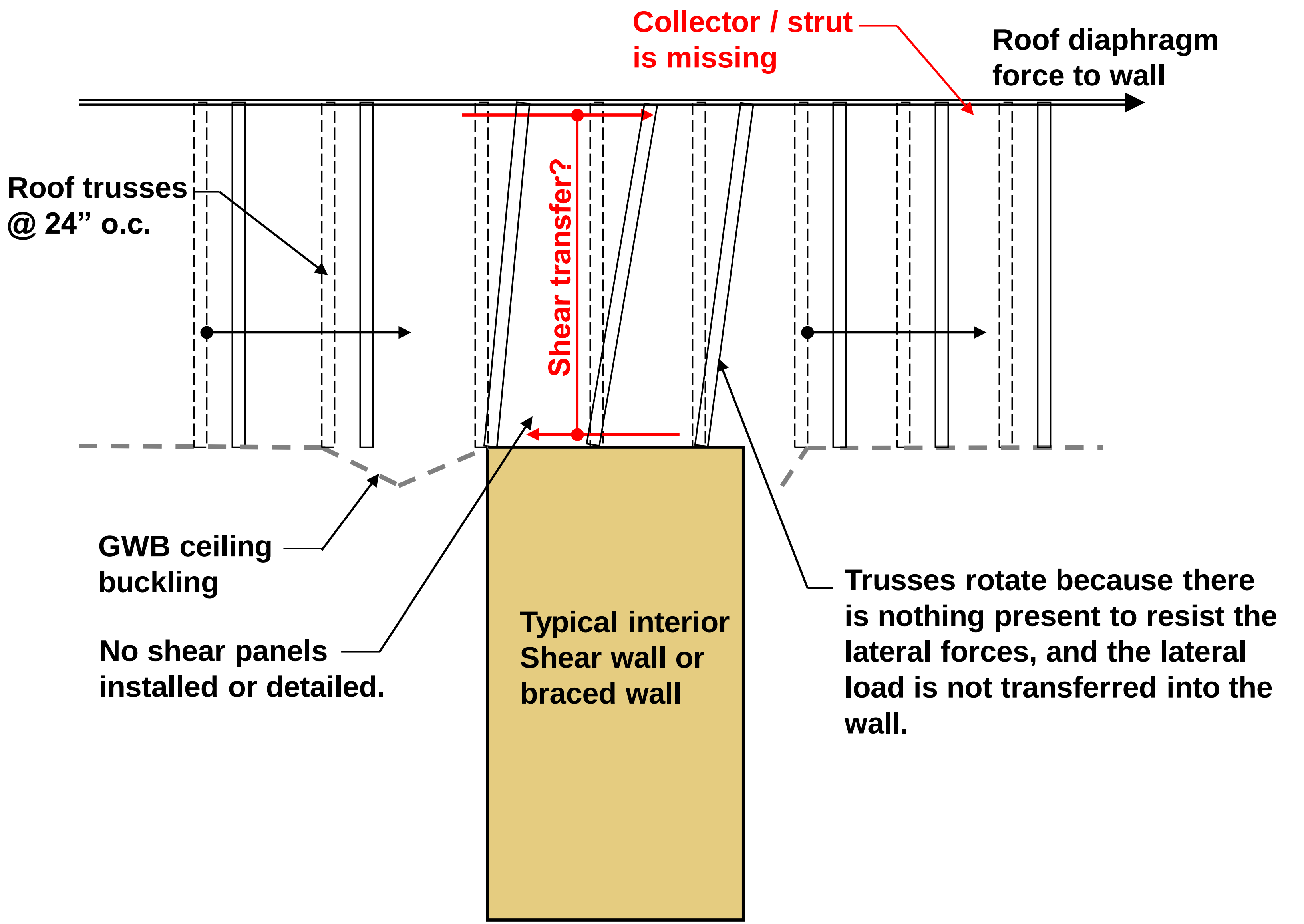
Let's take a look at load paths to the shear wall at grid line 2.



Typical Collector Framing and Connection Parallel to Shear Wall

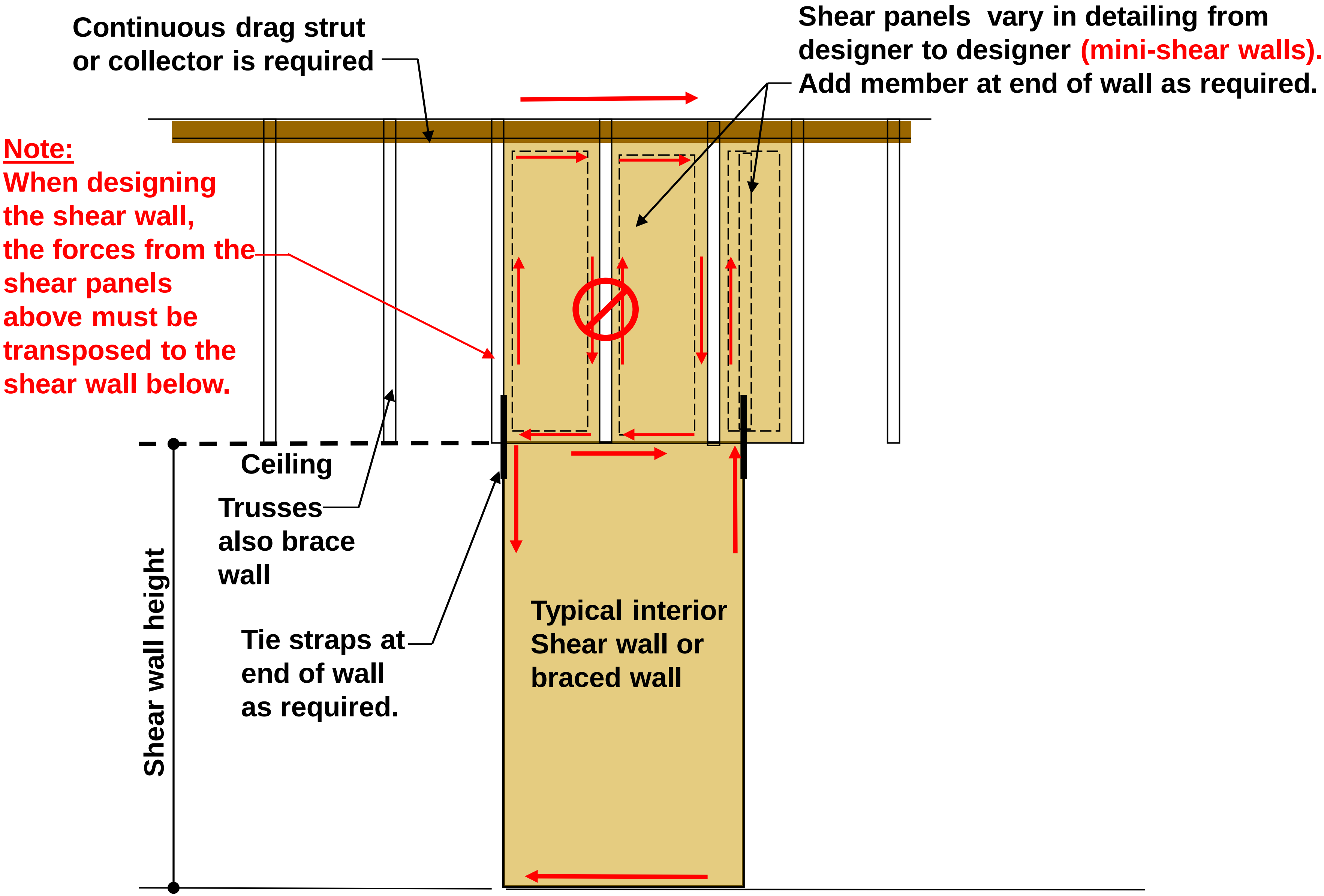


Corridor Shear Wall Line



Typical Collector Framing and Connections- Roof Framing Perpendicular to Shear Wall

Corridor Shear Wall with Shear Panels and Collector Added

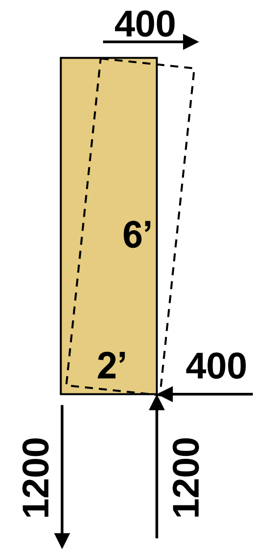


Framing Perpendicular to Wall

The shear panels shown are 24" wide by 6'-0" high.
The framing could easily be 16" wide by 15'-0" high or greater.

Wood shear panels between trusses

If v=200 plf



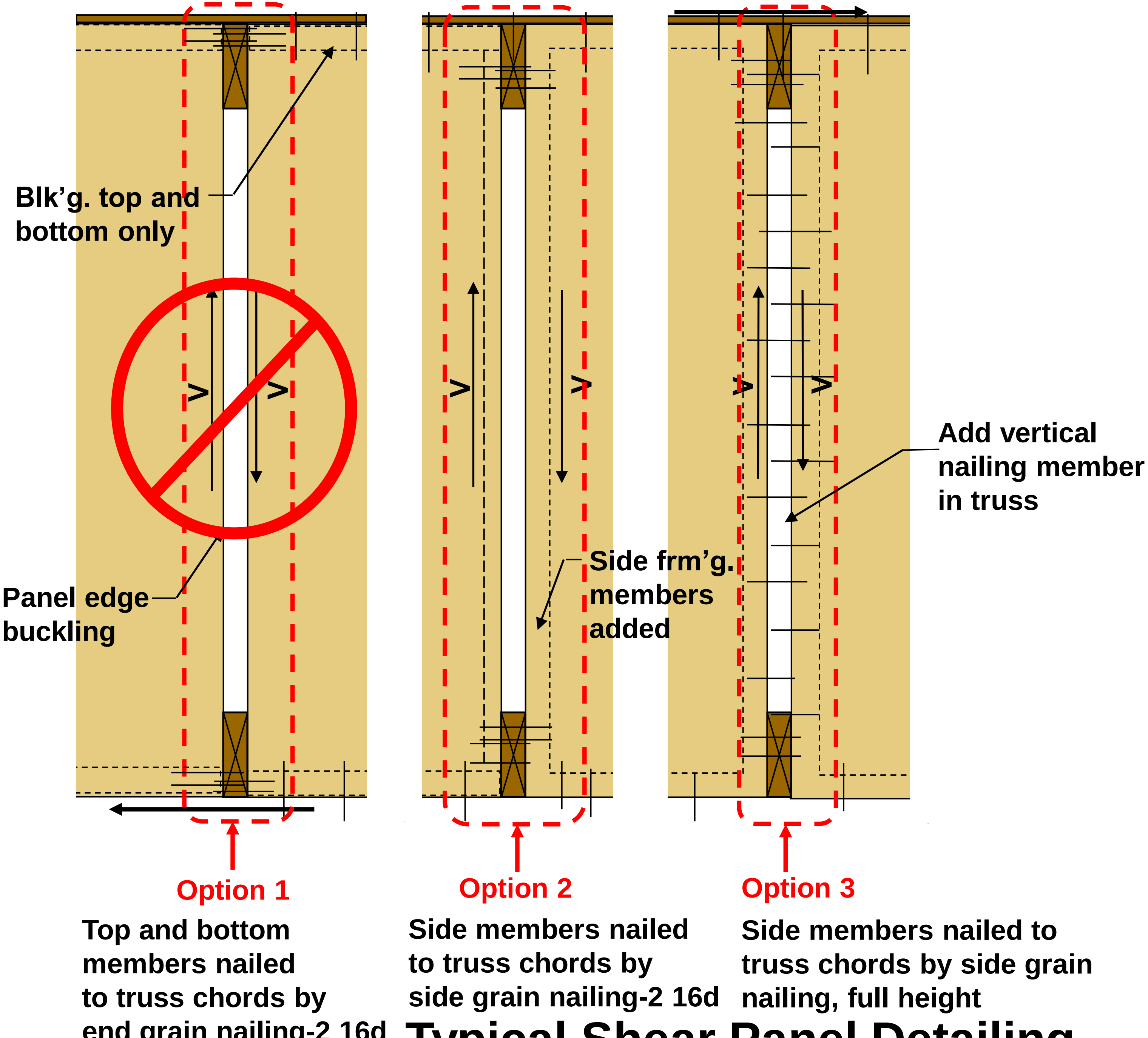
If 3:1 A/R

Wall studs at 8" o.c.

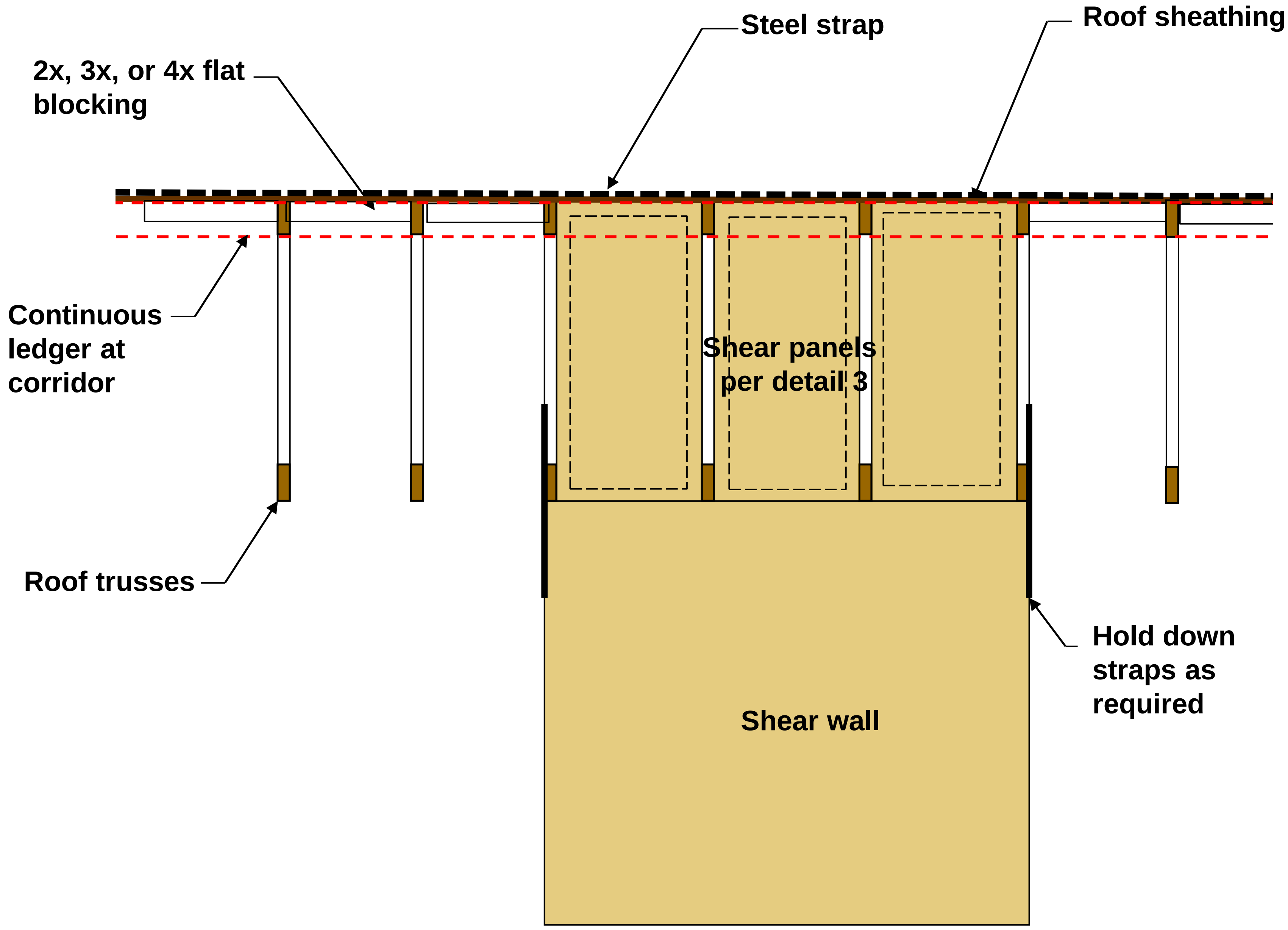
Does not appear to have vertical blocking at truss
(no shear transfer for vertical shear force).

Photo-Typical Shear Panels

Courtesy of Willdan Engineering

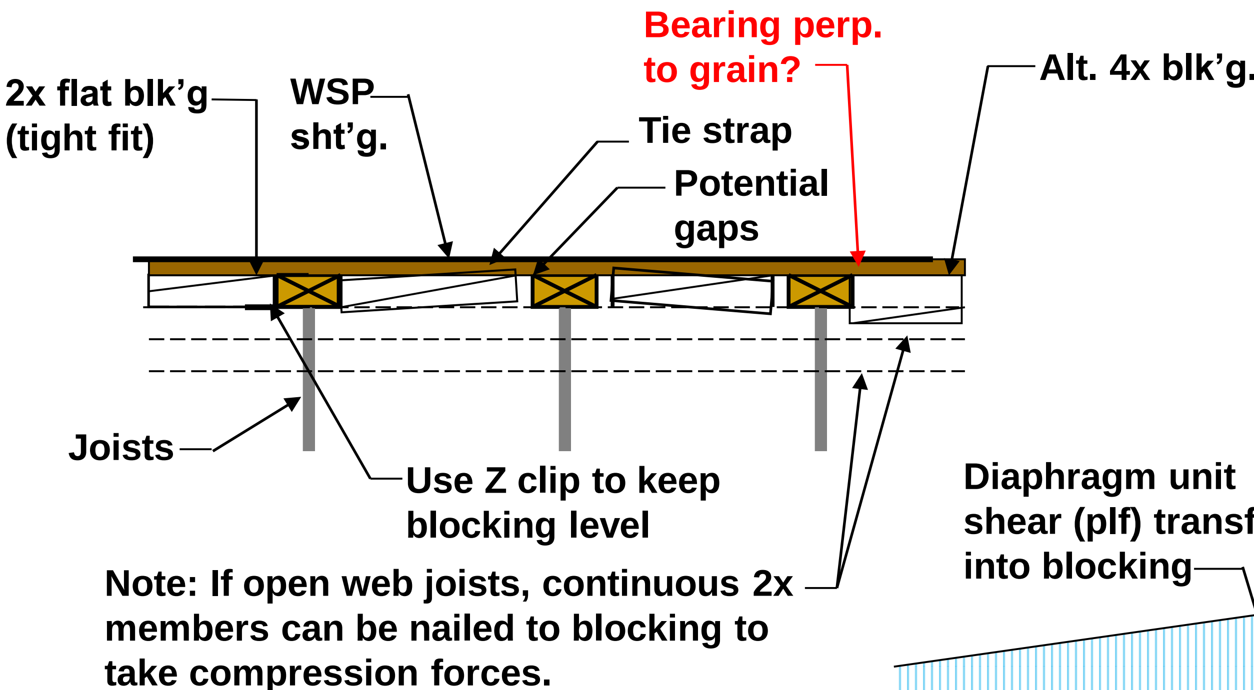


Typical Shear Panel Detailing

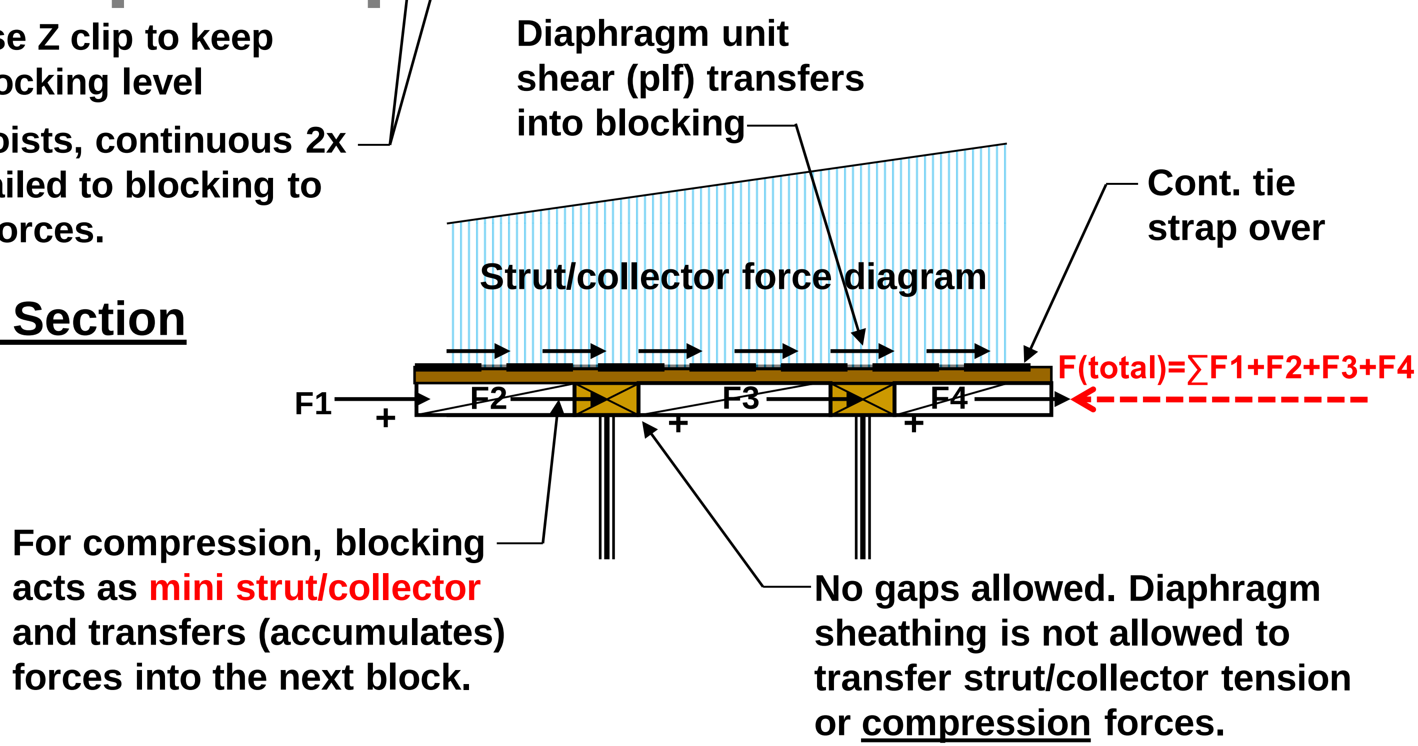


Collector Framing Option 1

Example of Partial Strut/Collector

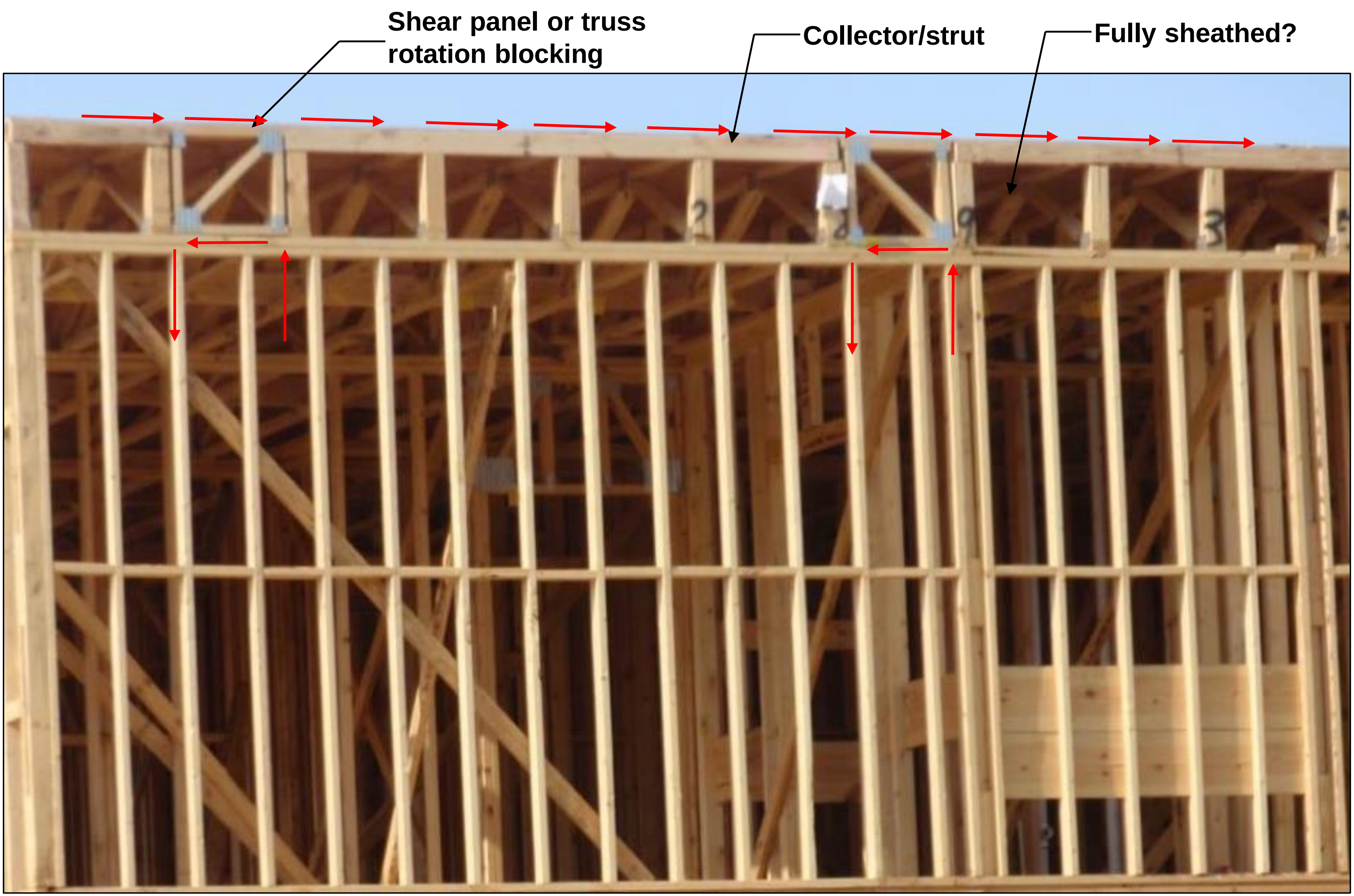


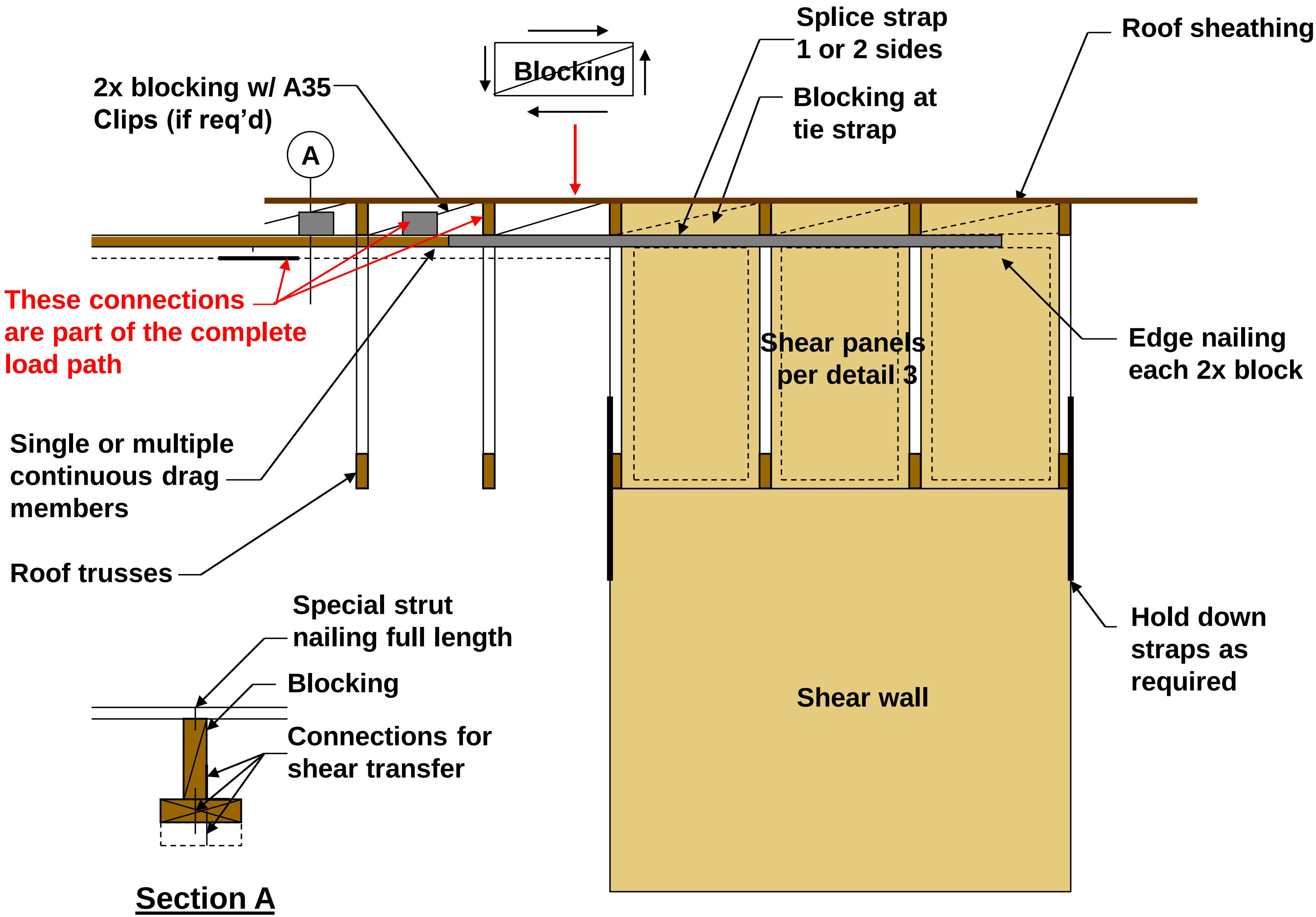
Typical Section



Collector force distribution

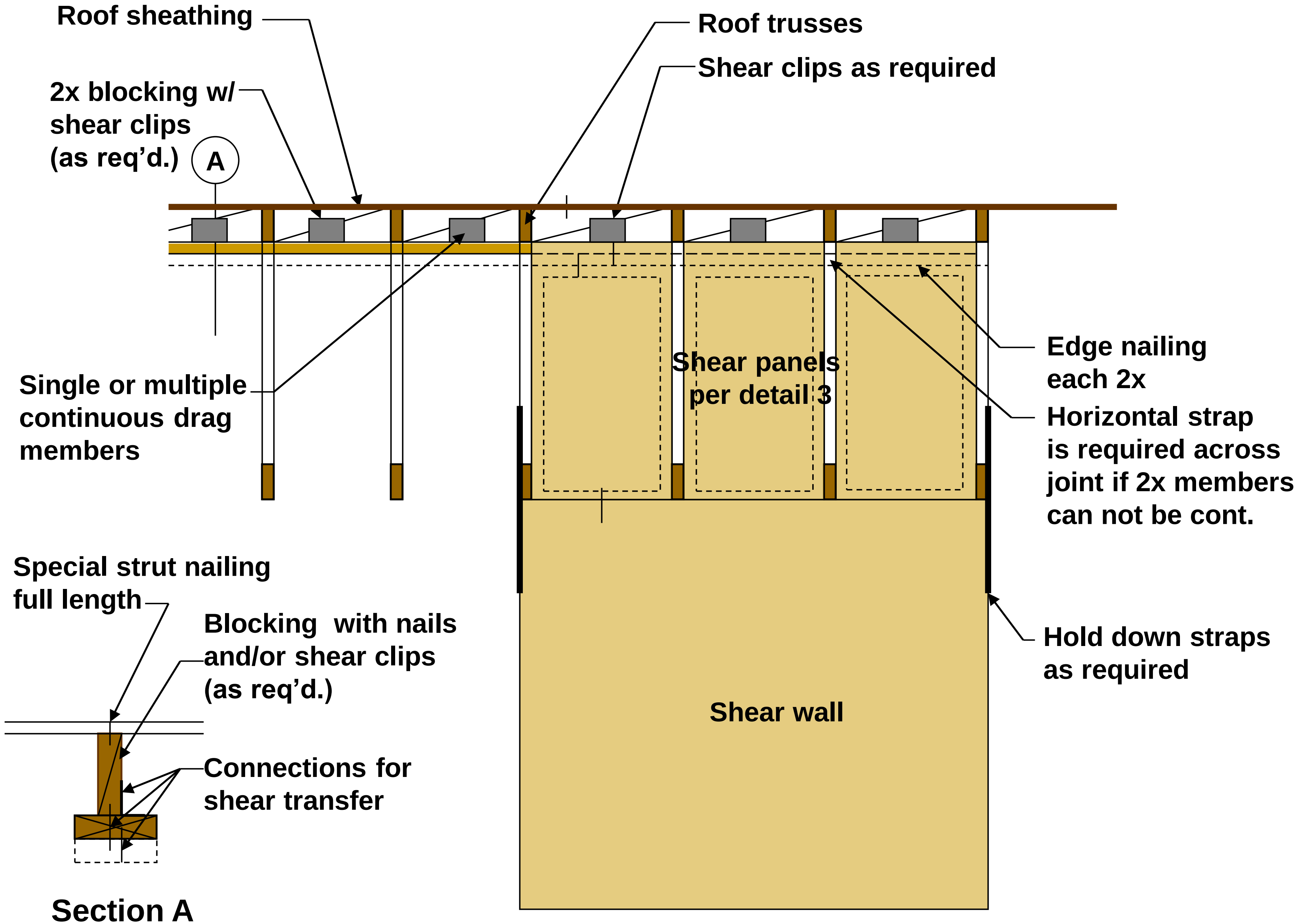
Corridor Wall Line?





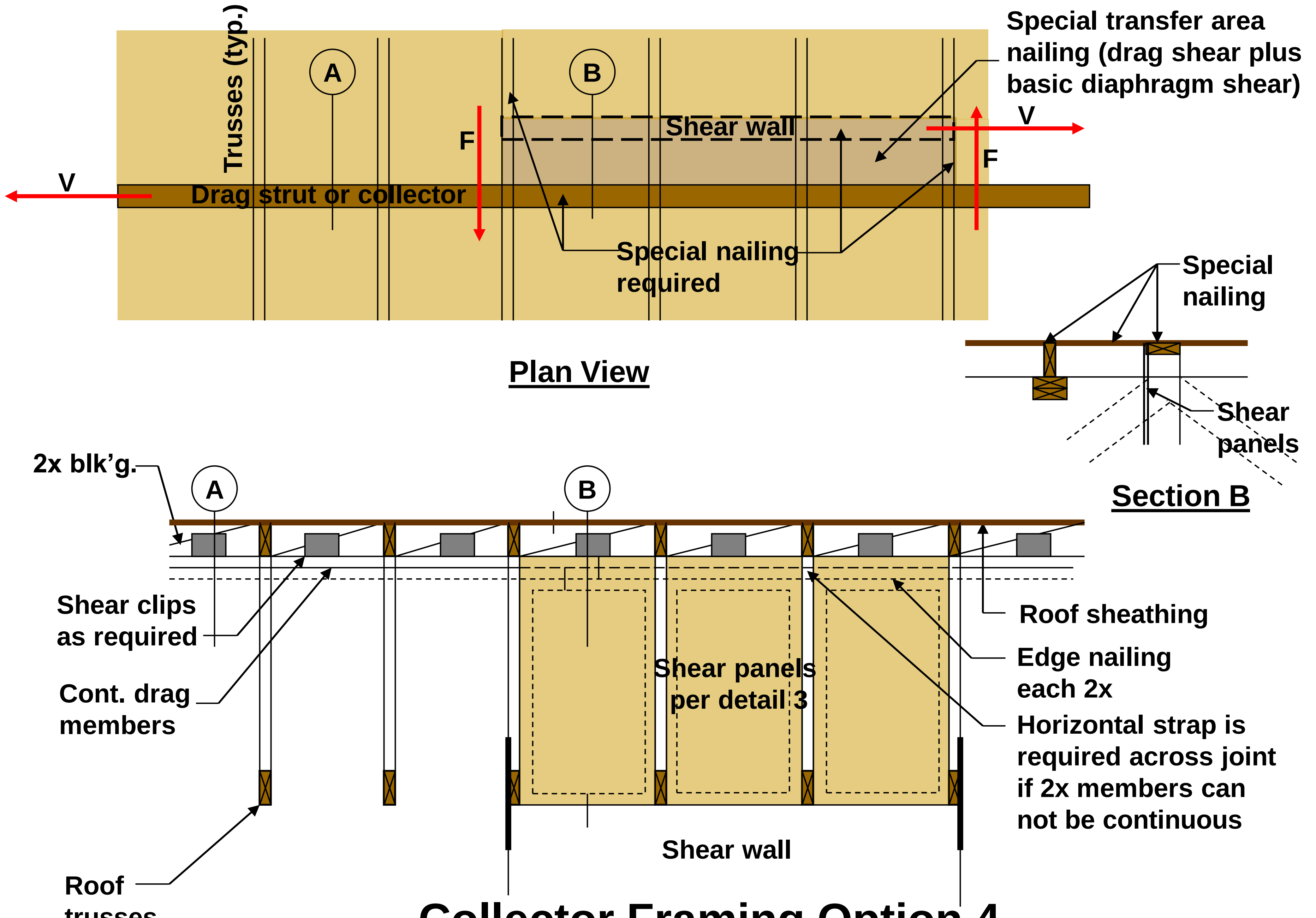
Collector Framing Option 2

(Assuming the collector does not fall at a truss joint)



Collector Framing Option 3

(Assuming the collector does not fall at a truss joint)



Collector Framing Option 4

(If the collector falls at a truss joint)

Presentation Topics

- Shear Wall Types
- Shear wall Anchorage
- SDPWS Code Requirements
- Complete Load Paths
- **Offset Shear Walls and Relevant Code Requirements**
- Mid-rise Diaphragms and Tall Shear Walls

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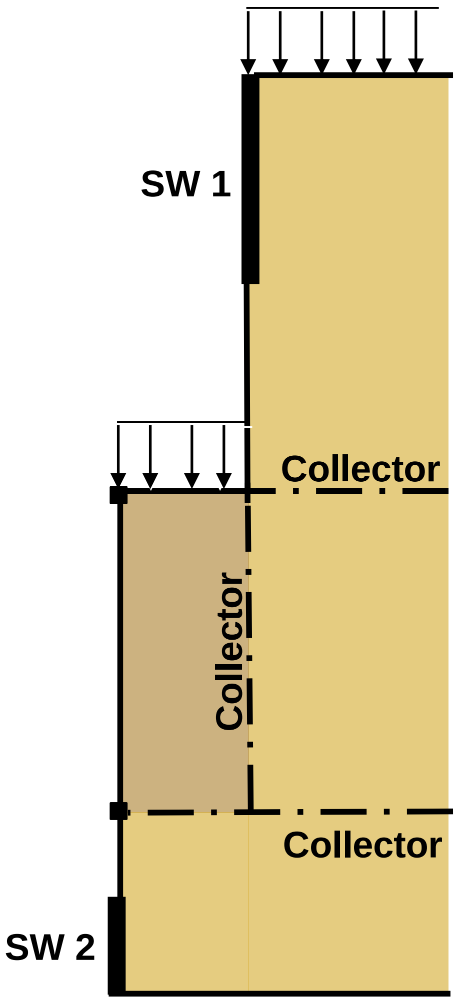
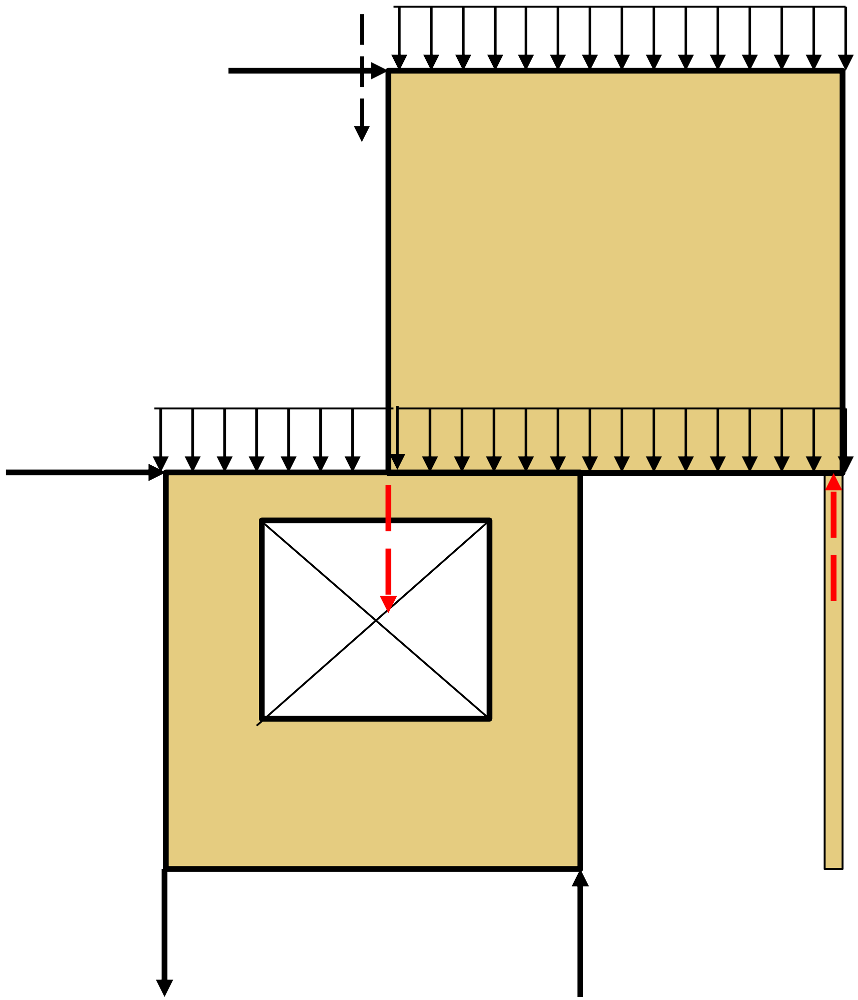
Offset Shear Walls – Code Requirements

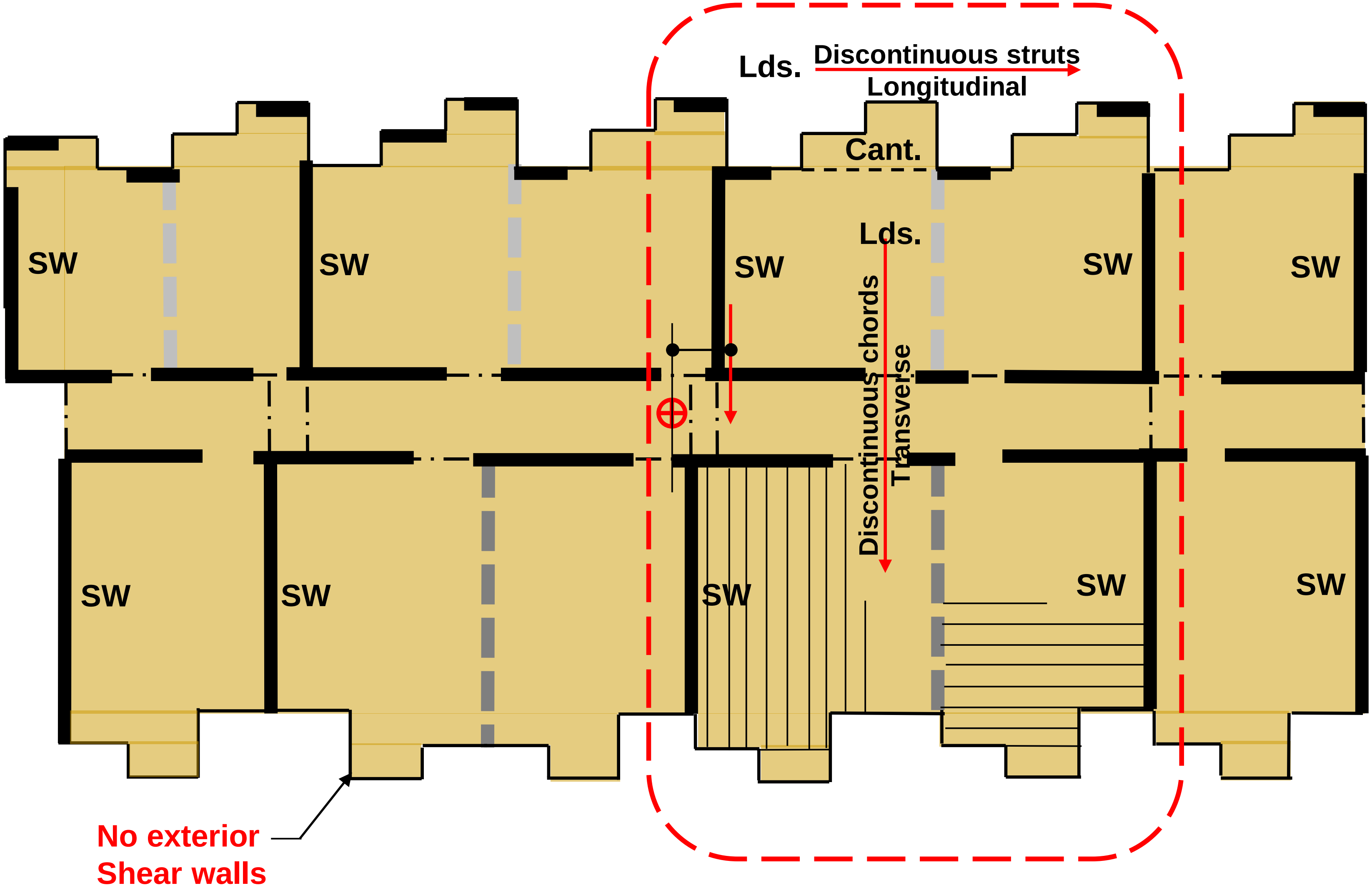


Out-of-plane Offsets

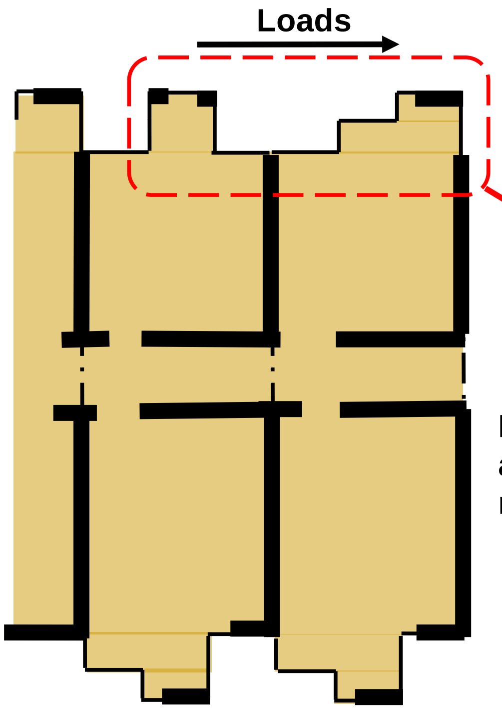


In-plane Offsets

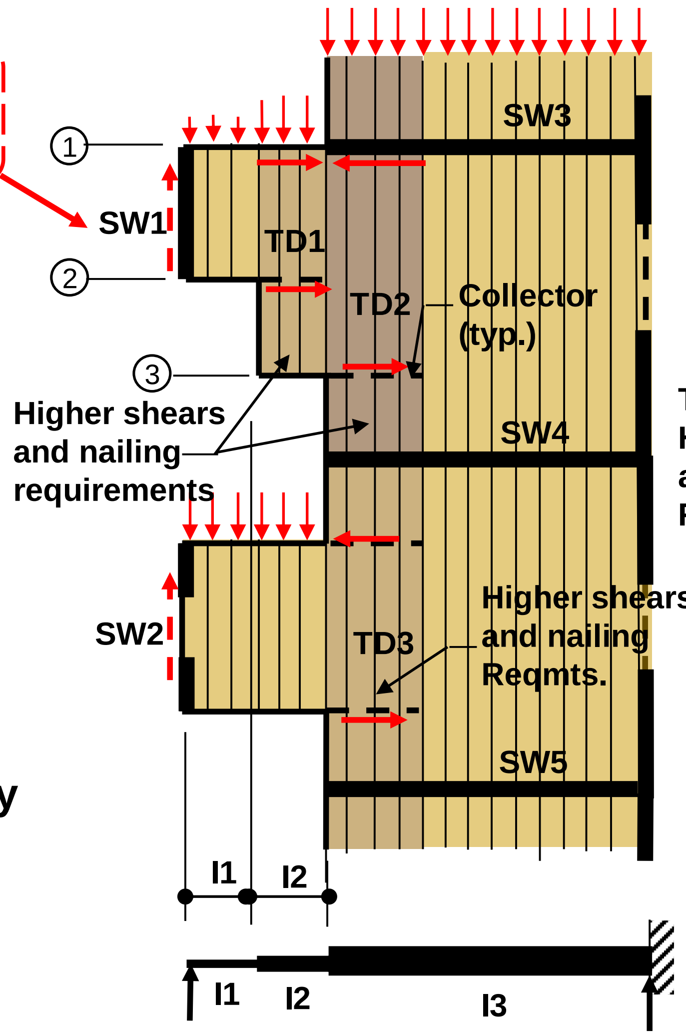
Mid-rise Multi-family



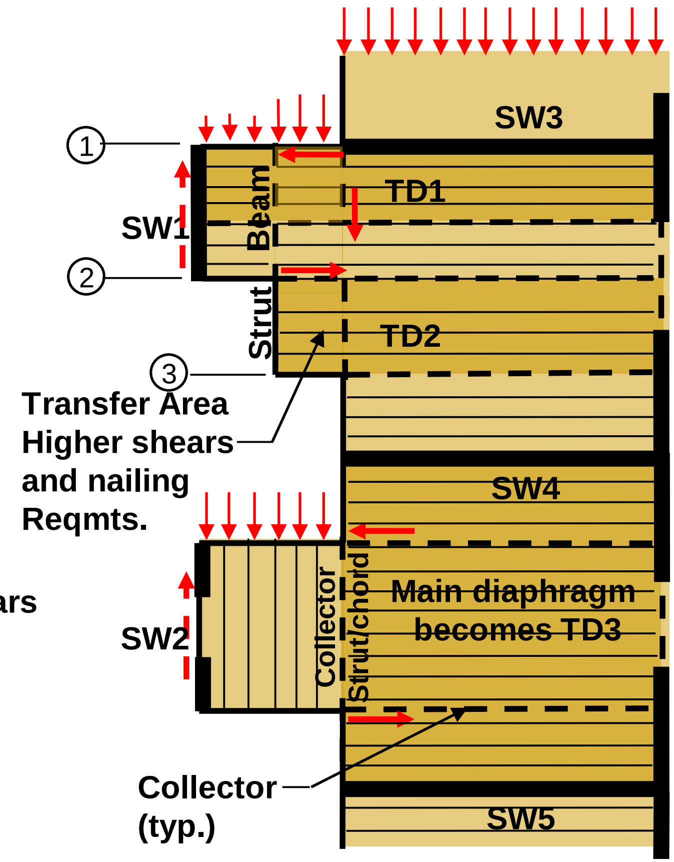
Flexible, semi-rigid, or rigid???



Multi Story, Multi-family Wood Structure



Diagraphm stiffness changes



Optional Framing Layouts

<u>ASCE7-10</u>

1.3.5 - Cont. ld. Paths

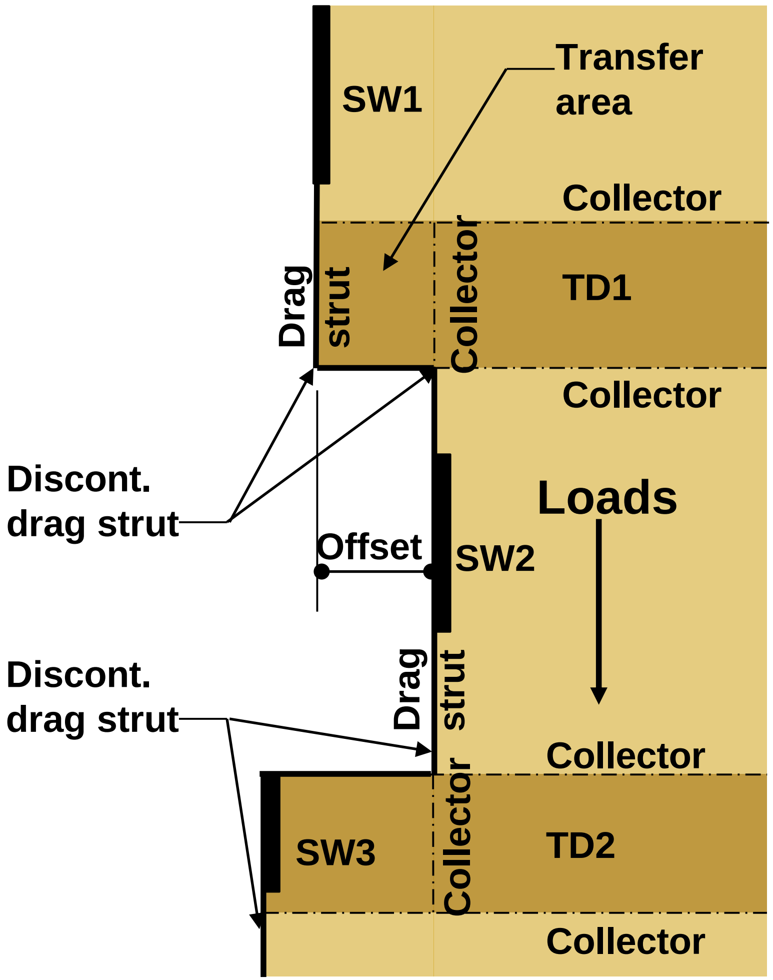
12.1.3 - Cont. ld. paths-inter-conn. Ties

12.10.1-Openings, re-entrant. –transfer of dis-cont. forces combined with other forces

12.10.2-Collector elements

Out-of-Plane Offset Shear Walls

Assumed to act in the Same Line of Resistance



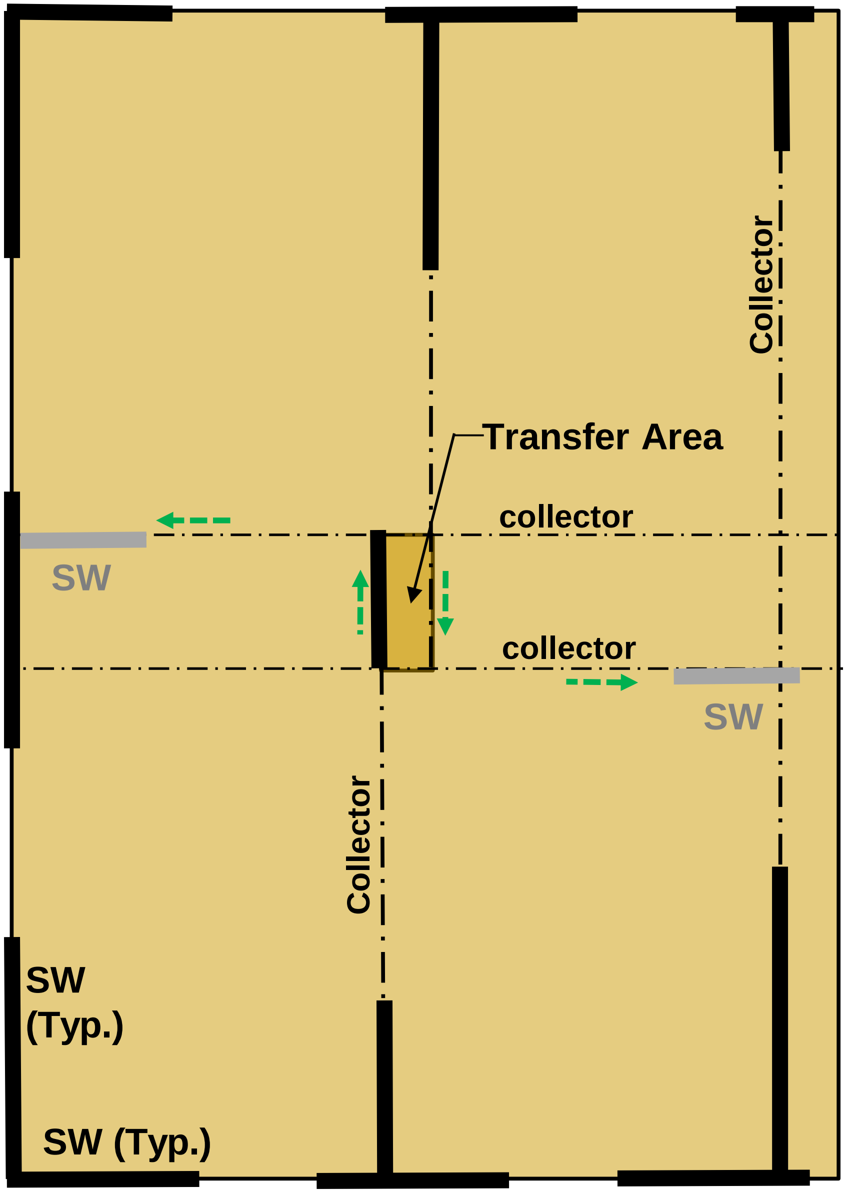
Typical mid-rise multi-family structure at exterior wall line

- **Offset walls are often assumed to act in the same line of lateral-force-resistance.**
- **Calculations are seldom provided showing how the walls are interconnected to act as a unit, or to verify that a complete lateral load path has been provided.**
- **Collectors are required to be installed to transfer the disrupted forces across the offsets.**

Where offset walls occur in the wall line, the shear walls on each side of the offset should be considered as separate shear walls and should be tied together by force transfer around the offset (<u>in the plane of the diaphragm</u>).

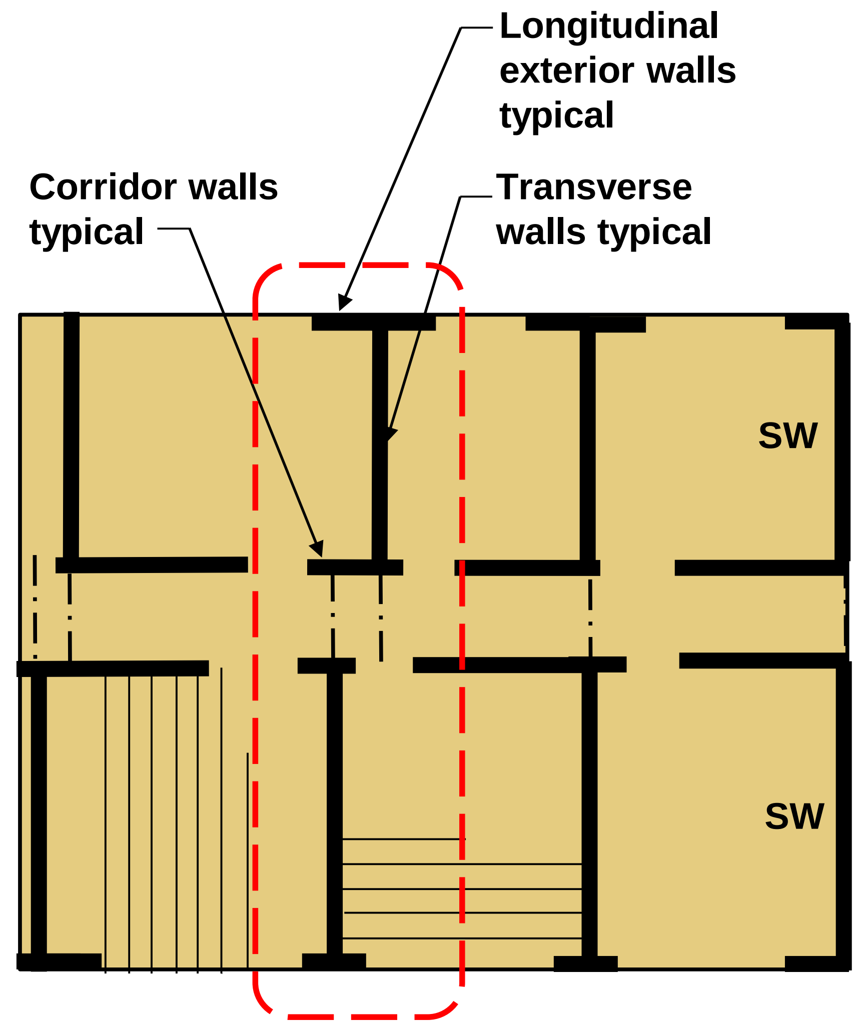
Check for Type 2 horizontal irregularity
Re-entrant corner irregularity

Tying Shear Walls Across the Corridor or a Large Diaphragm



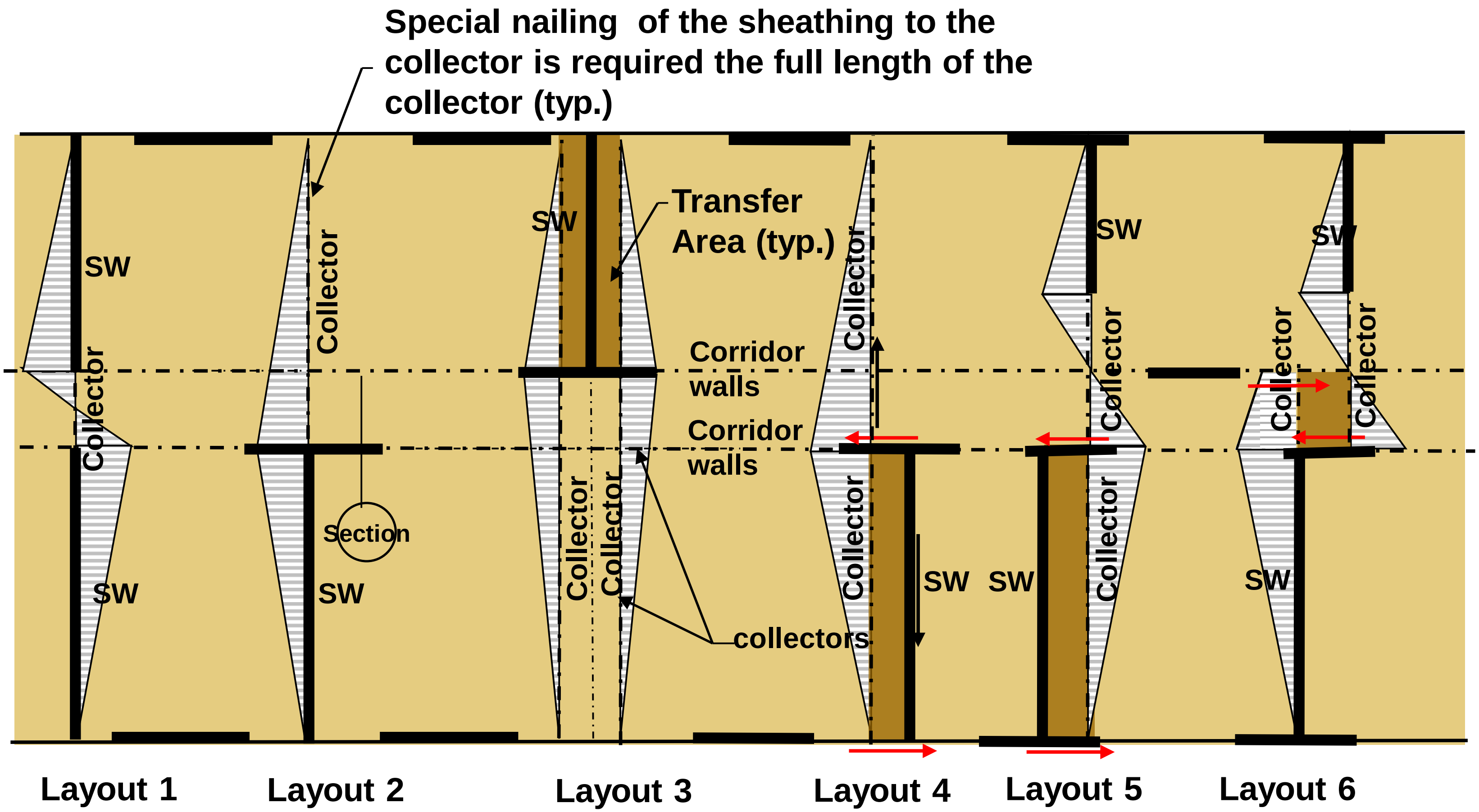
Tying Shear Walls Across a Large Diaphragm

Typical Rectangular plans



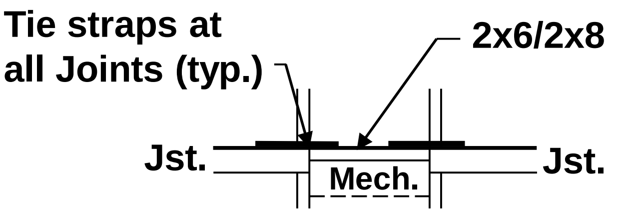
Tying Shear Walls Across the Corridor

Tying Shear Walls Across the Corridor



SDPWS 4.3.5.1

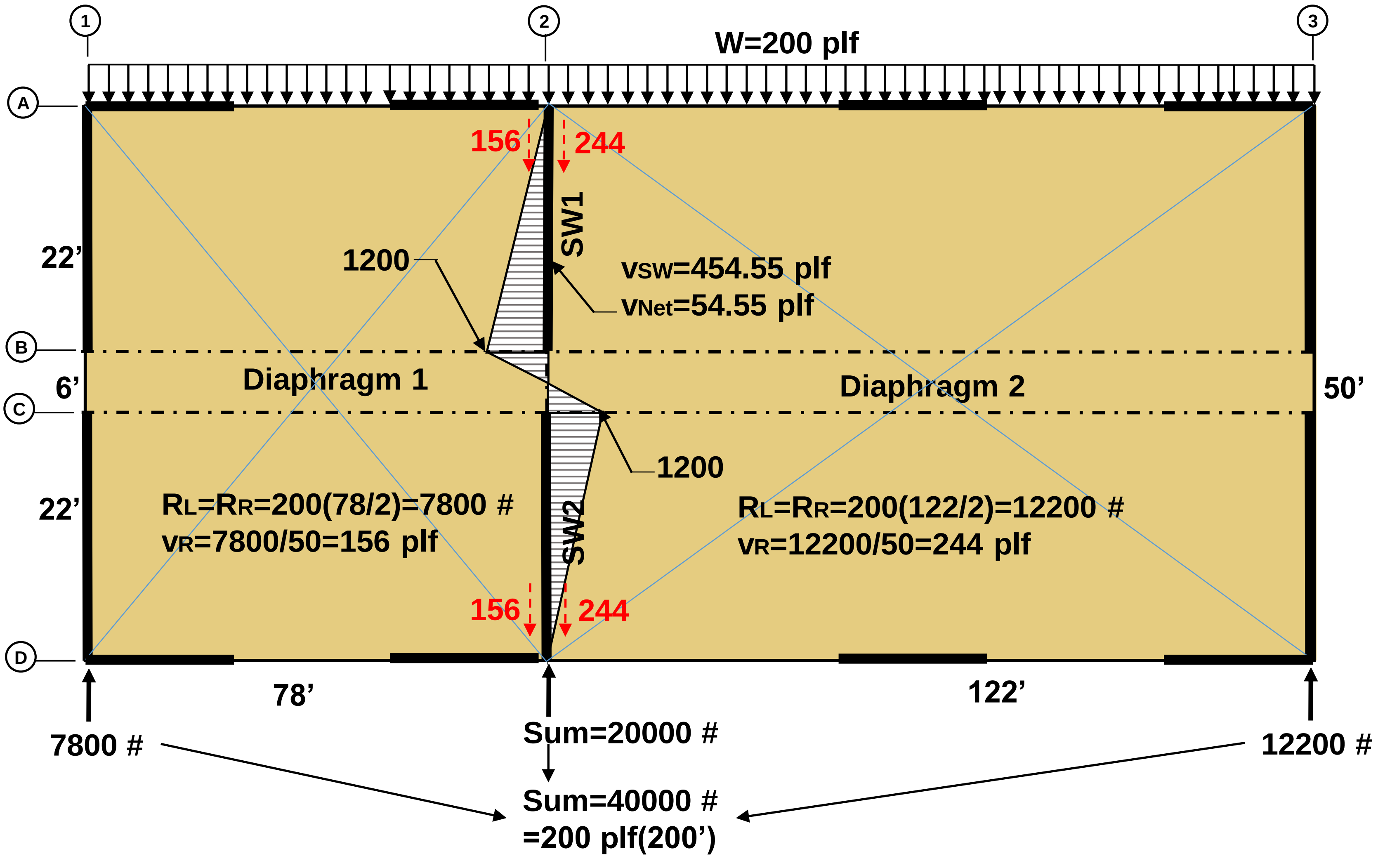
3. Collectors for shear transfer to individual full-height wall segments shall be provided.



Typical Corridor Section

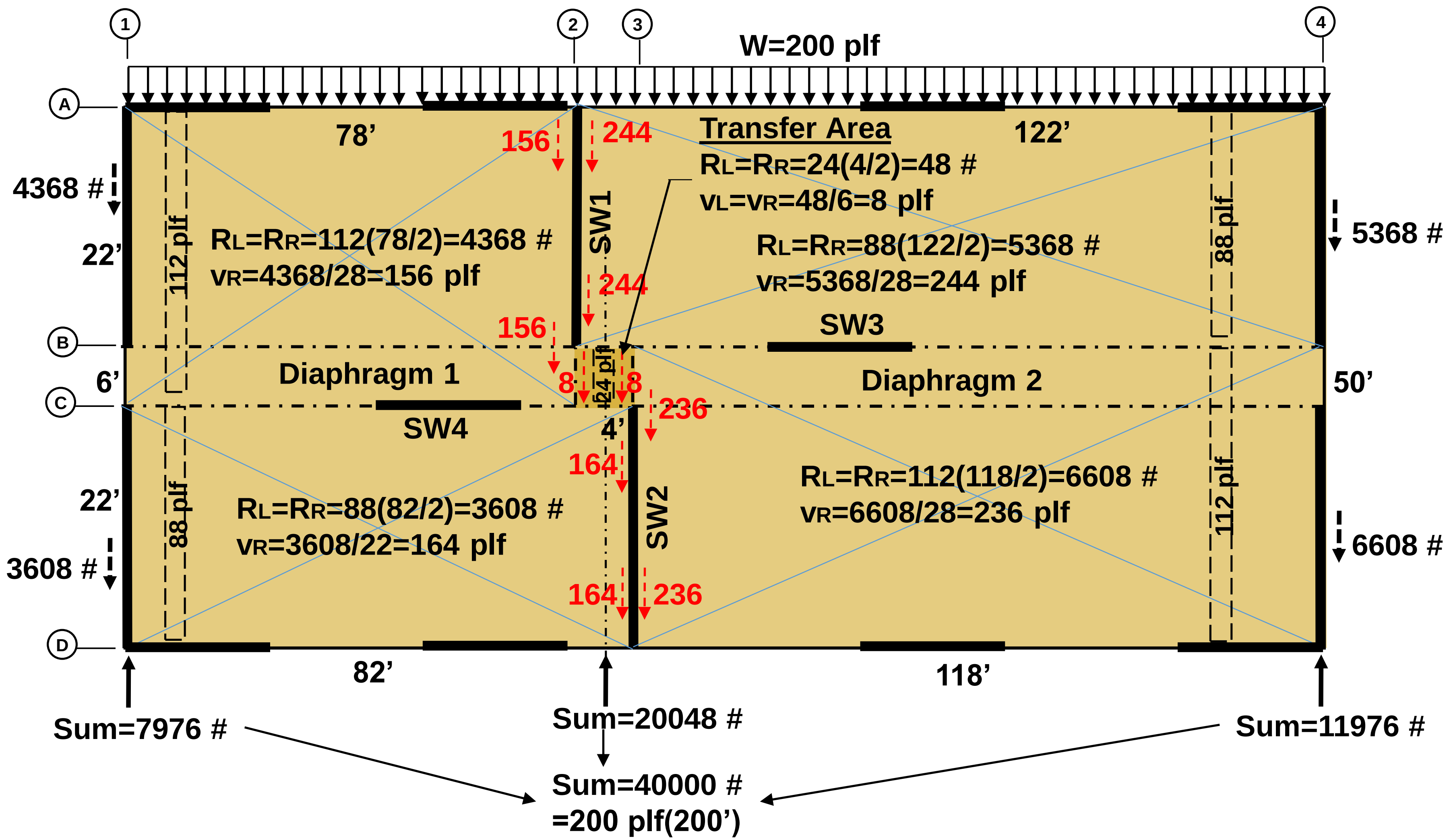
Common Transverse Wall Layouts

Layout 1-Full length walls aligned

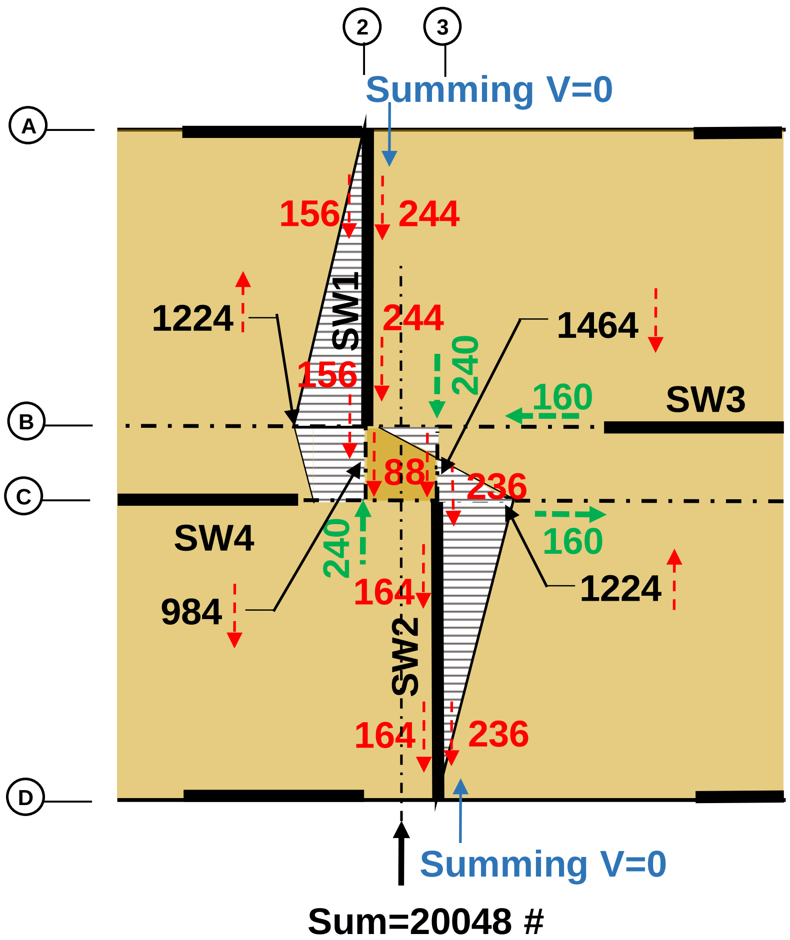


Example 1-Offset Shear Walls -Layout 6

Case 1-Full length offset walls



Case 1-Smaller resulting forces at corridor



All forces in lb., all shears in plf

Total load to grid lines 2 & 3

R_{23}=4368+3608+48+5368+6608+48=20048 # O.K.

L_{SW}=22+22=44'
V_{SW}=20048 #
v_{SW}=20048/44=455.64 plf

v_{net} SW1=455.64-156-244=55.64 plf
v_{net} SW2=455.64-164-236=55.64 plf Checks, they should be equal

SW1
F_{2AB}=55.64(22)=1224 #
F_{2BC}=(156+8)6=984 #
F_{2C}=1224-984=240 #

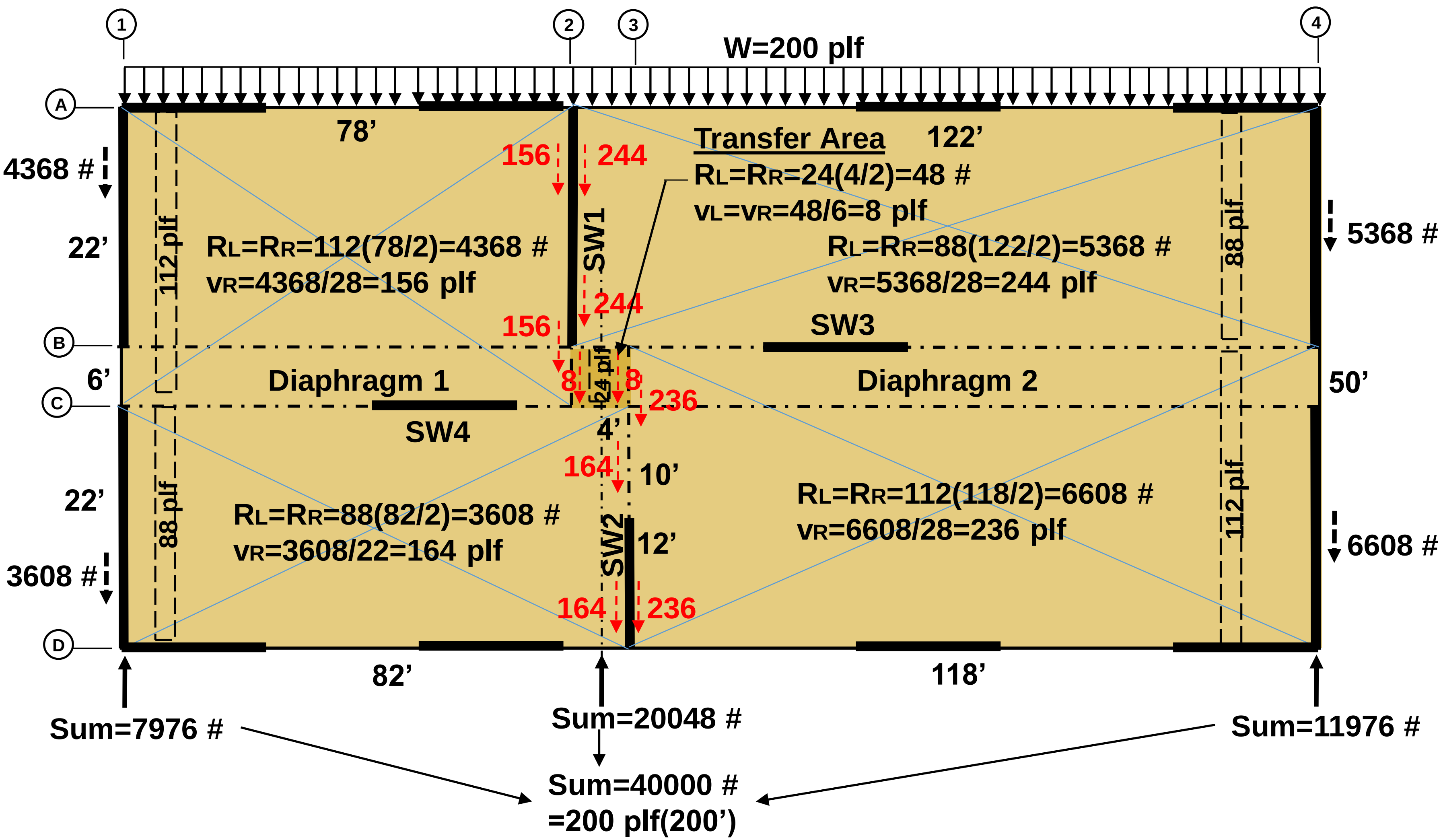
O.K.

SW2
F_{3CD}=55.64(22)=1224 #
F_{3CB}=(236+8)6=1464 #
F_{3B}=1224-1464= -240 #

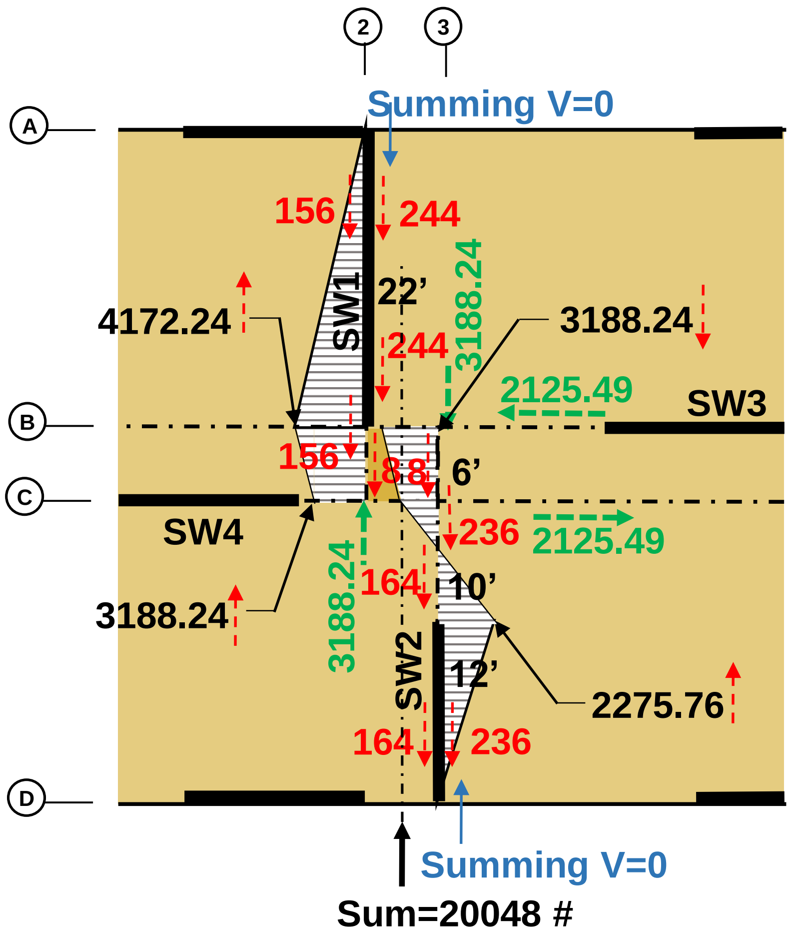
F_{B23}=F_{C23}=240(4)/6=160 #

Shear at transfer area=240/6+8=48 plf

Case 2-Full length plus partial length offset shear walls



Case 2-Larger resulting forces at corridor



All forces in lb., all shears in plf

Total load to grid lines 2 & 3

R_{23}=4368+3608+48+5368+6608+48=20048 # O.K.

L_{SW}=22+12=34'
V_{SW}=20048 #
v_{SW}=20048/34=589.65 plf

v_{net} SW1=589.65-156-244=189.65 plf
v_{net} SW2=589.65-164-236=189.65 plf Checks, they should be equal

SW1
F_{2AB}=189.65(22)=4172.24 #
F_{2BC}=(156+8)6=984 #
F_{2C}=4172.24-984=3188.24 #

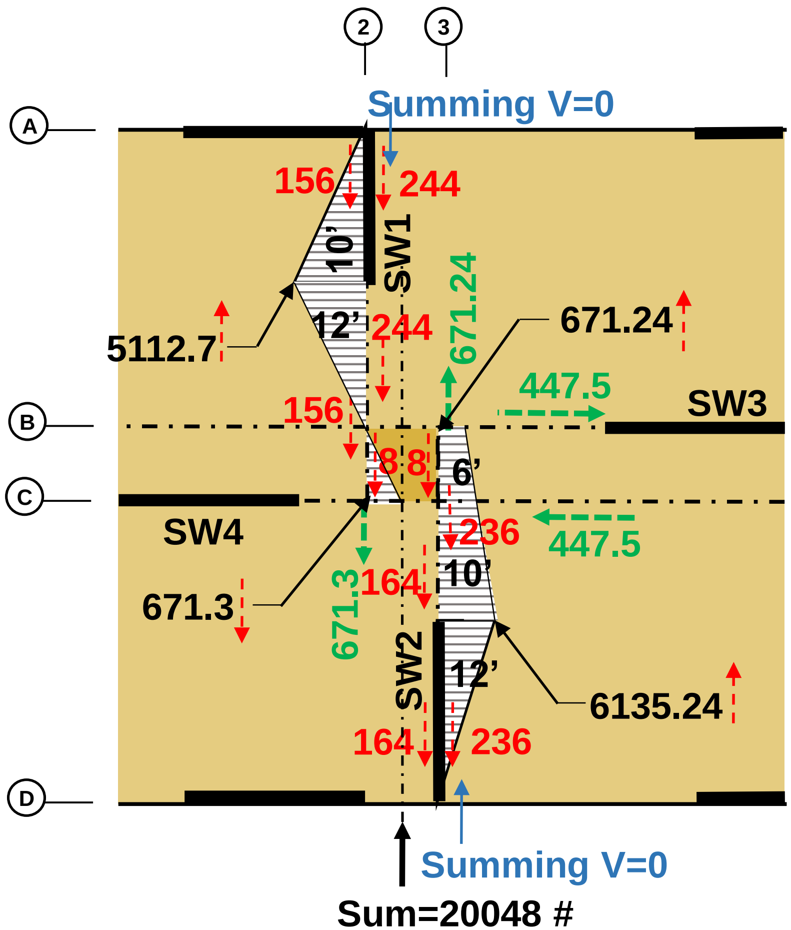
O.K.

SW2
F_{SW2} =189.65(12)=2275.76 #
F_{SW2} to $_{3B}$=(236+8)6+(236+164)10=5464 #
F_{3B}=2275.76-5464= -3188.24 #

F_{B23}=F_{C23}=3188.24(4)/6=2125.49 #

Shear at transfer area=3188.24/6+8=539.37 plf

Case 3-Smaller resulting forces at corridor



All forces in lb., all shears in plf

Total load to grid lines 2 & 3

R23=4368+3608+48+5368+6608+48=20048 # O.K.

LSW=12+10=22'
VSW=20048 #
vSW=20048/22=911.27 plf

vnet SW1=911.27-156-244=511.27 plf
vnet SW2=911.27-164-236=511.27 plf Checks, they should be equal

SW1
FSW1=511.27(10)=5112.7 #
FSW1 to 2B=(156+244)12=4800 #
F2BC=(156+8)6=984 #
F2C=5112.7-4800-984=-671.3 #

O.K.

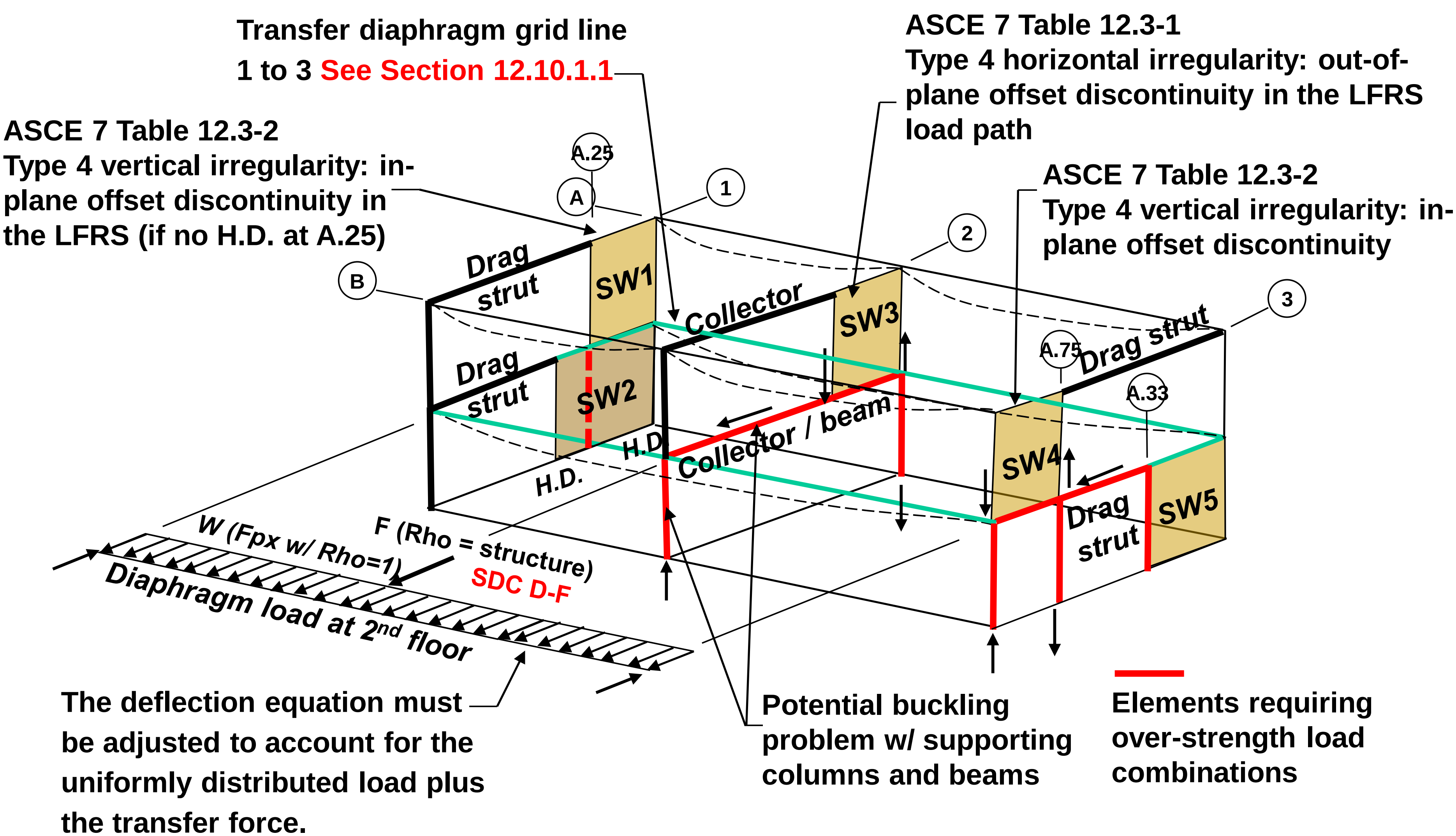
SW2
FSW2=511.27(12)=6135.24 #
FSW2 to 3C=(236+8)6+(236+164)10=5464 #
F3B=6135.24-5464= 671.24 #

FB23=FC23=671.3(4)/6=120 #

Shear at transfer area=671.3/6+8=119.9 plf

Relevant Irregularities Per ASCE 7-10

Horizontal Irregularities Table 12.3-1 and Vertical Irregularities Table 12.3-2

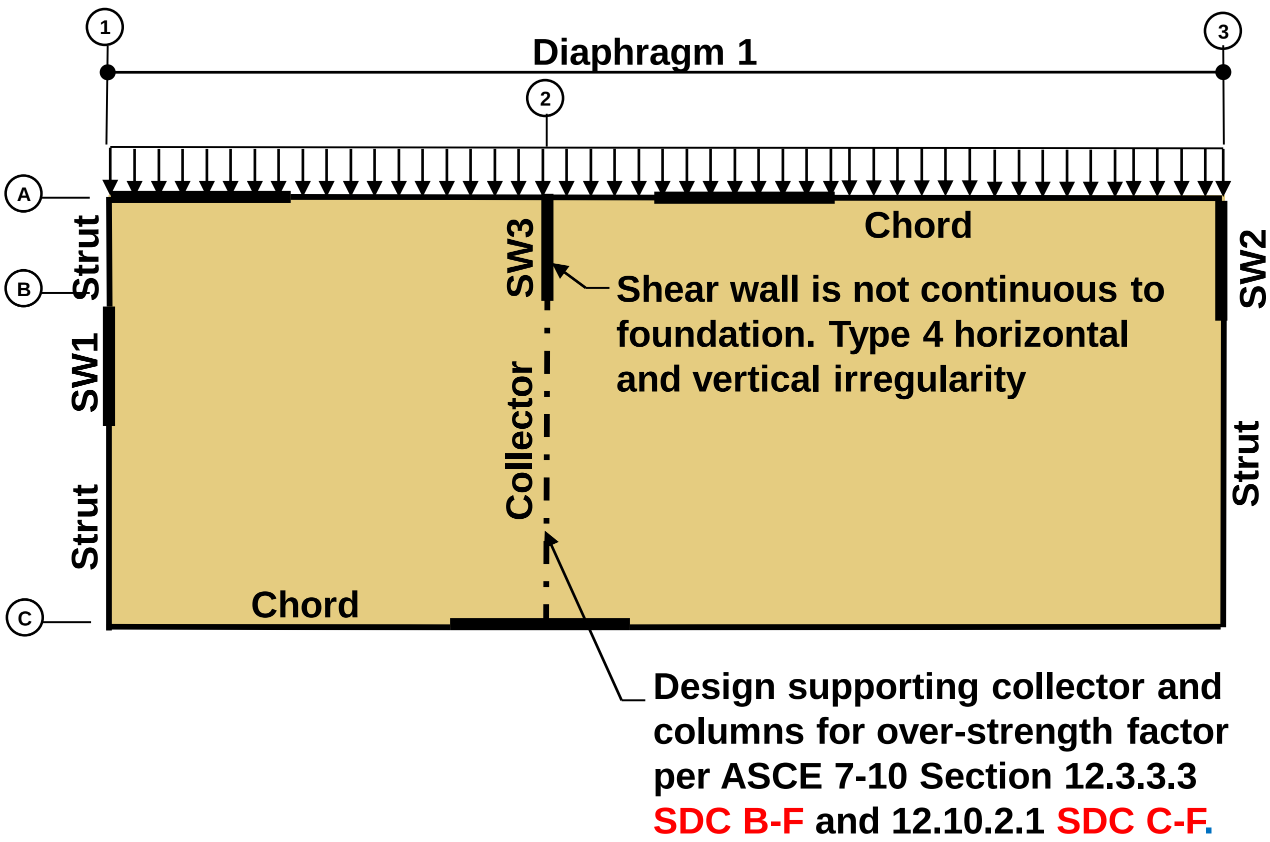


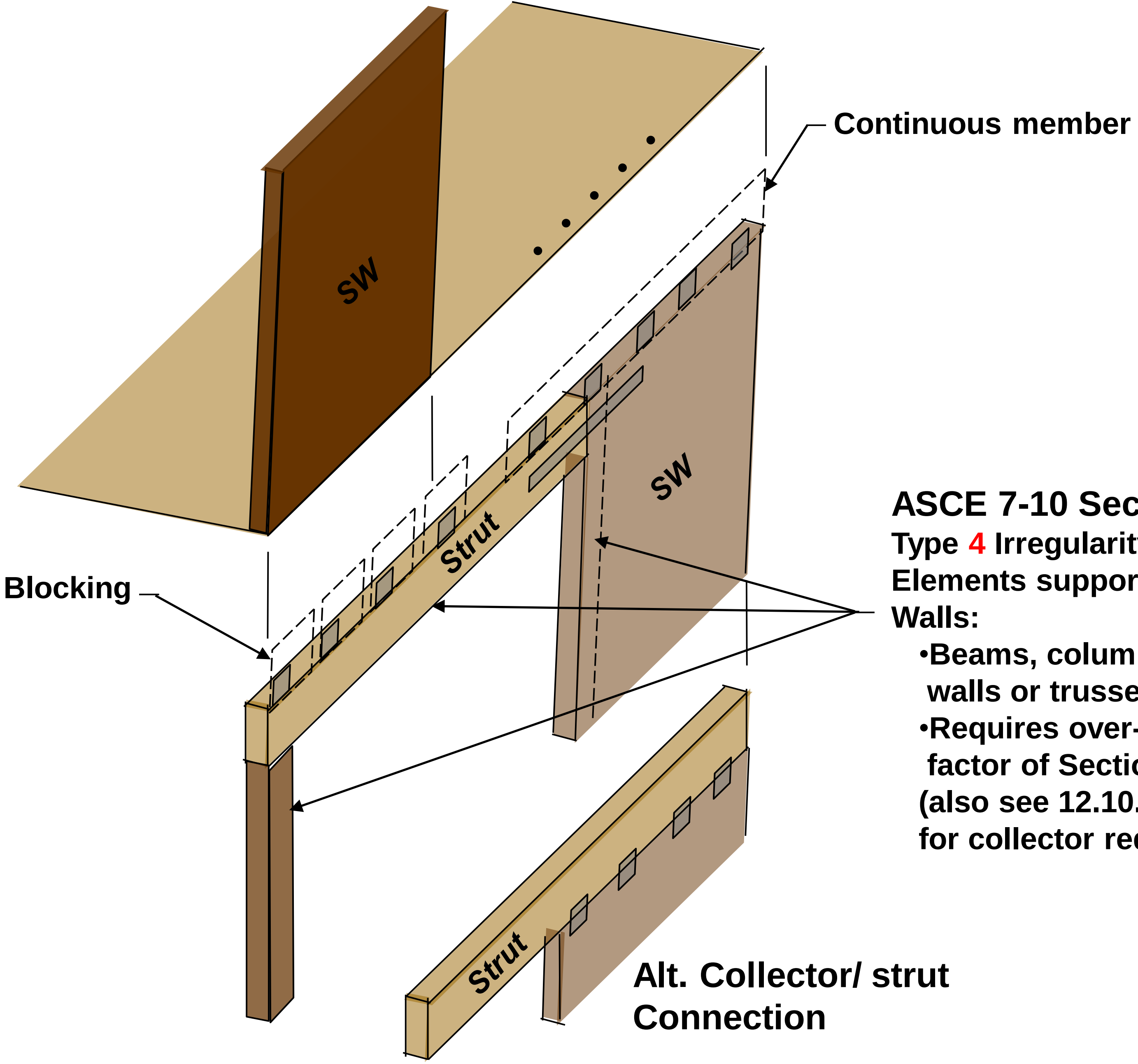
Type 4 Horizontal Offset Irregularity-Seismic

Type 4 horizontal irregularity-Out-of-plane offset irregularity occurs where there is a discontinuity in lateral force resistance path. Out-of-plane offset of at least one of the vertical lateral force resisting elements.

ASCE 7-10 Section 12.3.3.3 (SDC B-F)-Elements supporting discontinuous walls or frames. Type 4 horizontal or vertical irregularity.

- **Beams, columns, slabs, <u>walls</u> or trusses.**
- **Requires over-strength factor of Section 12.4.3**





ASCE 7-10 Section 12.3.3.3

Type 4 Irregularity (SDC B-F)

Elements supporting discont. Walls:

- **Beams, columns, slabs, walls or trusses.**
- **Requires over-strength factor of Section 12.4.3**

(also see 12.10.2.1 SDC C-F for collector requirements).

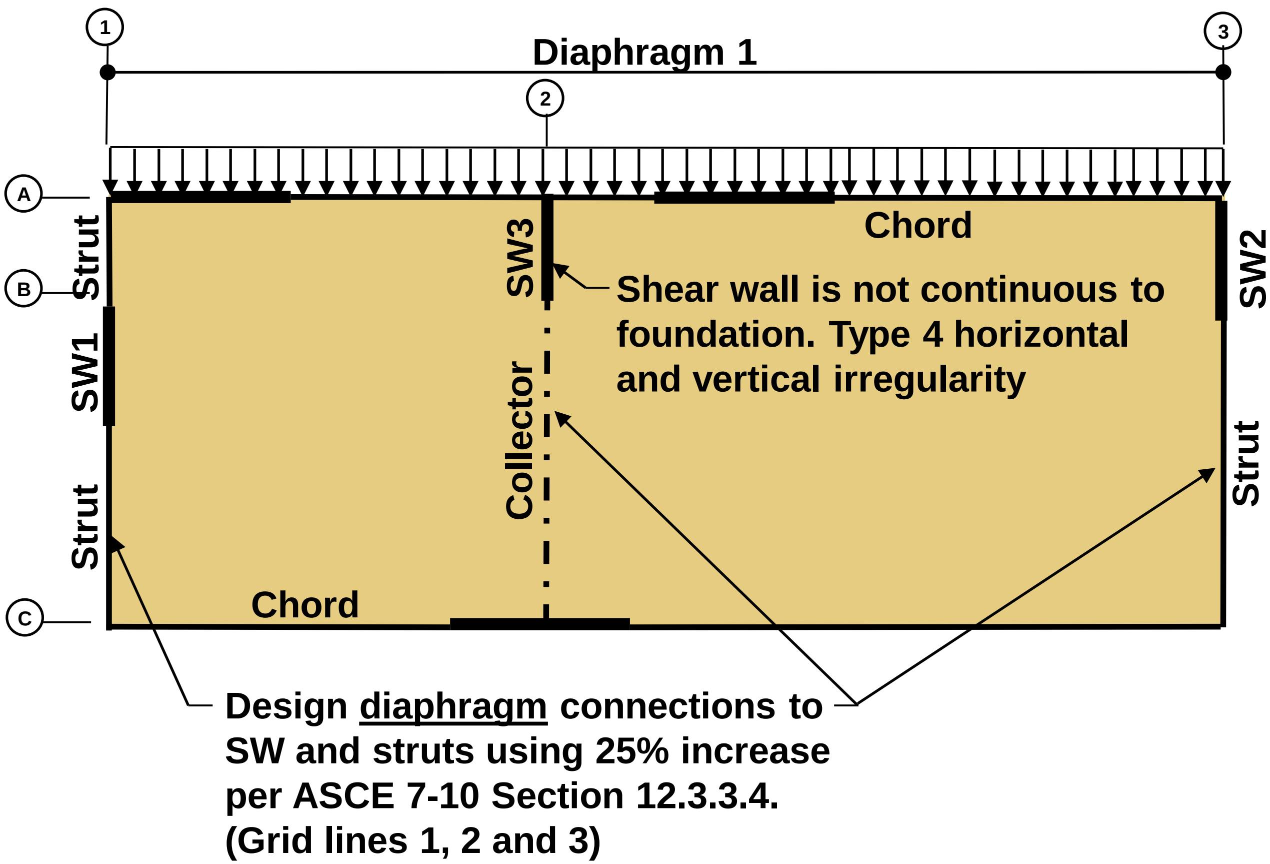
Type 2 and Type 4 Irregularity SDC D-F

(Interior collector similar)

Type 4 Horizontal Offset Irregularity-Seismic

ASCE 7-10 Section 12.3.3.4 (SDC D-F) -Type 4 Horizontal irregularity or Type 4 vertical irregularity requires a 25% increase in the diaphragm (inertial) design forces determined from 12.10.1.1 (F_{px}) for the following elements:

- **Connections of diaphragm to vertical elements and collectors.**



- **Diaphragm shears are not required to be increased 25%.**
- **The transfer force (SW) in SDC D-F must be increased by rho, per 12.10.1.1.**

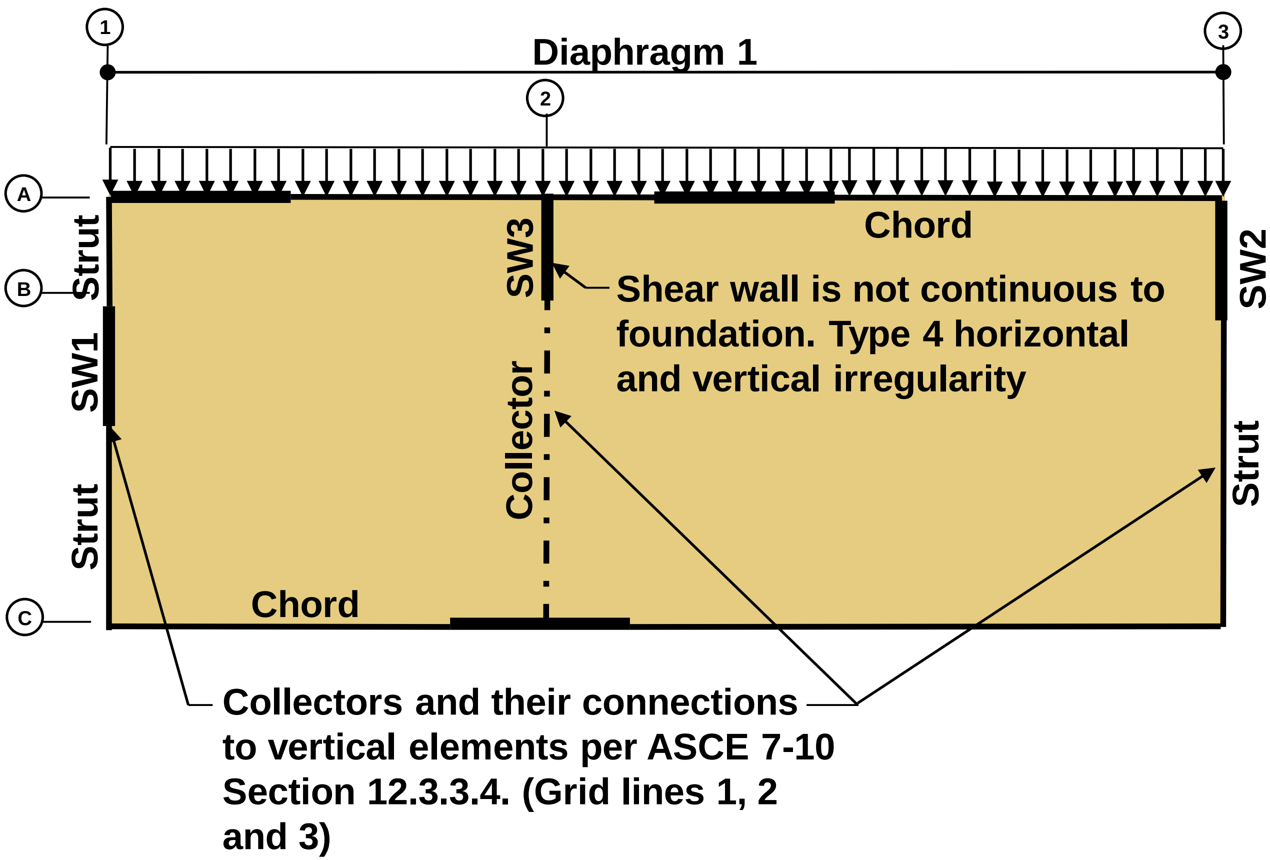
See 12.10.2 & 12.10.2.1 for collectors

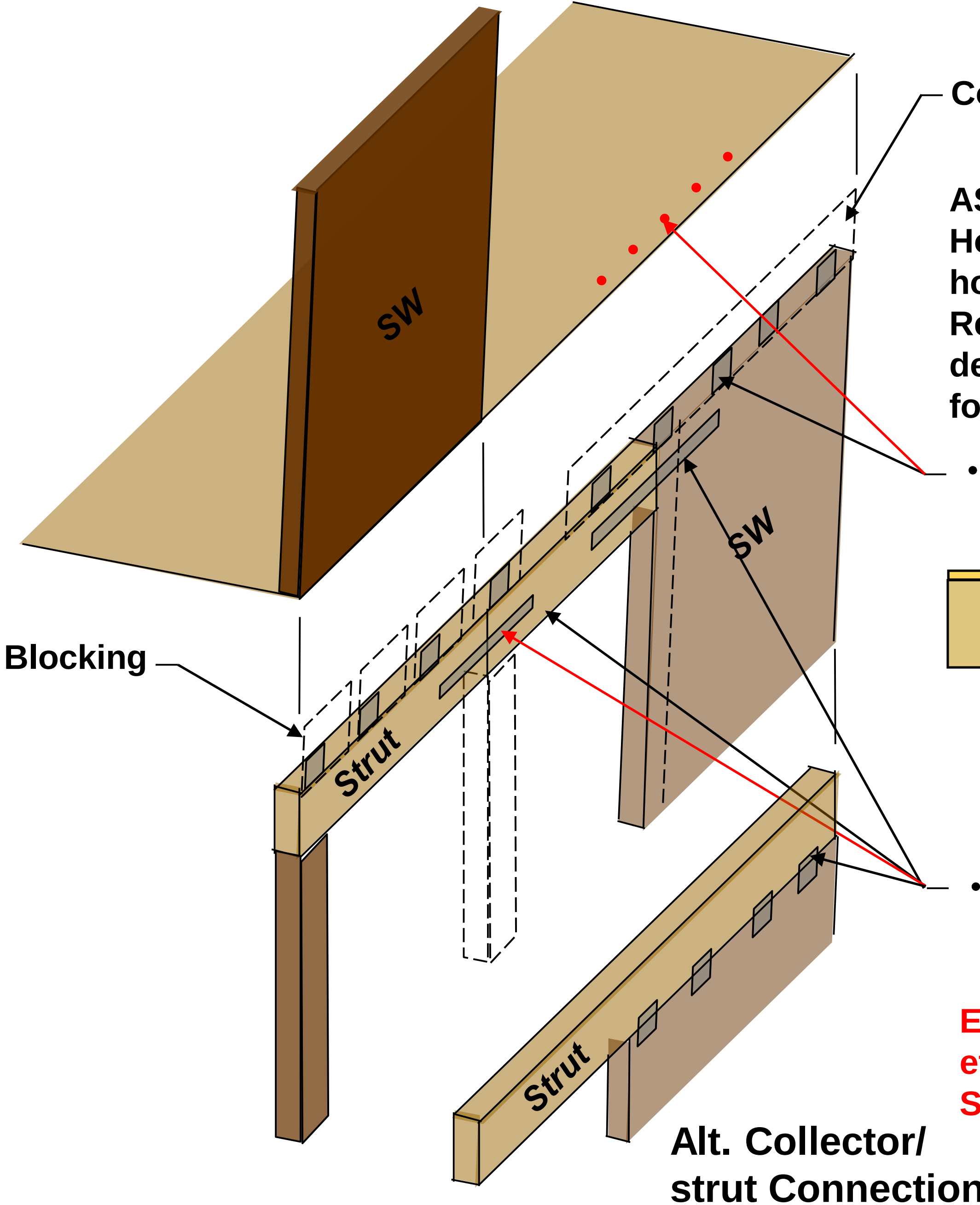
Type 4 Horizontal Offset Irregularity-Seismic

ASCE 7-10 Section 12.3.3.4 (SDC D-F) -Type 4 Horizontal irregularity or Type 4 vertical irregularity requires a 25% increase in the diaphragm (inertial) design forces determined from 12.10.1.1 (Fpx) for the following elements:

- **Collectors and their connections to vertical elements.**

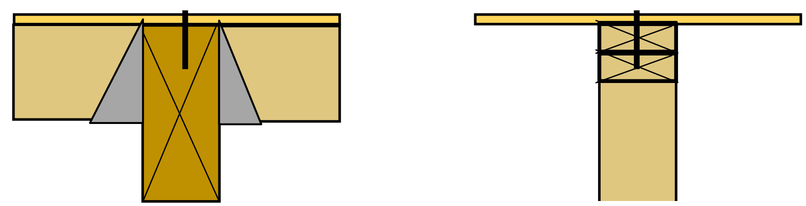
Exception: Forces using the seismic load effects including the over-strength factor of Section 12.4.3 need not be increased.





ASCE 7-10 Section 12.3.3.4 (SDC D-F)
Horizontal Type 2 Irreg.-Re-entrant corner, horizontal and vertical Type 4 irregularity: Requires a 25% increase in the diaphragm design forces determined from 12.10.1.1 (Fpx) for the following elements:

- Connections of diaphragm to vertical elements and collectors.

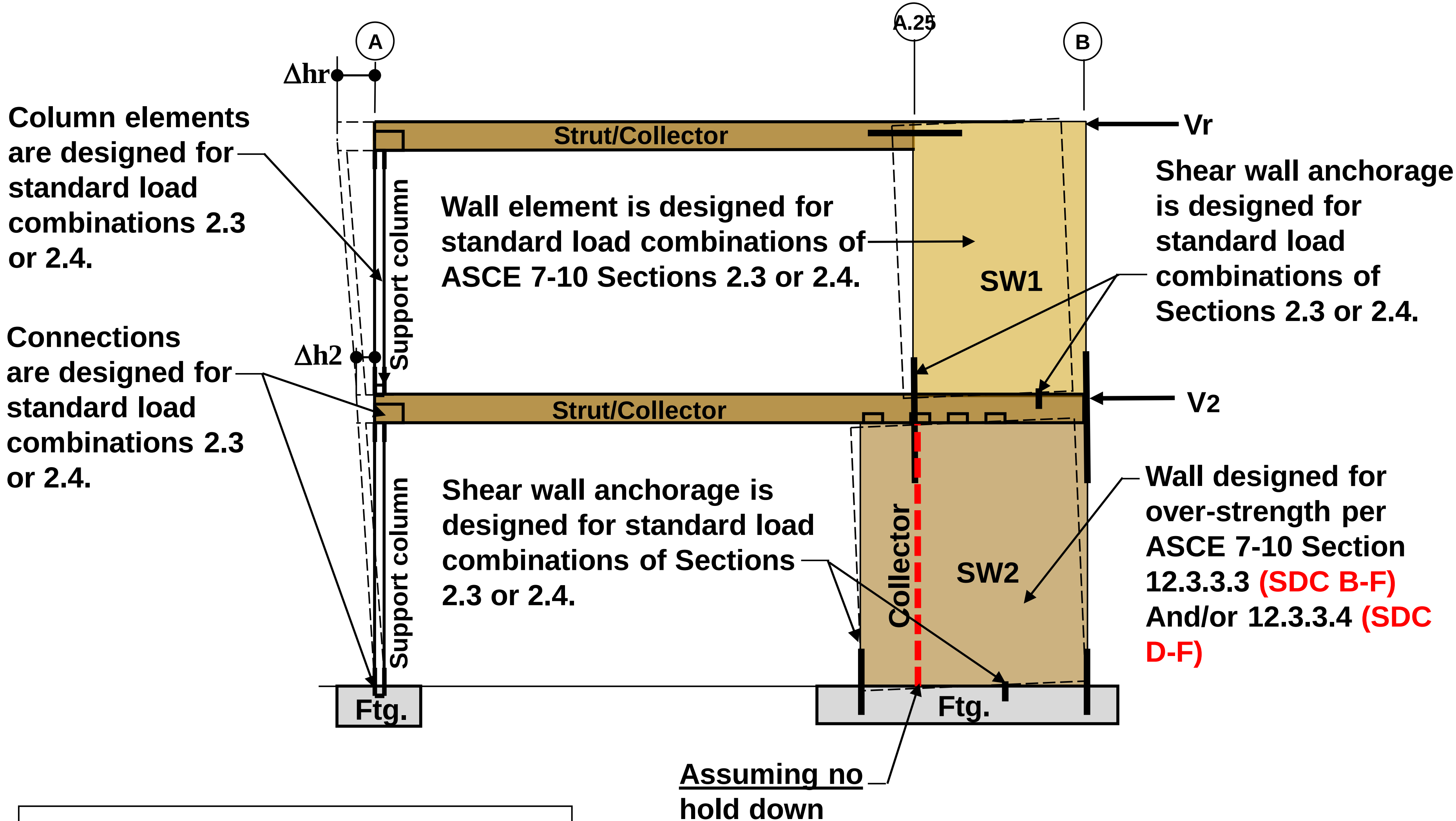


- Collectors and their connections, including their connections to vertical elements.

Exception: Forces using the seismic load effects including the over-strength factor of Section 12.4.3 need not be increased.

Type 2 and Type 4 Irregularity SDC D-F

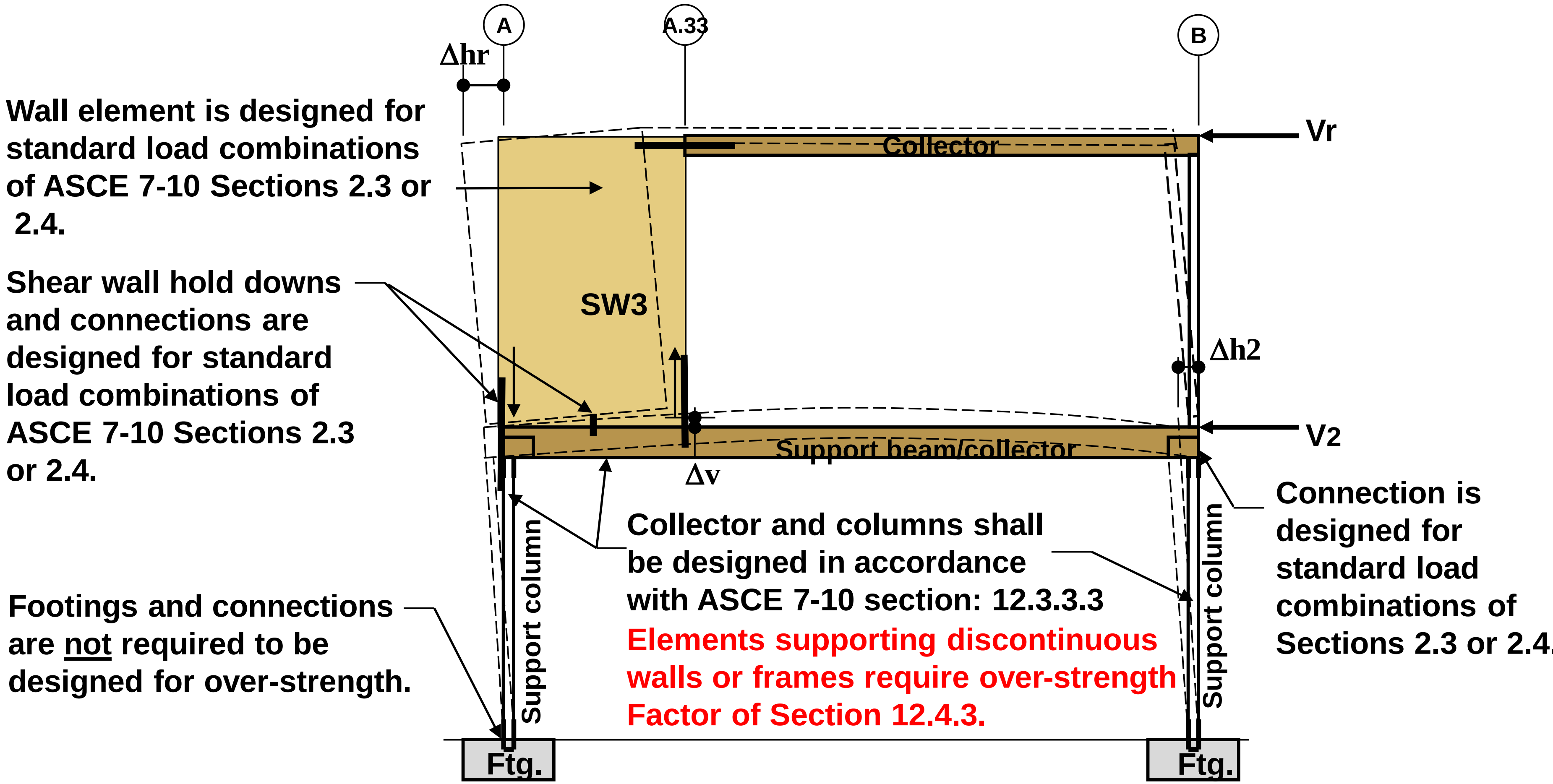
(Interior collector similar)



ASCE 7 Table 12.3-2-Type 4 vertical irregularity- In-plane offset discontinuity in the LFRS

See struts and collectors

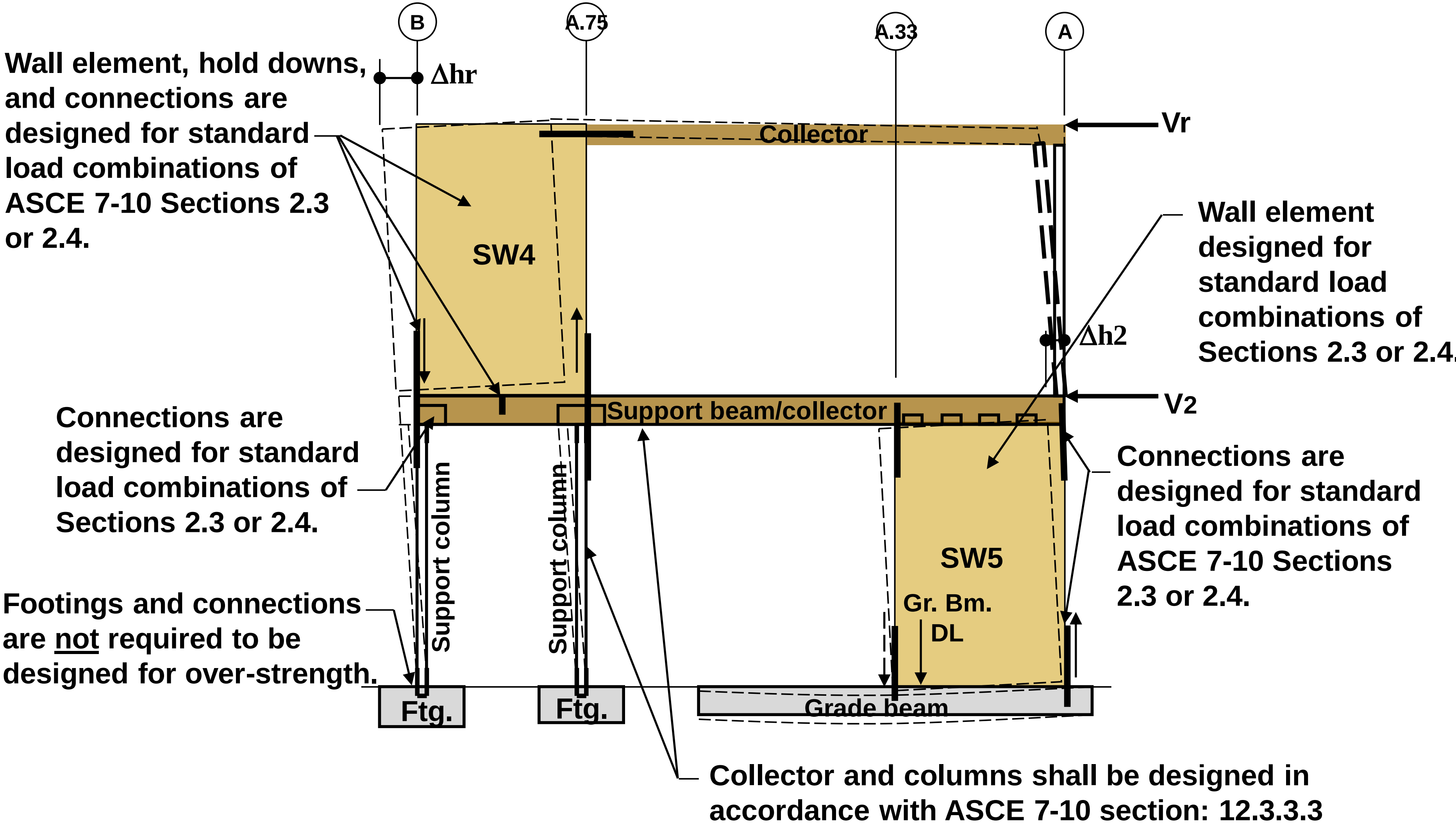
Shear wall system at grid line 1
In-plane Offset of Wall



ASCE 7 Table 12.3-1-Type 4 horizontal irregularity- out of-plane offset discontinuity in the LFRS

See struts and collectors

Shear wall system at grid line 2
Out-of-Plane Offset



Elements supporting discontinuous walls or frames require over-strength factor of Section 12.4.3.

ASCE 7 Table 12.3-2-Type 4 vertical irregularity- In-plane offset discontinuity in the LFRS

See struts and collectors

Shear wall system at grid line 3-In-plane Offset

Struts and Collectors

Struts and Collectors-Seismic

Struts / collectors and their connections shall be designed in accordance with ASCE 7-10 sections:

12.10.2 SDC B - Collectors can be designed <u>w/o</u> over-strength but not if they support discontinuous walls or frames.

12.10.2.1 SDC C thru F- Collectors and their connections, <u>including connections to the vertical resisting elements</u> require the over-strength factor of Section 12.4.3, except as noted:

<u>Shall be the maximum of:</u>

$\Omega_o F_x$ - **Forces determined by ELF Section 12.8 or Modal Response Spectrum Analysis procedure 12.9**

$\Omega_o F_{px}$ - **Forces determined by Diaphragm Design Forces (Fpx), Eq. 12.10-1 or**

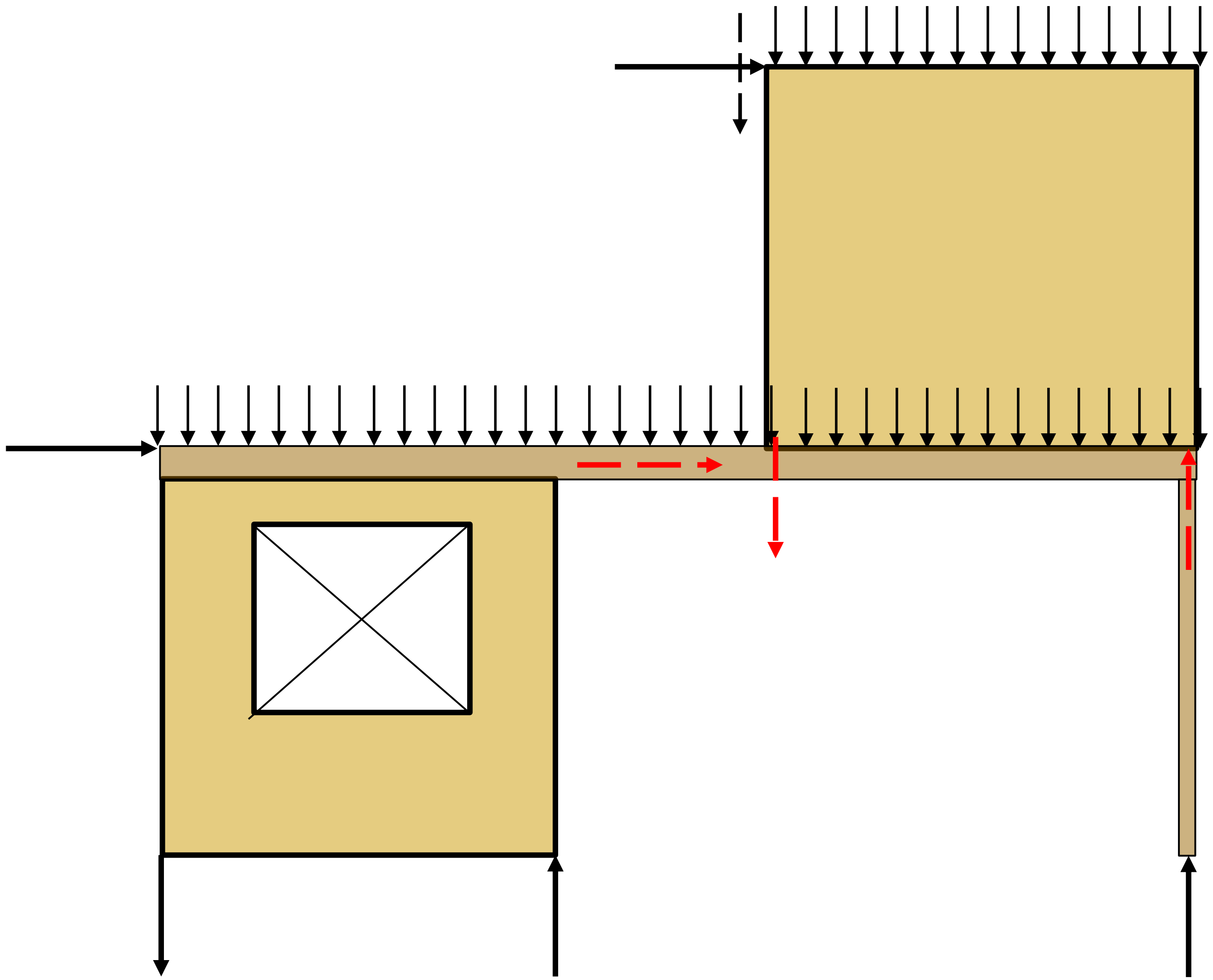
$F_{px\,min} = 0.2 S_{DS} I_e w_{px}$ - **Lower bound seismic diaphragm design forces determined by Eq. 12.10-2 (Fpx_{min}) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength)-do not require the over-strength factor.**

$F_{px\,max} = 0.4 S_{DS} I_e w_{px}$ - **Upper bound seismic diaphragm design forces determined by Eq. 12.10-2 (Fpx_{max}) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength)-do not require the over-strength factor.**

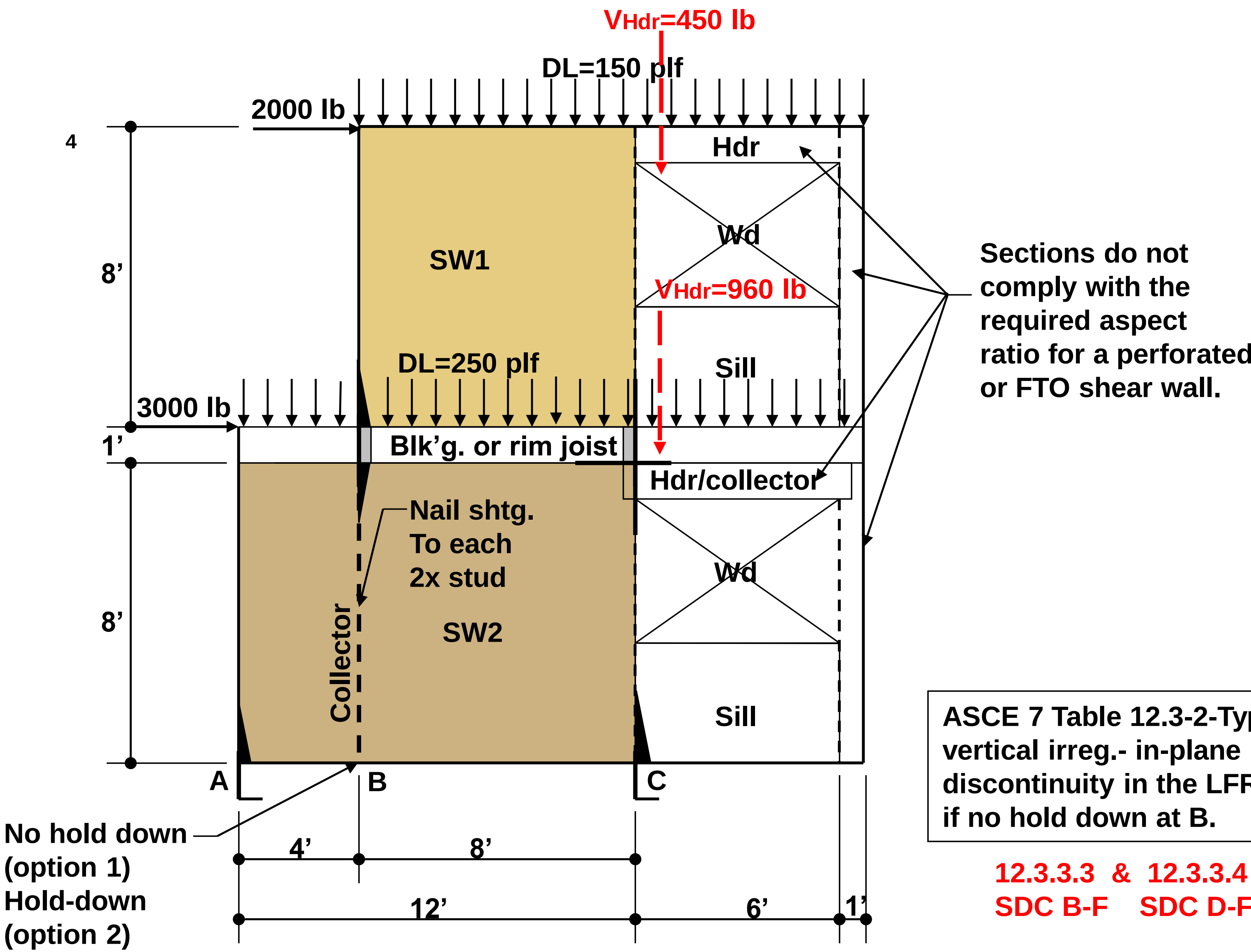
<u>Exception:</u>

1. **In structures (or portions of structures) <u>braced entirely by light framed shear walls</u>, collector elements and their connections, including connections to vertical elements need only be designed to resist forces using the standard seismic force load combinations of Section 12.4.2.3 with forces determined in accordance with Section 12.10.1.1 (Diaphragm inertial Design Forces, F_{px}).**

In-plane Offset Shear Walls



Example 5-In-plane Offset Segmented Shear Wall -with Gravity Loads



ASCE 7 Table 12.3-2-Type 4 vertical irreg.- in-plane discontinuity in the LFRS if no hold down at B.

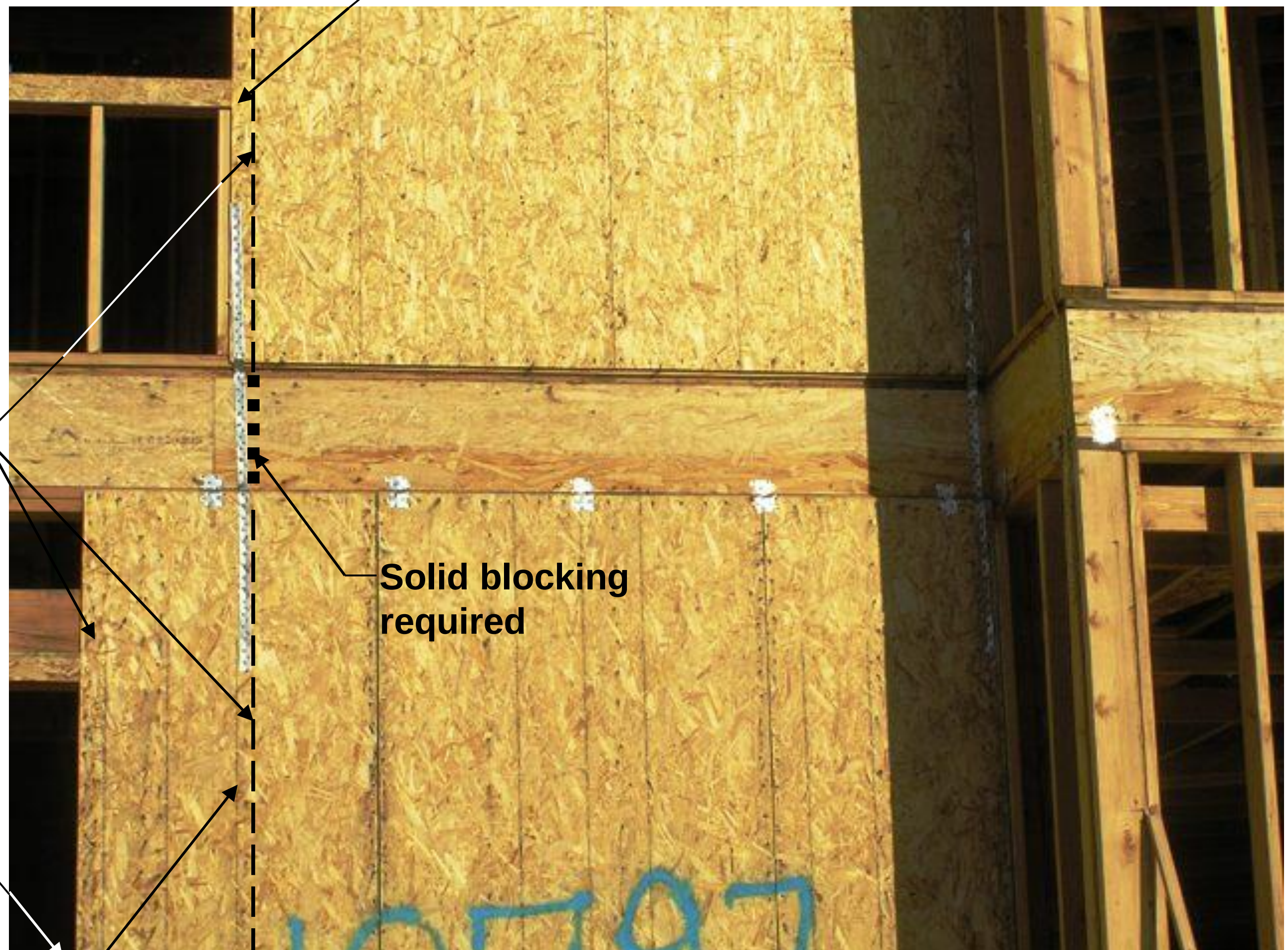
12.3.3.3 & 12.3.3.4
SDC B-F SDC D-F

Ends of wall panels do not line up. Requires special nailing of sheathing into stud below.

Requires same number of studs above and below with boundary nailing each stud

Solid blocking required

Hold down

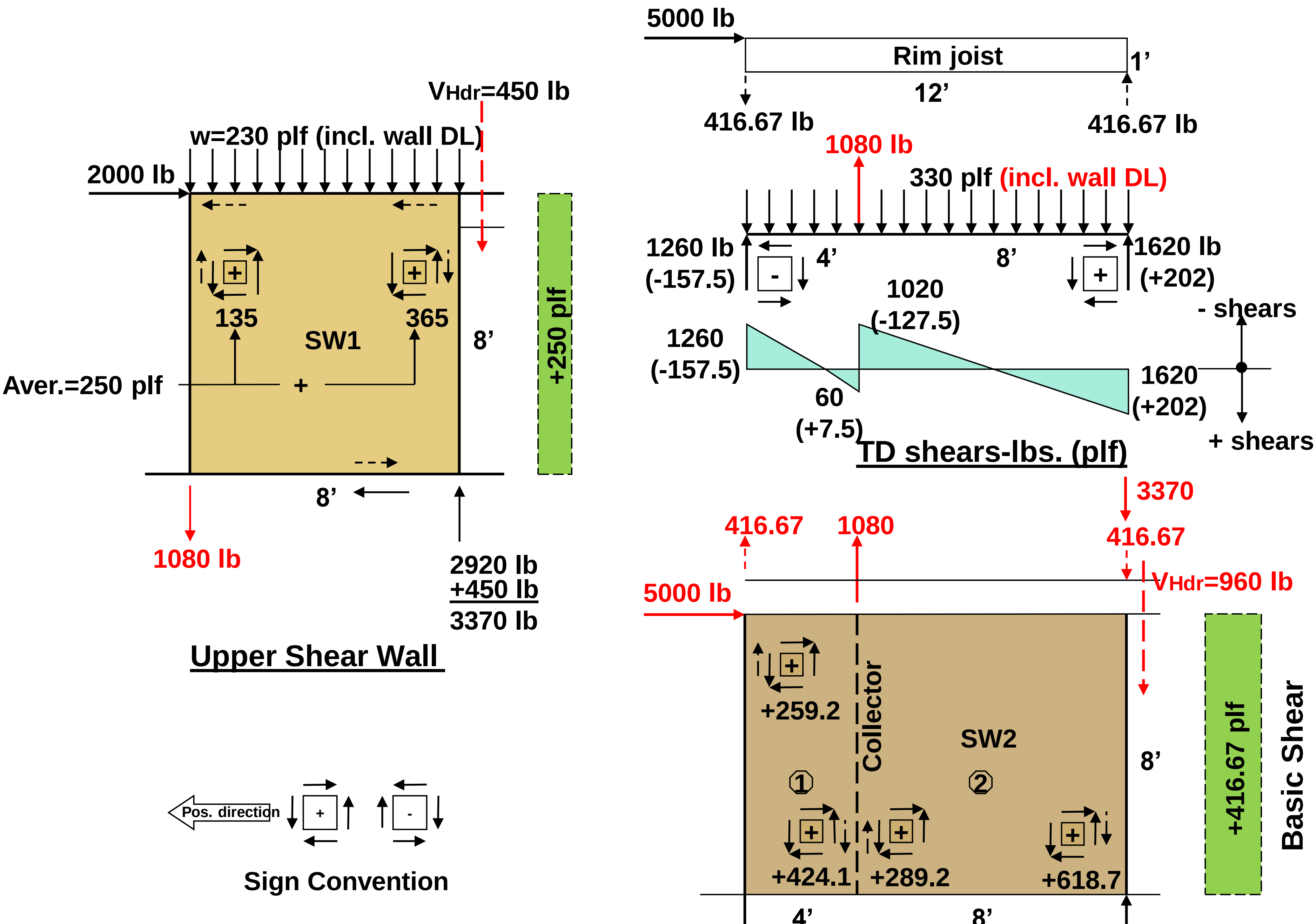


Nailing found in field was 12” o.c.

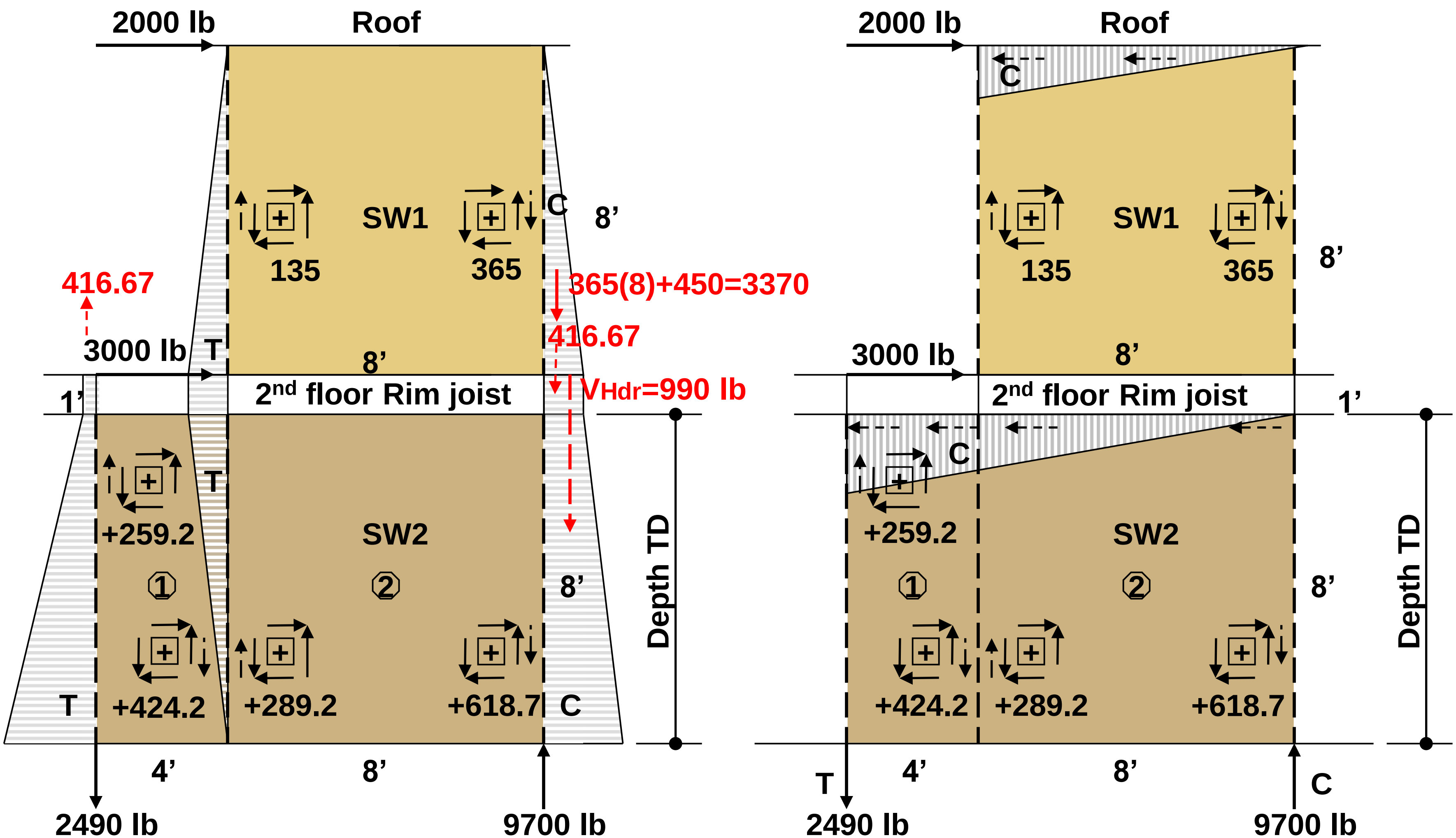
No hold-down below

Hold down

Photo-In-plane Offset Segmented Shear Walls

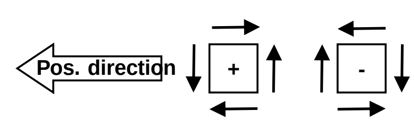


Wall and Transfer Diaphragm Shears



Vertical Collector Forces

Horizontal Collector Forces



Sign Convention

Collector Force Diagrams

Presentation Topics

- Shear Wall Types
- Shear wall Anchorage
- SDPWS Code Requirements
- Complete Load Paths
- Offset Shear Walls and Relevant Code Requirements
- **Mid-rise Diaphragms and Tall Shear Walls**

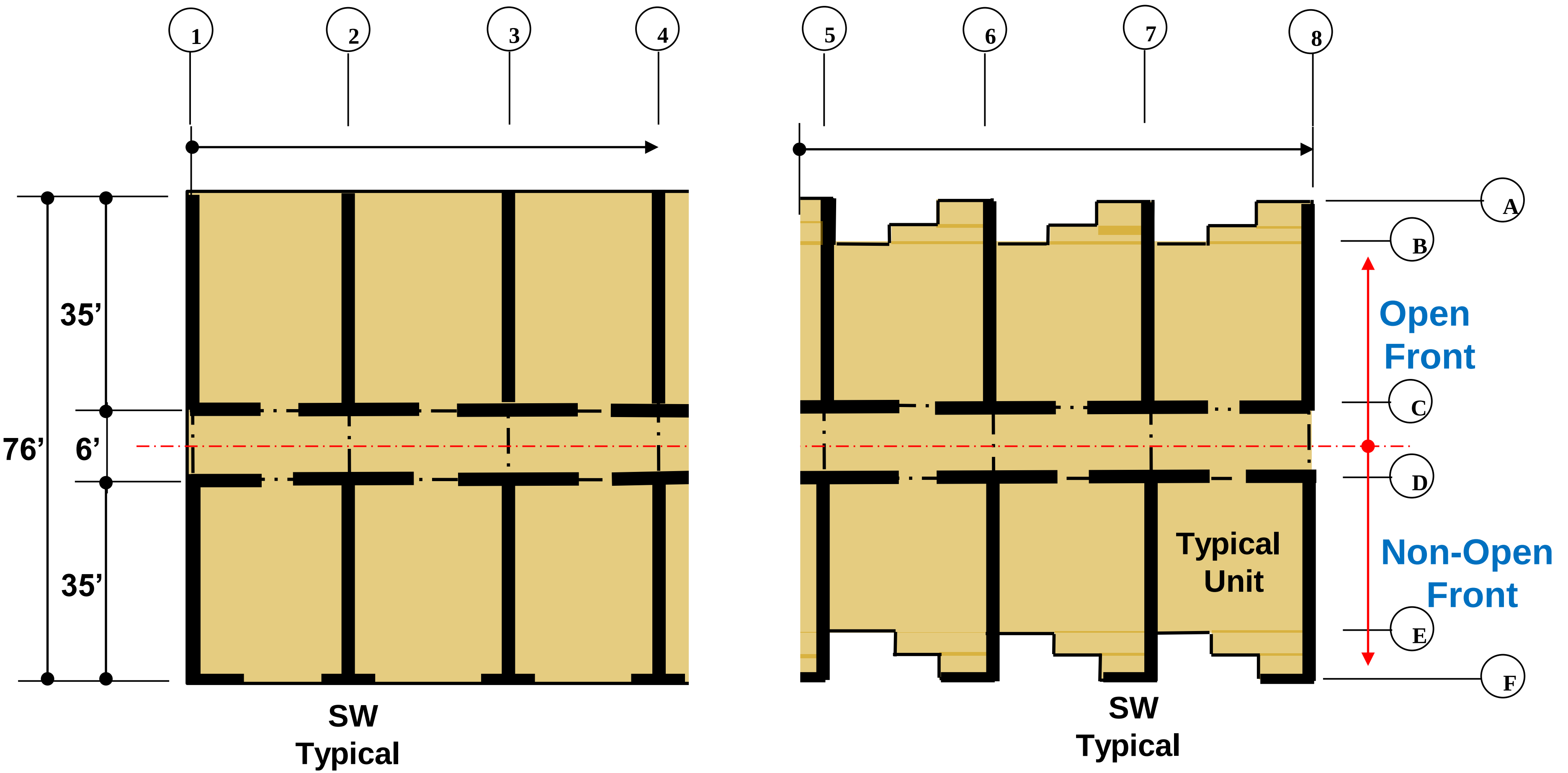
120 Union, San Diego, CA
Togawa Smith Martin

Mid-rise Diaphragms and Shear Walls

Crescent Terminus building- credit: Richard Lubrant.

Important Design considerations:

- **Continuous load paths**
- **Vertical/horizontal irregularities**
- **Diaphragm stiffnesses and flexibility**
- **Shear wall stiffness and the effects of tall walls**
- **Horizontal distribution of forces within the diaphragm and shear walls**
- **The effects of offset diaphragms and shear walls**



Open Front & Non-open Front Floor Plan w/ and w/o offsets

Non-open front

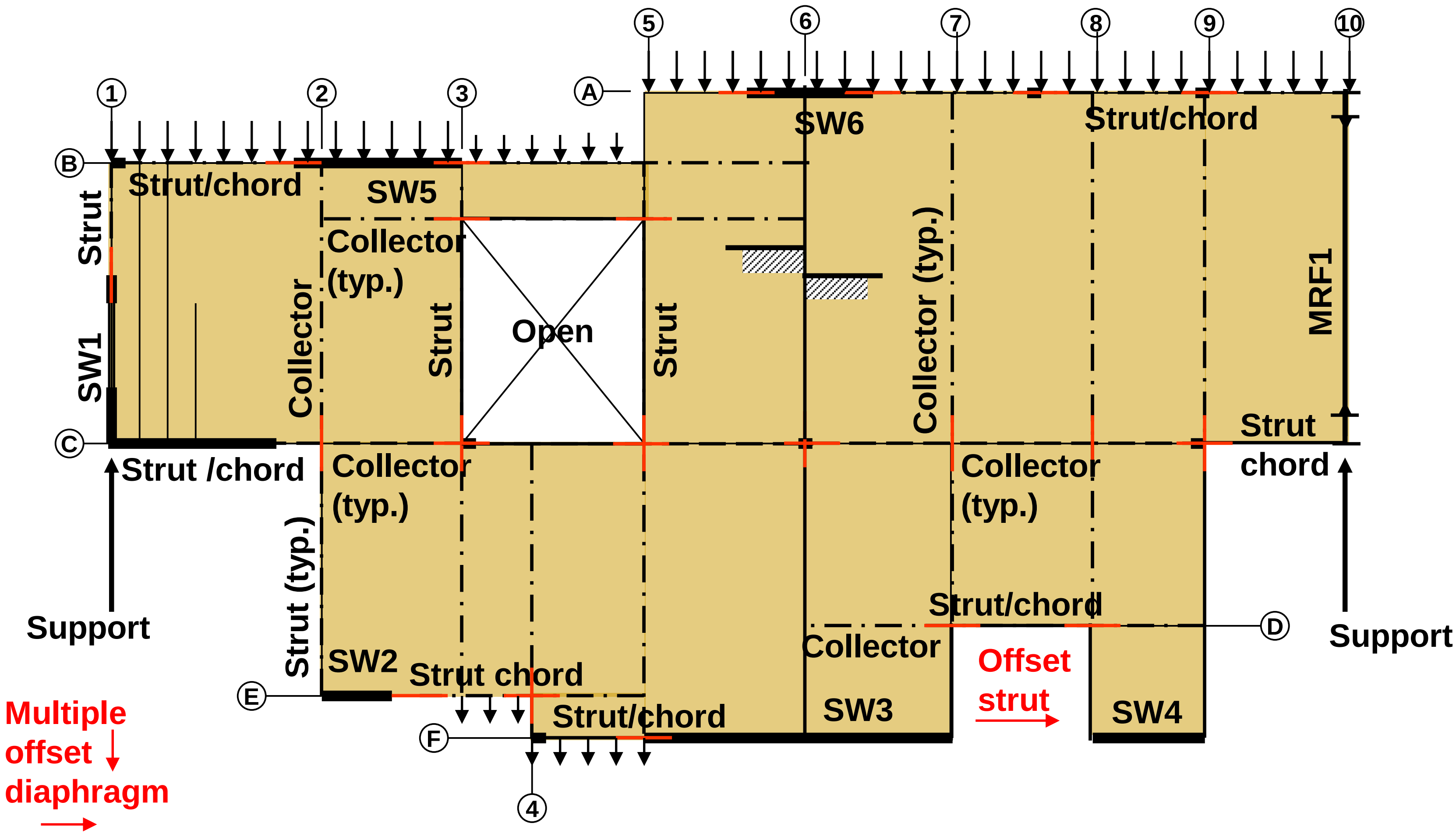
- **ASCE 7-10 Section 12.3.1.1- (c), Light framed construction, meeting all conditions:**
    - **Longitudinal-Typically permitted to be idealized as flexible, provided exterior shear walls exist. However, diaphragm can be semi-rigid or open front, even if exterior walls exist.**
    - **Transverse-Traditionally assumed to be flexible diaphragm.**
    - **If token or questionable wall stiffness at exterior, use semi-rigid analysis using the <u>envelope method-</u>(conservative), or semi-rigid, or idealize as rigid.**

Includes flexible and rigid analysis

Open Front Structures:

- **SDPWS 4.2.5.2 :**
    - **Loading parallel to open side - Model as semi-rigid (min.), shall include shear and bending deformation of the diaphragm, or idealized as rigid.**
    - **Loading perpendicular to open side-traditionally assumed to be flexible.**
    - **Drift at edges of the structure ≤ the ASCE 7 allowable story drift ratio when subject to seismic design forces including torsion, and accidental torsion (strength, multiplied by Cd). No set drift requirements required for wind (Drift can be tolerated).**

Complete Load Paths

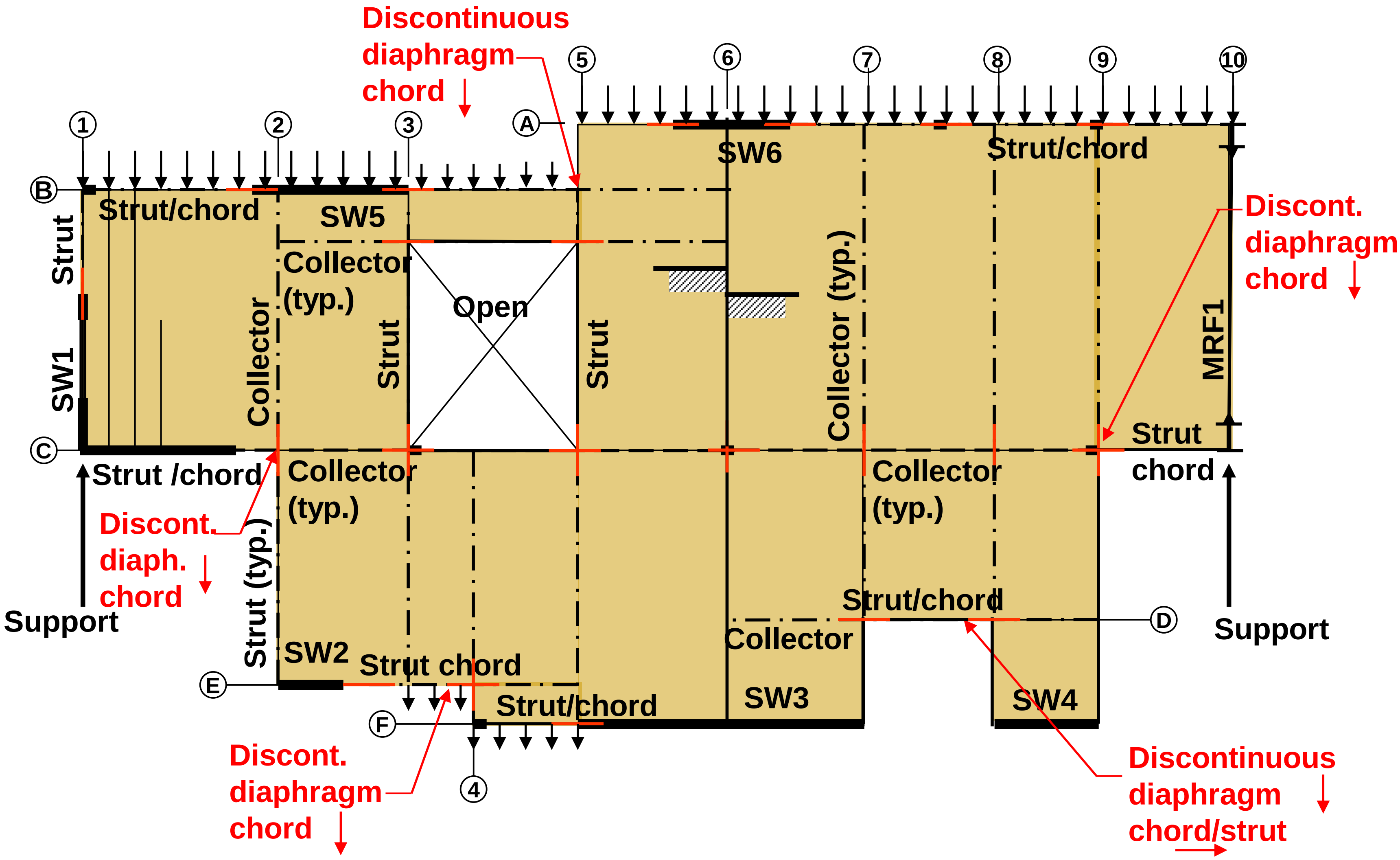


Analysis: ASCE7-10 Sections:

- 1.3.1.3.1-Design shall be based on a rational analysis
- 12.10.1-At diaphragm discontinuities such as openings and re-entrant corners, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

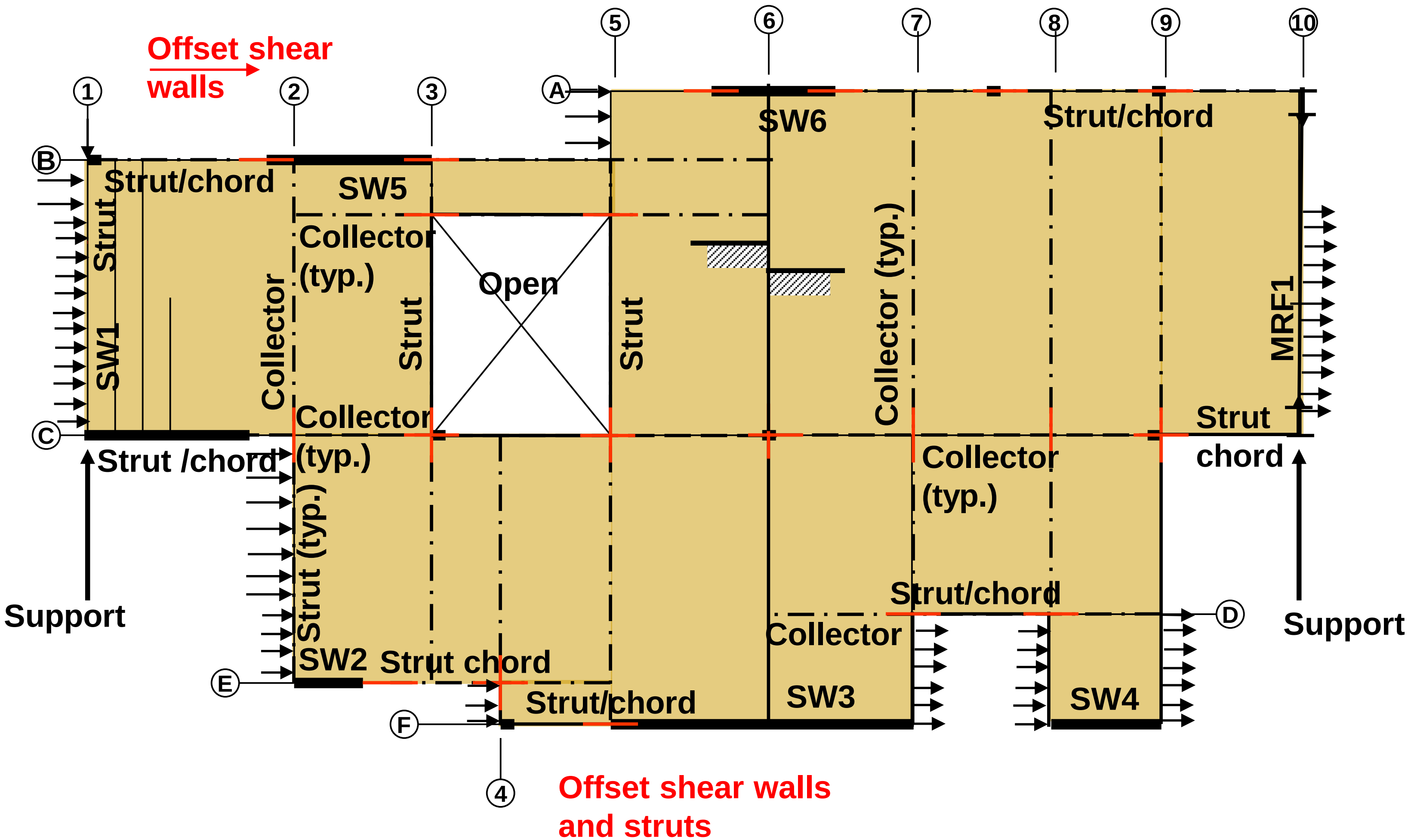
What does this mean?

Complete Continuous Lateral Load Paths



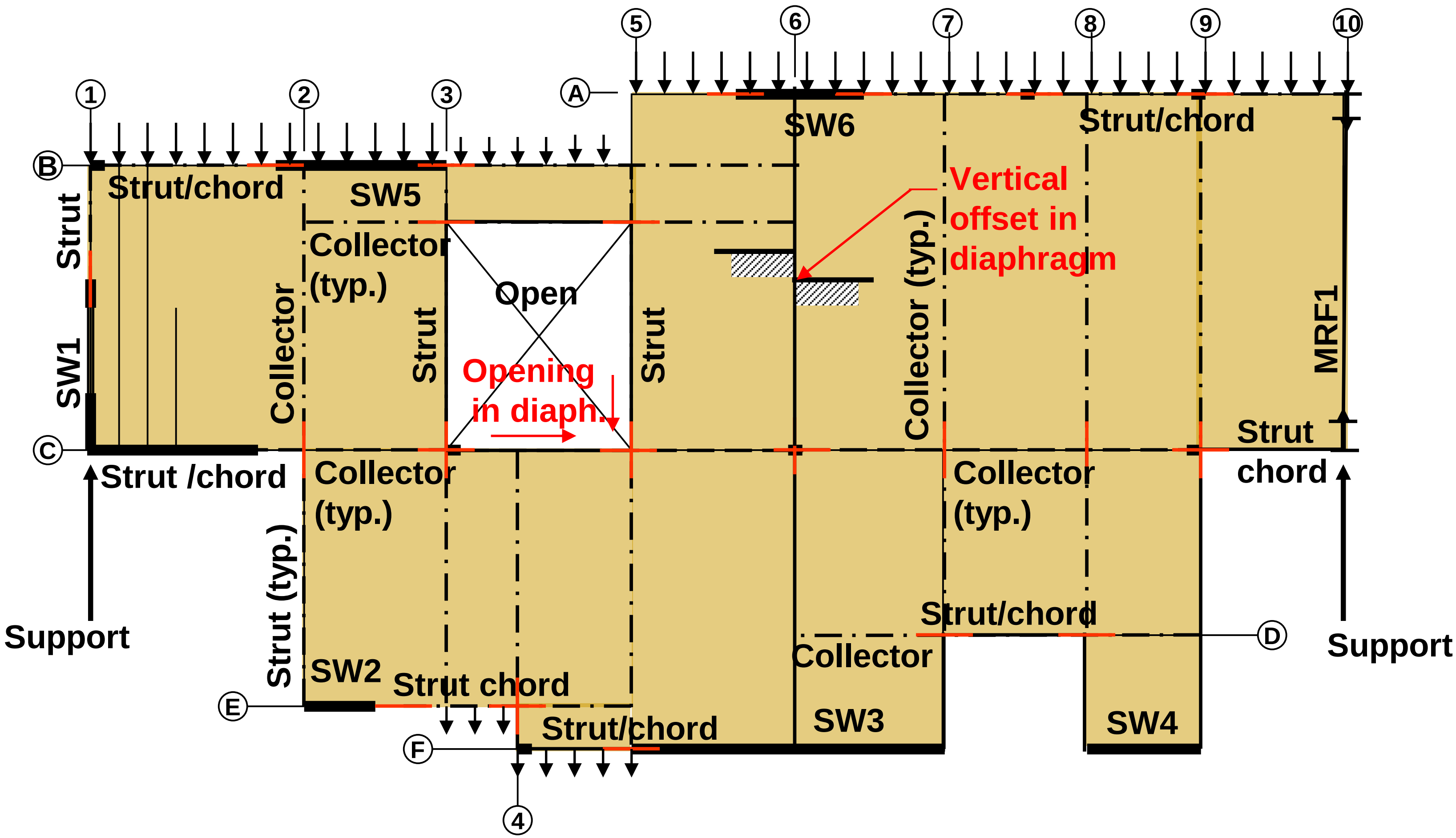
ASCE7-10 Section 1.4-Complete load paths are required including members and their splice connections

Complete Continuous Lateral Load Paths



ASCE7-10 Section **1.4-Complete load paths are required including members and their splice connections**

Complete Continuous Lateral Load Paths



Design:

- IBC 2305.1.1-Openings in shear panels that materially effect their strength shall be fully detailed on the plans and shall have their edges adequately reinforced to transfer all shear stresses.

Complete Continuous Lateral Load Paths

ASCE7-10 Section 12.3 Diaphragm Flexibility Seismic

Section 12.3.1- The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the lateral force resisting system.

Start

Is diaphragm untopped steel decking or Wood Structural Panels

Yes

Is any of the following true?		
1 & 2 family Dwelling	Vertical elements one of the following : 1. Steel braced frames 2. Composite steel and concrete braced frames 3. Concrete, masonry, steel SW or composite concrete and steel shear walls.	Light framed construction where all of the following are met: 1. Topping of concrete or similar material is not placed over wood structural panel diaphragms except for non-structural topping not greater than 1 ½" thick. 2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12-1.

Yes

Idealize as flexible

No

No

Is diaphragm concrete slab or concrete filled steel deck ?

Yes

Is span to depth ratio ≤ 3 and having no horizontal irregularities ?

Yes

Idealize as rigid

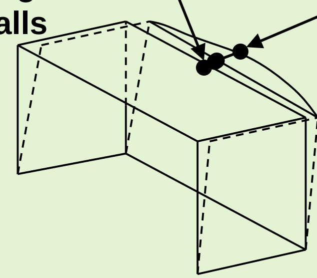
No

No

(Could apply to CLT or Heavy timber diaphragms)

Average drift of walls

Maximum diaphragm deflection



Is maximum diaphragm deflection (MDD) >2x average story drift of vertical elements, using the Equivalent Force Procedure of Section 12.8?

Yes

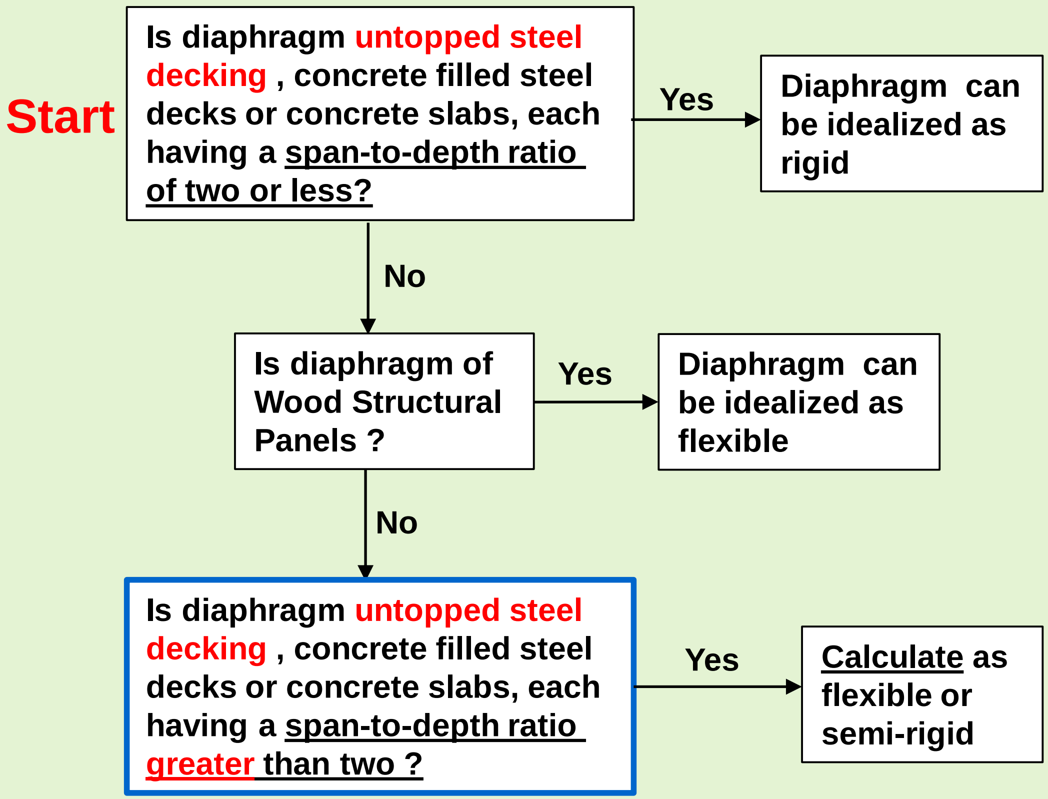
Idealize as flexible

No

Structural analysis must explicitly include consideration of the stiffness of the diaphragm (i.e. semi-rigid modeling).

Semi-rigid (Envelope Method)

ASCE7-10, Section 26.2 Diaphragm Flexibility Wind



Tall Shear Walls

Marselle Condominiums
Engineer engineer: Yu & Trochalakis, PLLC
Photographer: Matt Todd Photographer

5 stories of wood over 6 stories concrete Structural (podium) 2 above grade

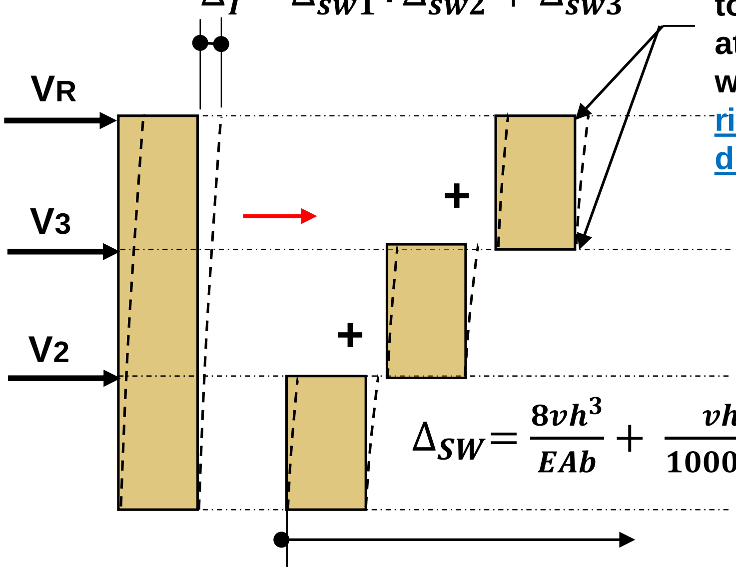
Tall Shear Walls

Testing shows that the traditional deflection equation is less accurate for walls with aspect ratios higher than 2:1.
(Dolan)

- **Traditional floor to floor method is good for 3 stories or less (w/ exception)**

Traditional Shear Walls

$$\Delta_T = \Delta_{sw1} + \Delta_{sw2} + \Delta_{sw3}$$



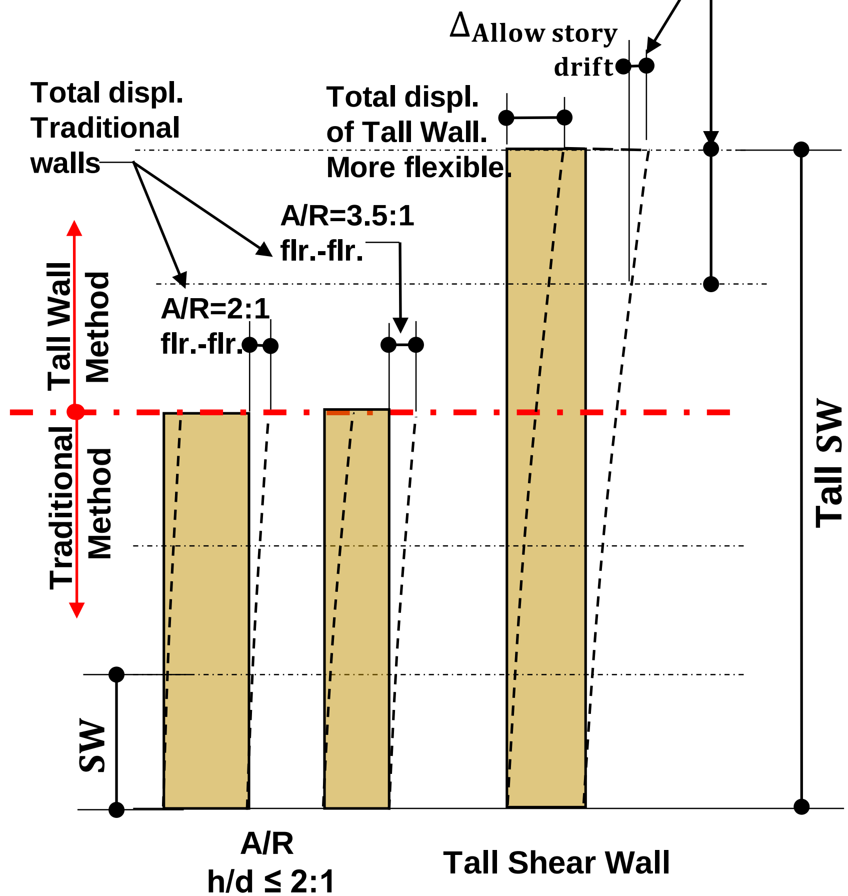
Walls assumed to be pinned at top and bottom at each floor, rigid wall sections and [rigid out-of-plane diaphragm stiffness]

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

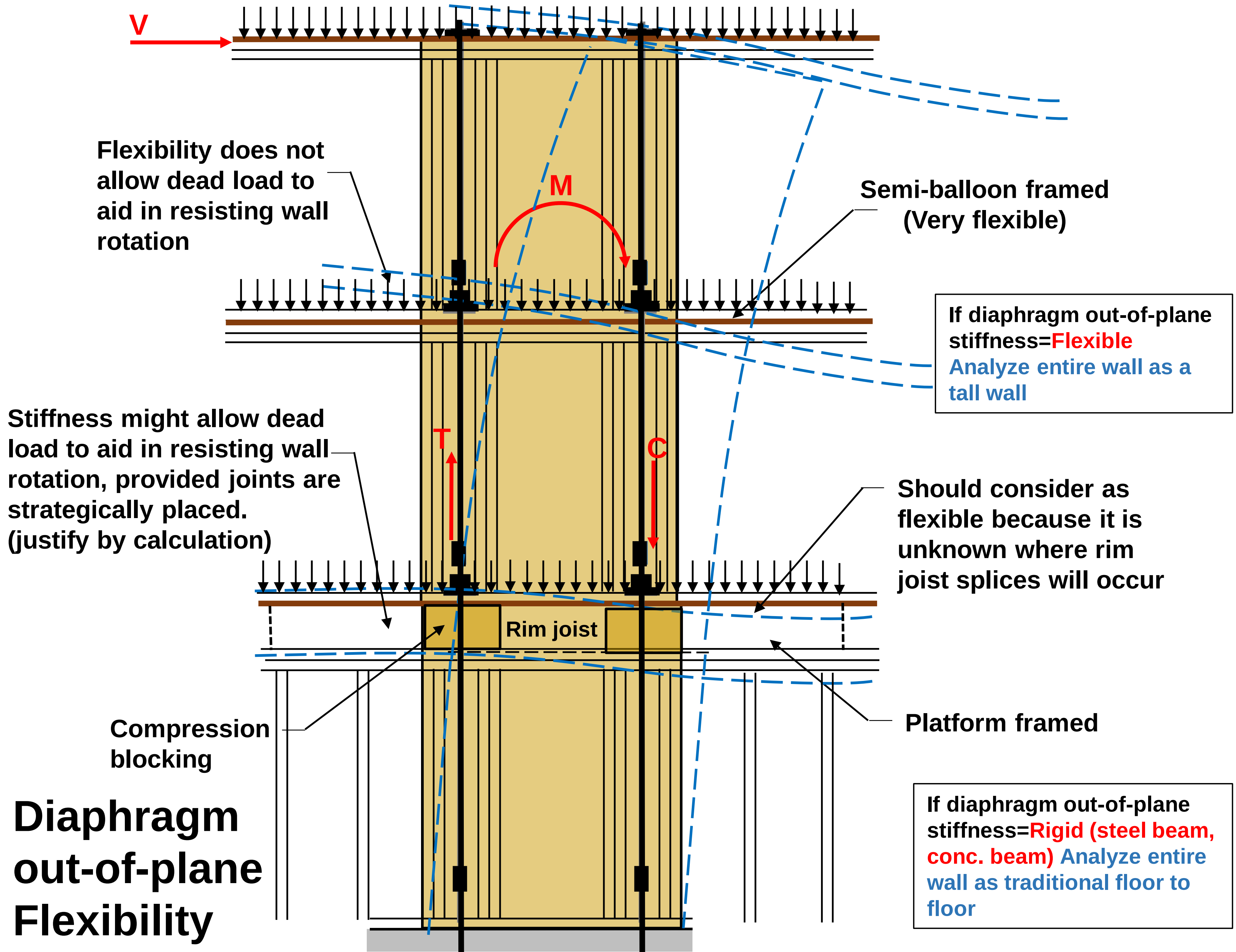
Doesn't account for multi-story shear wall effects

Tall Shear Walls

- **Tall walls greater than 3 stories should consider flexure and wall rotation.**
  - **Rotation and moment from walls above and wall rotation effects from walls below.**
- **Allowable story drift for traditional and tall shear walls is checked floor to floor.**



Floor to floor A/R's and Stiffness of Shear Walls


V
Flexibility does not allow dead load to aid in resisting wall rotation
M
Semi-balloon framed (Very flexible)
If diaphragm out-of-plane stiffness=Flexible Analyze entire wall as a tall wall
Stiffness might allow dead load to aid in resisting wall rotation, provided joints are strategically placed. (justify by calculation)
T
C
Should consider as flexible because it is unknown where rim joist splices will occur
Rim joist
Compression blocking
Platform framed
If diaphragm out-of-plane stiffness=Rigid (steel beam, conc. beam) Analyze entire wall as traditional floor to floor
Diaphragm out-of-plane Flexibility

Current Examples of Tall Shear Wall Analysis

- **FPInnovations-Website**

  ”A Mechanics-Based Approach for Determining Deflections of Stacked Multi-Storey Wood-Based Shear Walls”, Newfield

  Design Example: ”Design of Stacked Multi-Storey Wood-Based Shear Walls Using a Mechanics-Based Approach ”, Canadian Wood Council

- **APEGBC Technical & Practice Bulletin Revised April 8, 2015**

  “5 and 6 Storey Wood Frame Residential Building Projects (Mid-Rise)”-Based on FPInnovations Mechanics Based Approach

Current Examples of Mid-rise Analysis

- **Shiotani/Hohbach Method-Woodworks Slide archive**

  http://www.woodworks.org/wp-content/uploads/HOHBACH-Mid-Rise-Shear-Wall-and-Diaphragm-Design-WSF-151209.pdf

- **Thompson Method-Woodworks Website**

  Webinar http://www.woodworks.org/education/online-seminars/

  Paper http://www.woodworks.org/wp-content/uploads/5-over-1-Design-Example.pdf

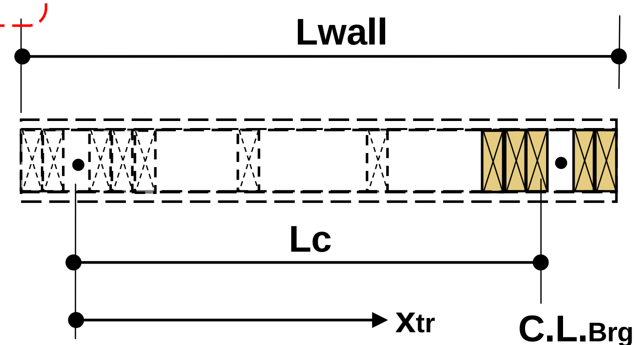
Deflections of Stacked Multi-story Shear Walls-U.S. Standards

Based on FPInnovations ”A Mechanics-Based Approach for Determining Deflections of Stacked Multi-Storey Wood-Based Shear Walls”, Newfield-(Metric)

$$\Delta_i = \frac{\sum M_i H_i^2}{2(EI)_i} + \frac{\sum V_i (H^3)}{3(EI)_i} + \frac{V_i H_i}{G_{v,i} t_{v,i}} + 0.75 H_i e_{n,i} + \frac{H_i}{L_i} d_{a,i} + \sum_{j-1}^{i-1} (\Delta_{b,i} + \Delta_{r,i})$$

U.S. customary units

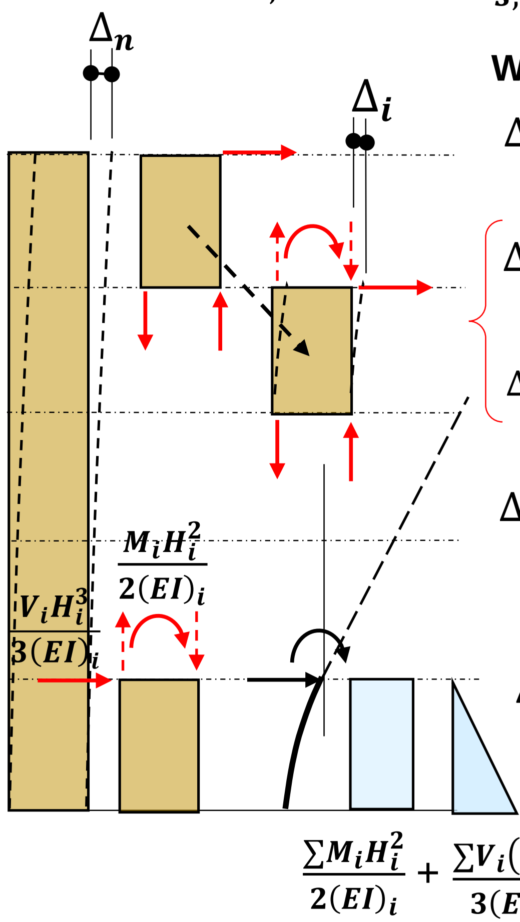
Bending deflection $\Delta_{b,i}$ | Shear Defl. $\Delta_{s,i}$ | Nail Slip $\Delta_{n,i}$ | Rod Elong. crushing $\Delta_{a,i}$ | Sum rotation below $\Delta_{r,i}$



ATS - cont. hold down used

$$I_{tr} = A_{t,tr}(d_1^2) + A_c(d_2)^2$$

Transformed composite section



Where

$$\Delta_{b,i} = \frac{V_i H_i^3}{3(EI)_i} + \frac{M_i H_i^2}{2(EI)_i}$$

$$\Delta_{s,i} = \frac{V_i H_i}{G_{v,i} t_{v,i}}$$

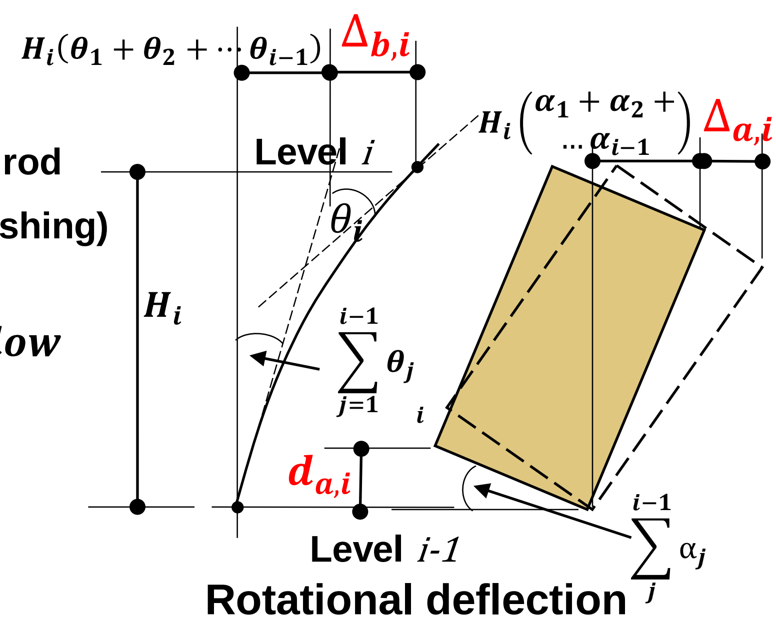
Alt.

$$\Delta_{s,i} = \frac{V_i H_i}{1000 G_{a,i}}$$

$$\Delta_{n,i} = 0.75 H_i e_{n,i}$$

$\Delta_{a,i} = \frac{H_i}{L_i} d_{a,i}$ **(Includes affects of rod elongation and crushing)**

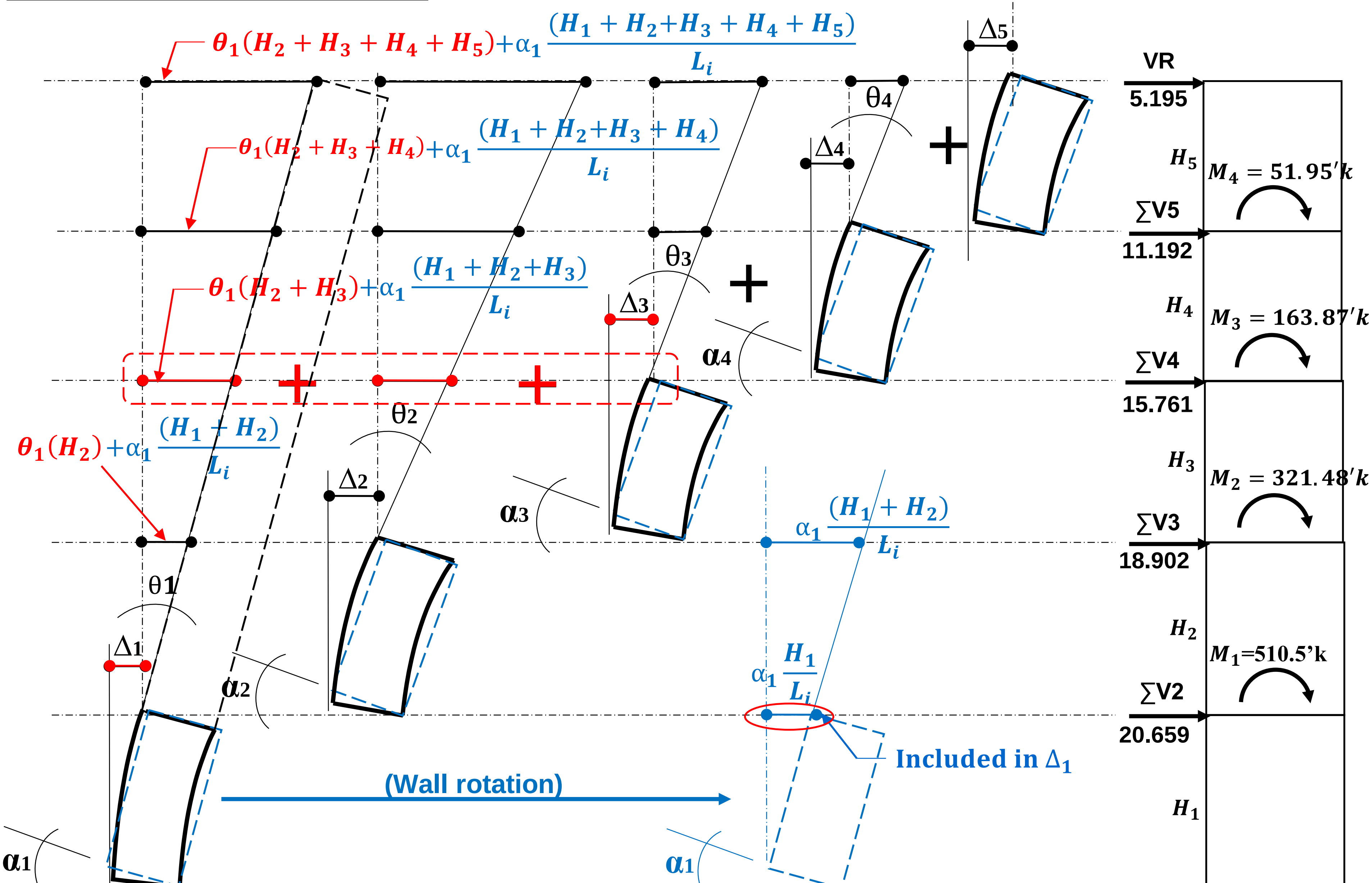
$$\Delta_{r,i} = \sum_{j-1}^{i-1} (\Delta_{b,i} + \Delta_{r,i}) \textit{ walls below}$$



Rotational deflection

Tall Wall Deflection

Example 3rd floor $\Delta_{Total} = \Delta_1 + \theta_1(H_2 + H_3)(1728) + \Delta_2 + \theta_2 H_3(1728) + \Delta_3$



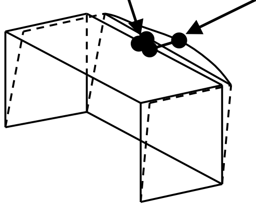
Deflection-Bending +Rotation
translates to top

Deflection-Wall rotation)
translates to top

2015 SDPWS 4.2.5 Horizontal Distribution of Shear (Revised)

Distribution of shear to vertical resisting elements shall be based on:

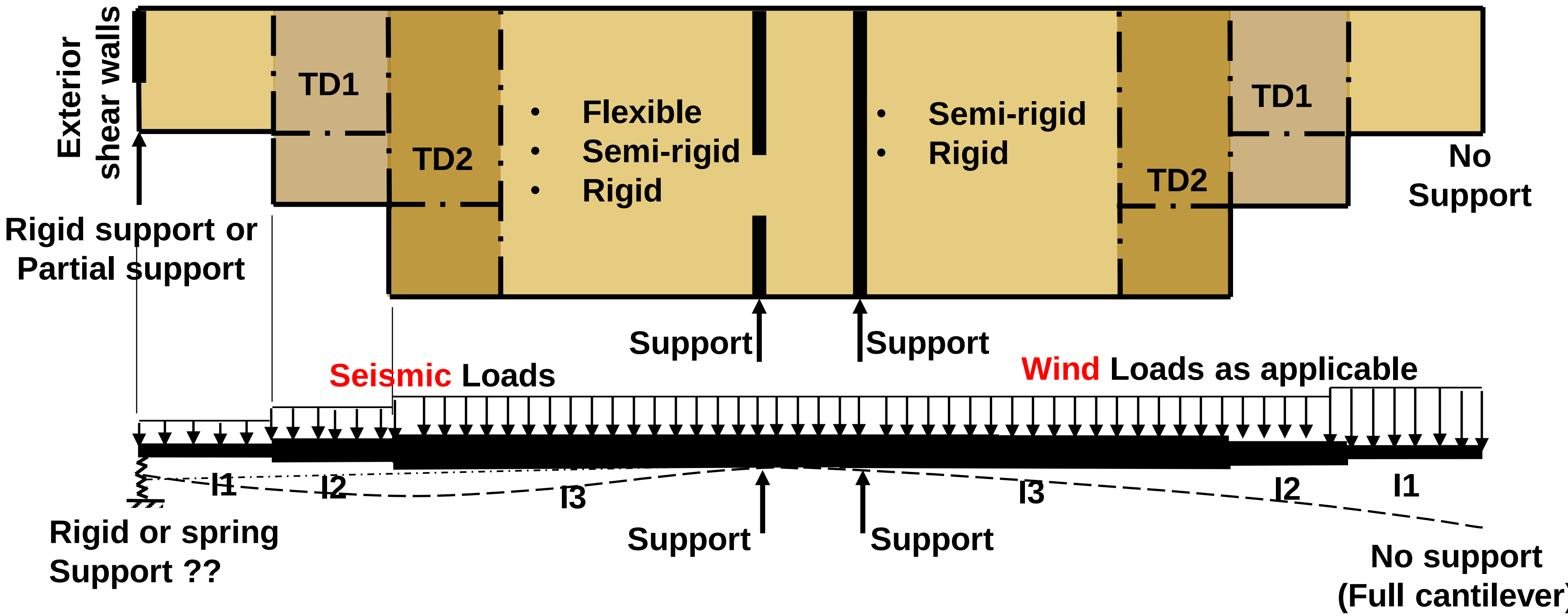
- **Analysis where the diagram is modeled as :**
  - **Idealized as flexible-based on tributary area.**
    - Can under-estimate forces distributed to the corridor walls (long walls) and over-estimate forces distributed to the exterior walls (short walls)
    - Can inaccurately estimate diaphragm shear forces
  - **Idealized as rigid-Distribution based on relative lateral stiffnesses of vertical-resisting elements of the story below.**
    - More conservatively distributes lateral forces to corridor, exterior and party walls
    - Allows easier determination of building drift
    - Can over-estimate torsional drift
    - Can also inaccurately estimate diaphragm shear forces
  - **Modelled as semi-rigid.**
    - **Not idealized as rigid or flexible**
    - **Distributed to the vertical resisting elements based on the relative stiffnesses of the diaphragm <u>and</u> the vertical resisting elements accounting for both shear and flexural deformations.**
    - **In lieu of a semi-rigid diaphragm analysis, it shall be permitted to use an enveloped analysis (Minimum analysis).**



Maximum diaphragm deflection (MDD) >2x average story drift of vertical elements, using the ELF Procedure of Section 12.8?

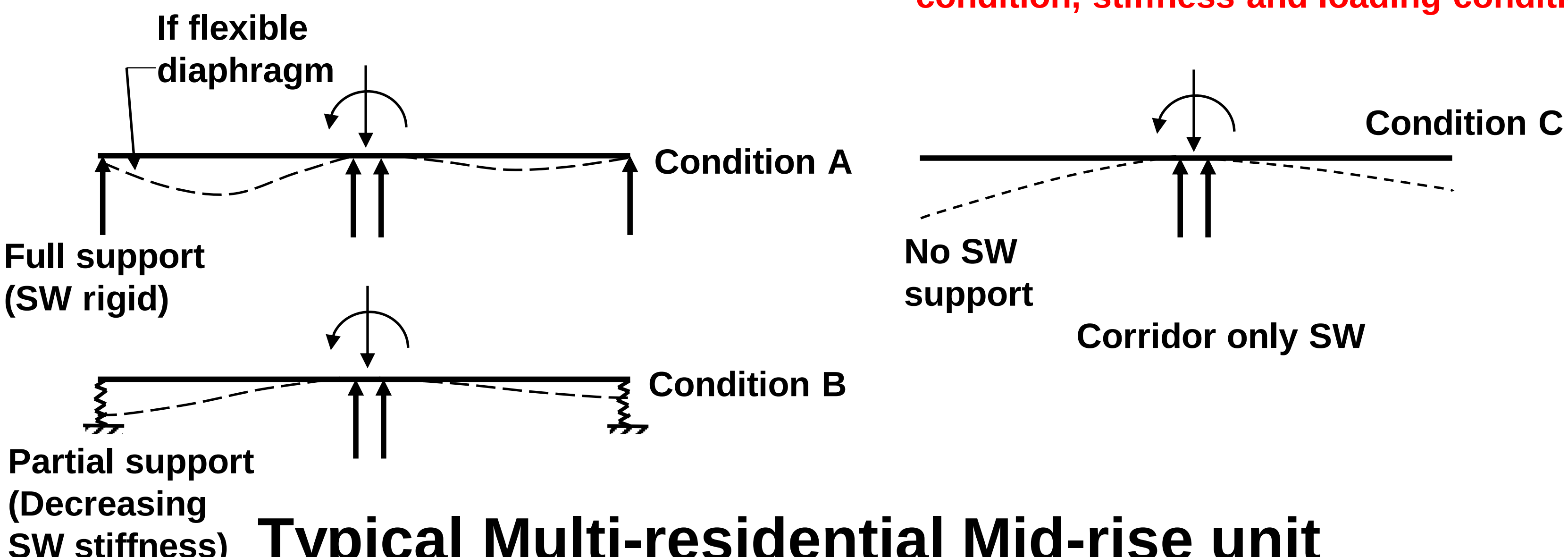
<u>Calculated as Flexible</u>

Note:
Offsets in diaphragms can also affect the distribution of shear in the diaphragm due to changes in the diaphragm stiffness.



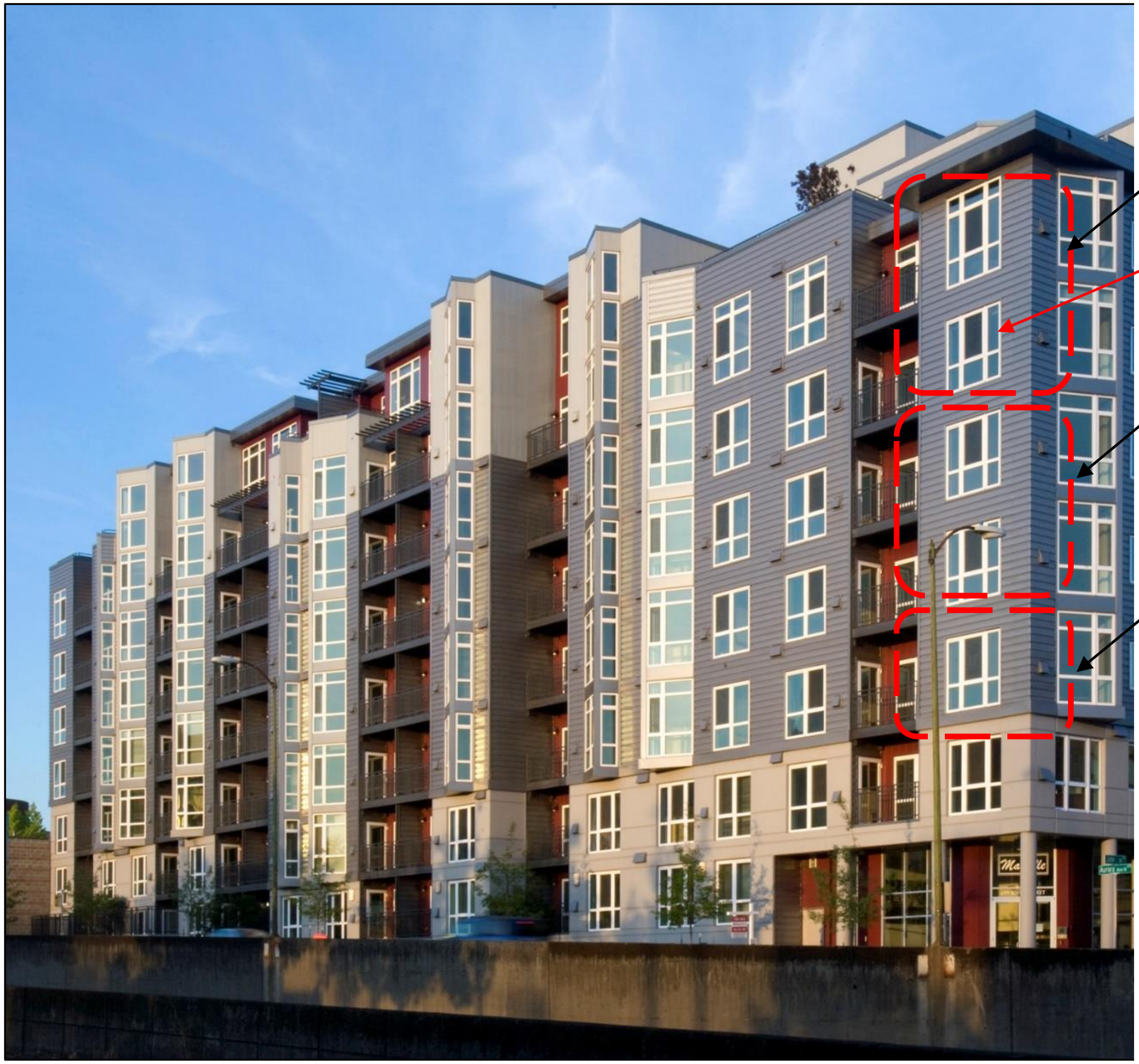
$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_C X)}{2b} \quad \text{IBC Eq. } 23-1$$

Adjust terms of the equation for support condition, stiffness and loading conditions



Typical Multi-residential Mid-rise unit

Effects of Tall Shear Walls



open front?

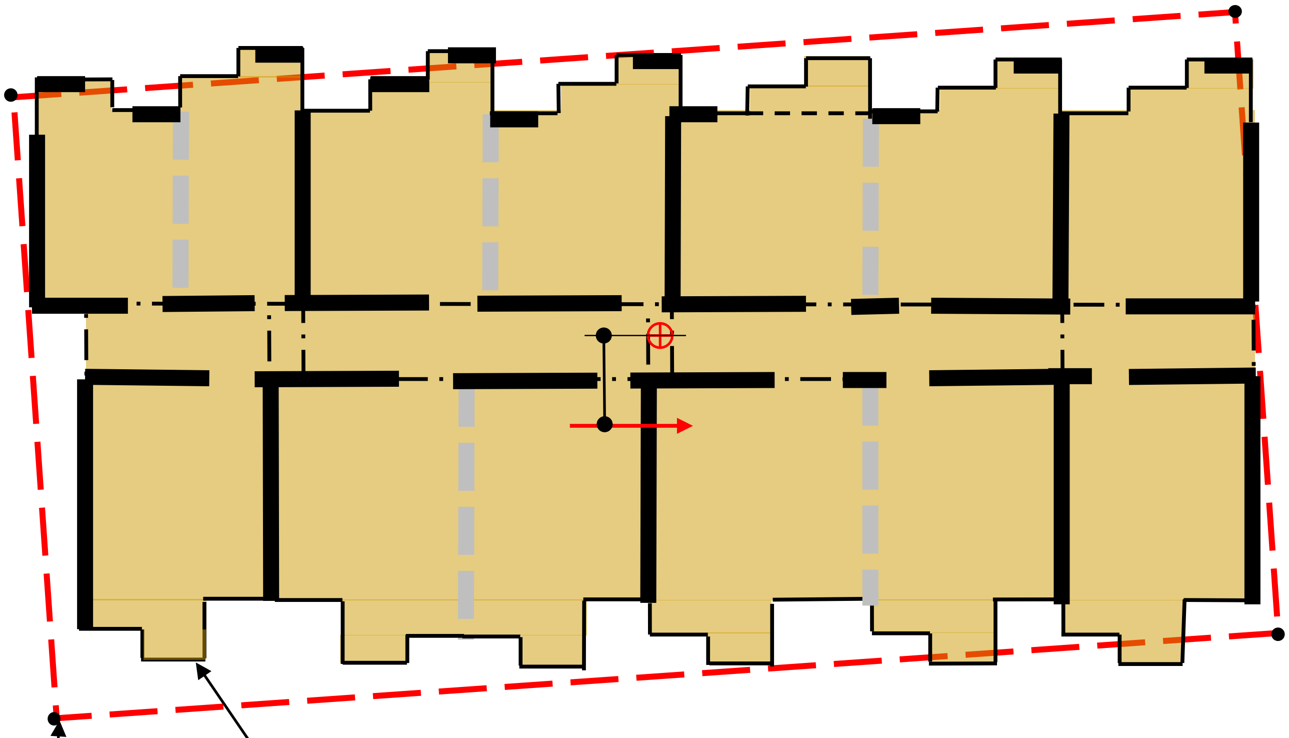
Effects of openings?

Semi-rigid?

Flexible?

Note that possible changes in diaphragm flexibility can occur from floor to floor: non-open front (flexible) vs. non-open front or open front (rigid or semi-rigid).

Longitudinal Loading

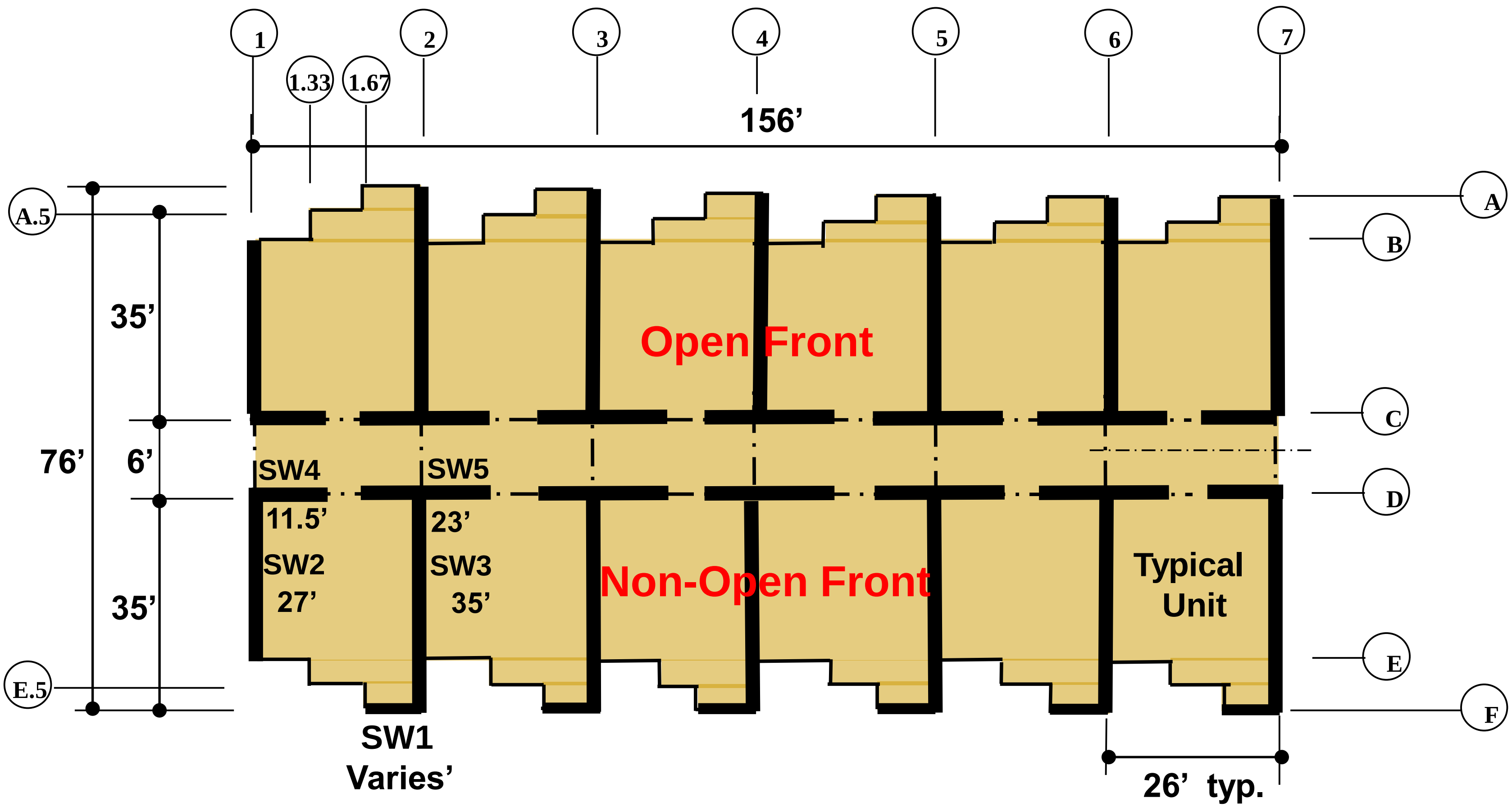


No exterior
Shear walls ??

Drift by semi-rigid or
rigid analysis only

Calculation of drift at corners is required for all open front or cantilever diaphragms **(seismic only)** including torsion and accidental torsion (Deflection-strength level amplified by Cd.)

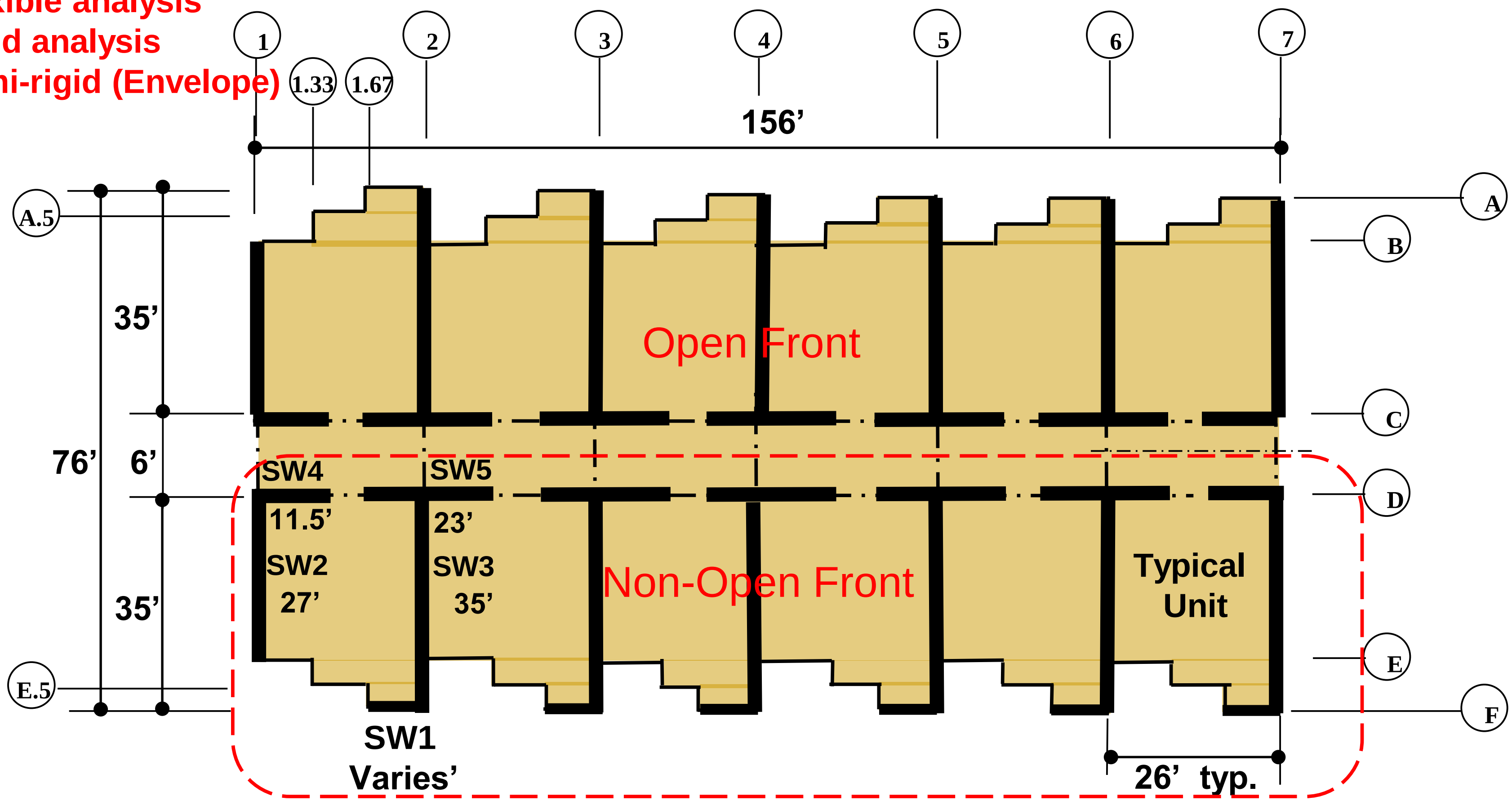
Case Studies-Open-front & Non-Open Front w/ Offsets



The following information is from an on-going example calculation and is subject to further revisions and validation. The information provided is project specific, and is for informational purposes only. It is not intended to serve as recommendations or as a recognized method of analysis, and as such is based on the authors opinions only.

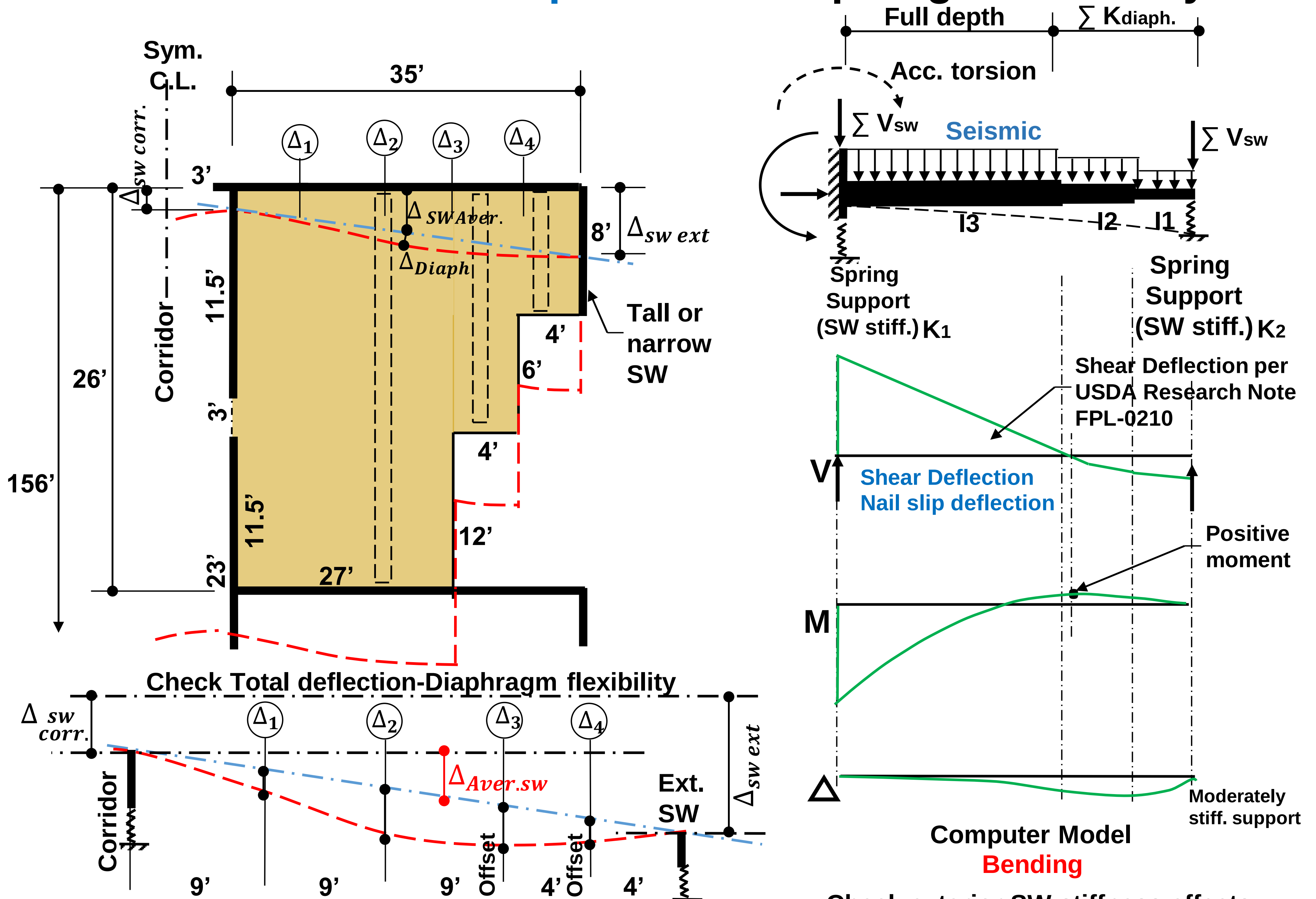
Case Study-Non-Open Front Floor Plan With Offsets

- Flexible analysis
- Rigid analysis
- Semi-rigid (Envelope)



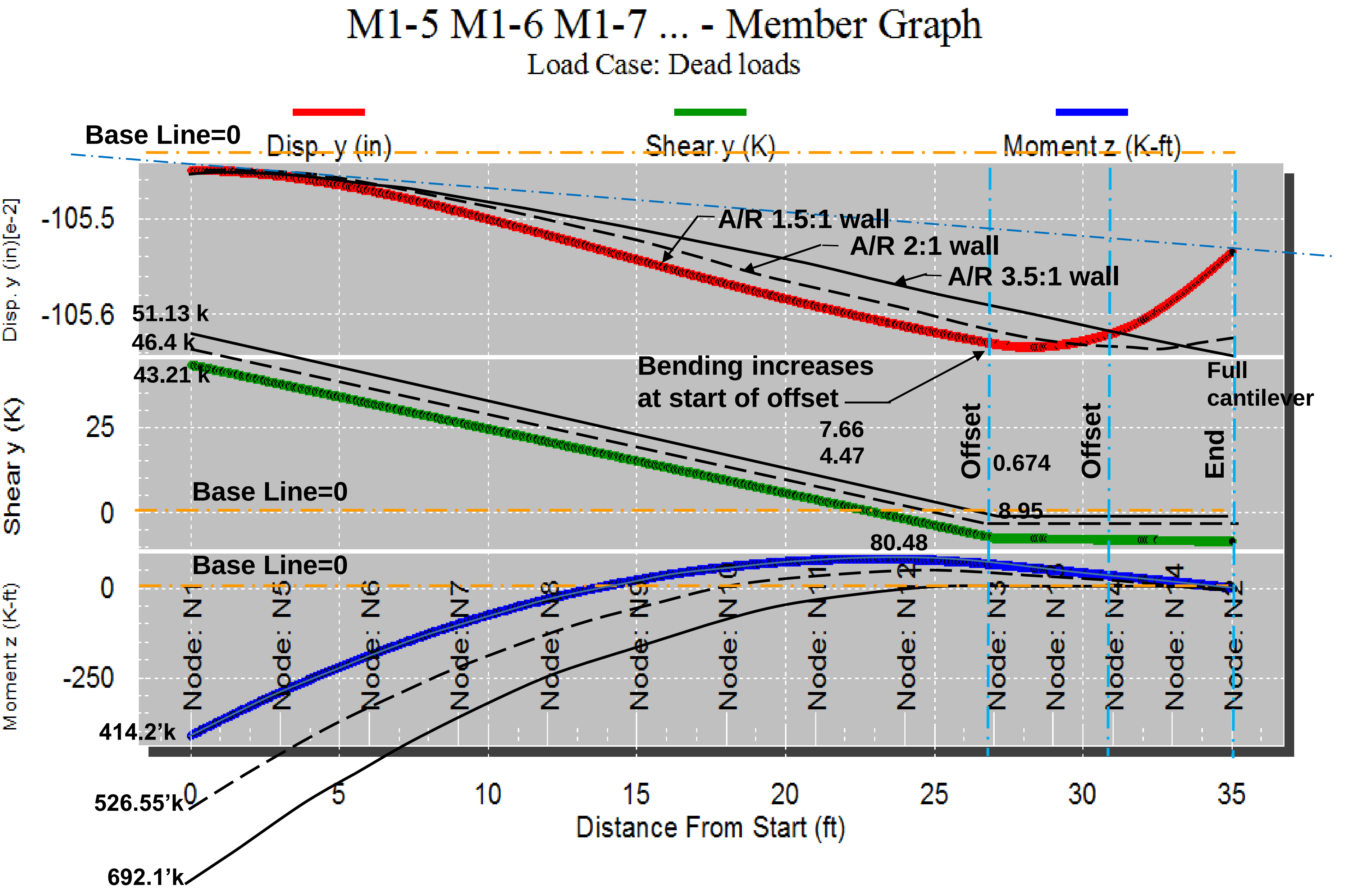
- ASCE 7-10 Section 12.3.1.1- (c), Light framed construction, meeting all conditions:
  - All Light framed construction
  - Non-structural concrete topping ≤ 1 ½"
  - <u>Each elements of the seismic line of vertical force-resisting system complies with the allowable story drift of Table 12.12-1</u>
  - Longitudinal-Allowed to be idealized as flexible, provided exterior shear walls exist and is in compliance with ASCE 7-10 Section 12.3.1.1 (c). If calculations show that the drift at a line of lateral force resistance exceeds the allowable limit, flexible diaphragm behavior cannot be used.

Determination of Non-Open Front Diaphragm Flexibility

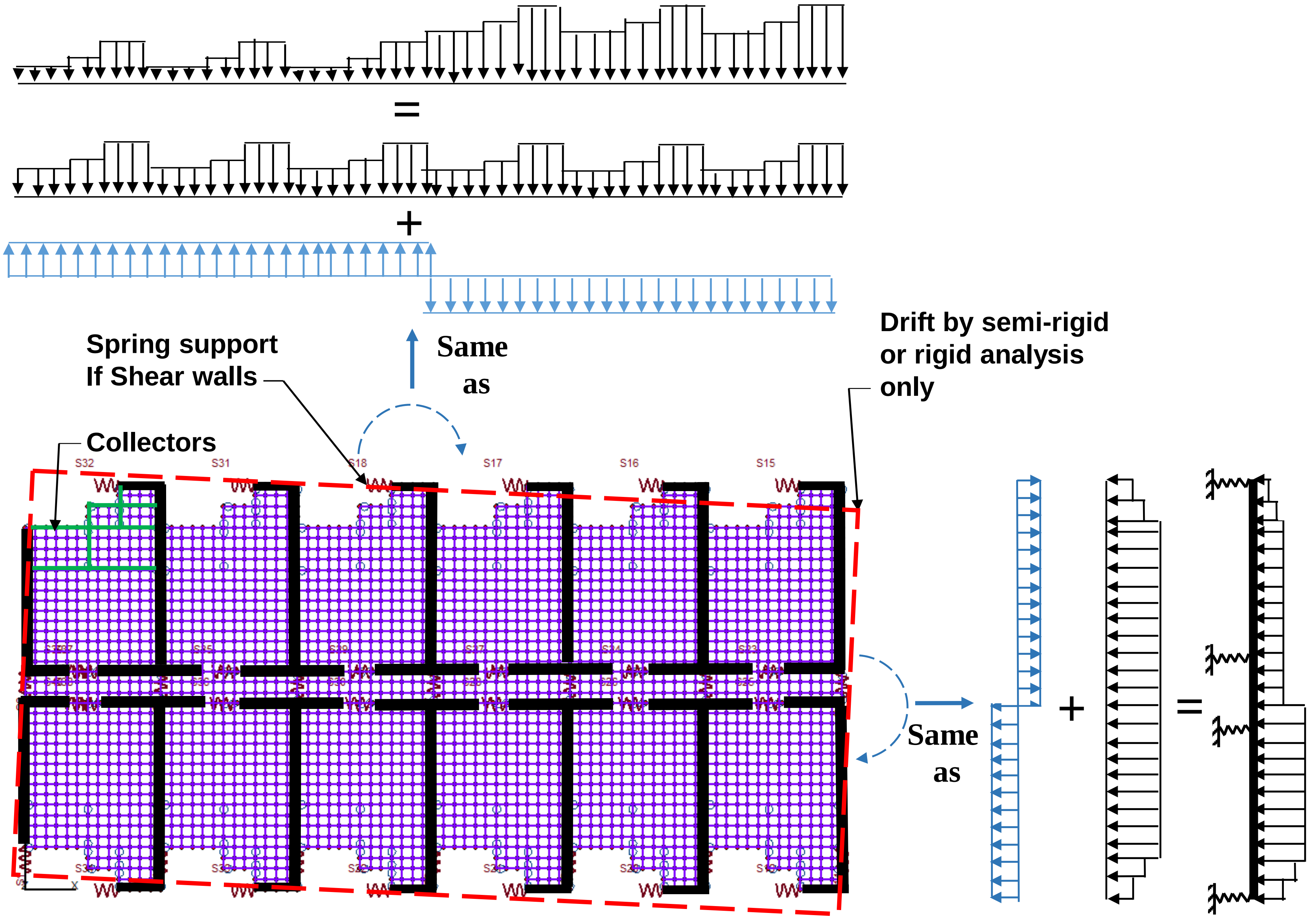


Check exterior SW stiffness effects

Roof Plot- Longitudinal Loads- A/R=1.5:1 (8” wall)

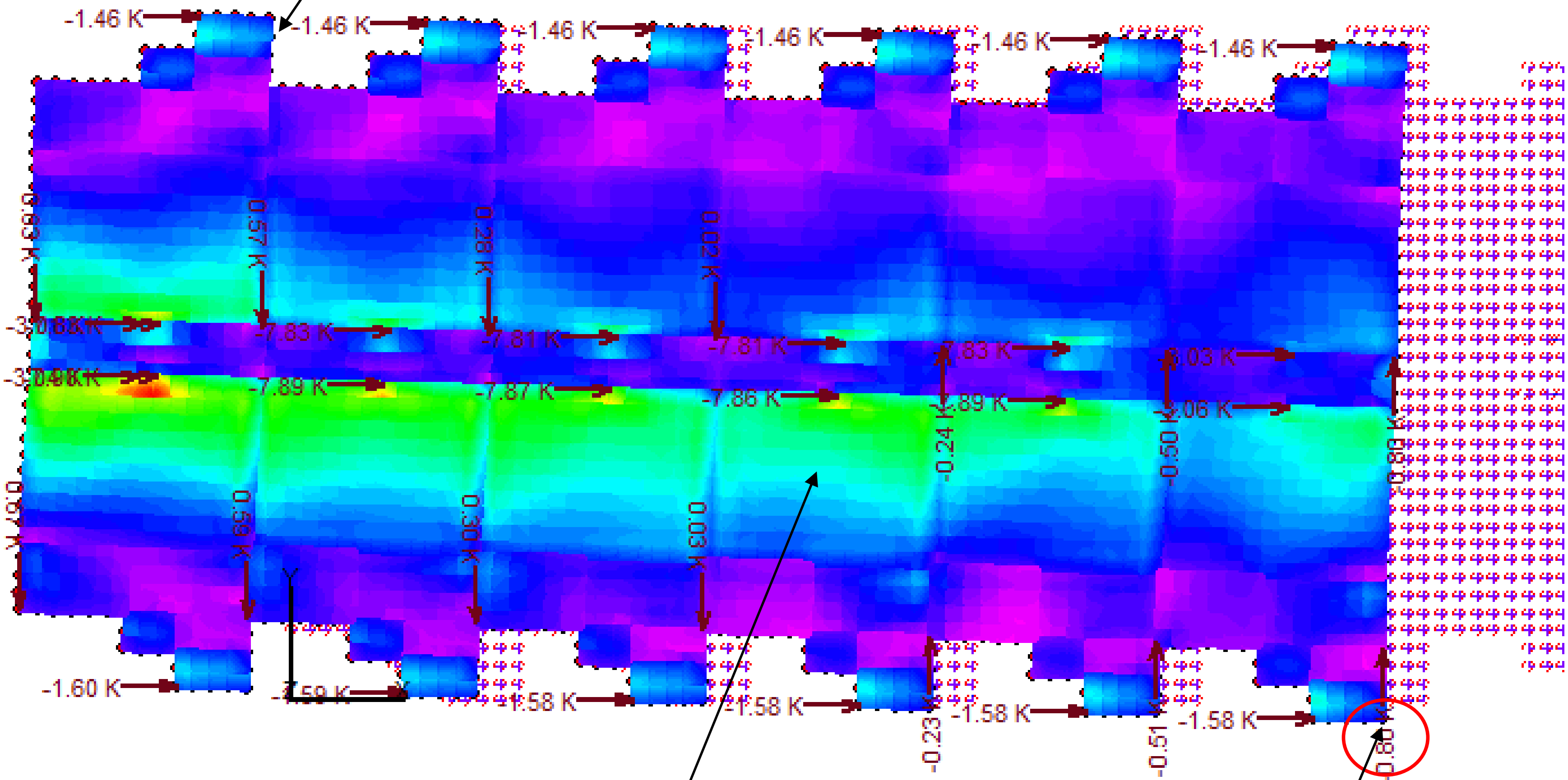


Check Story Drift-3D model or 2D plane Grid



Story Drift Check Longitudinal + Torsion (8’ ext. walls)

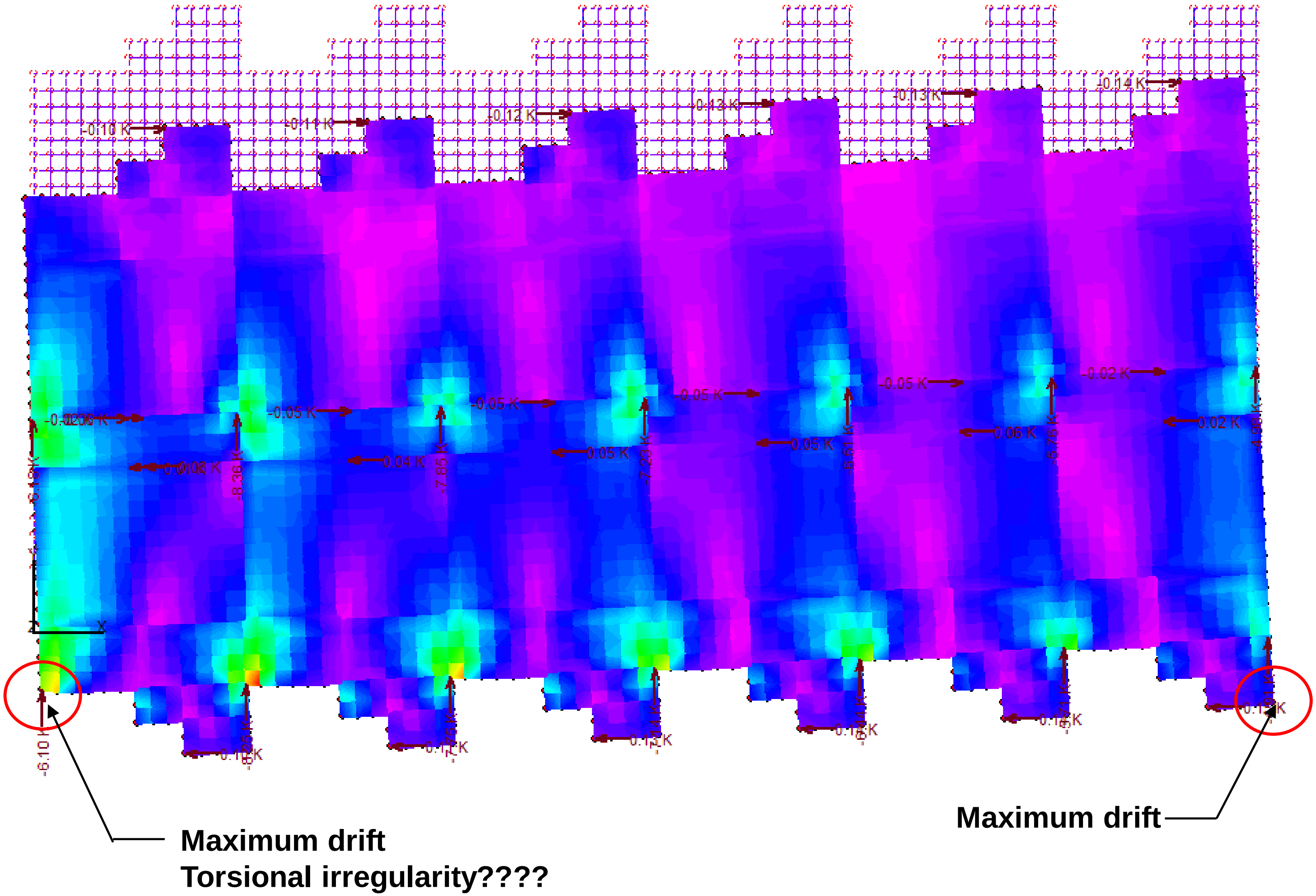
Notice curvature (bending) of offset sections



Significant shear distributed to corridor walls (cantilever action)

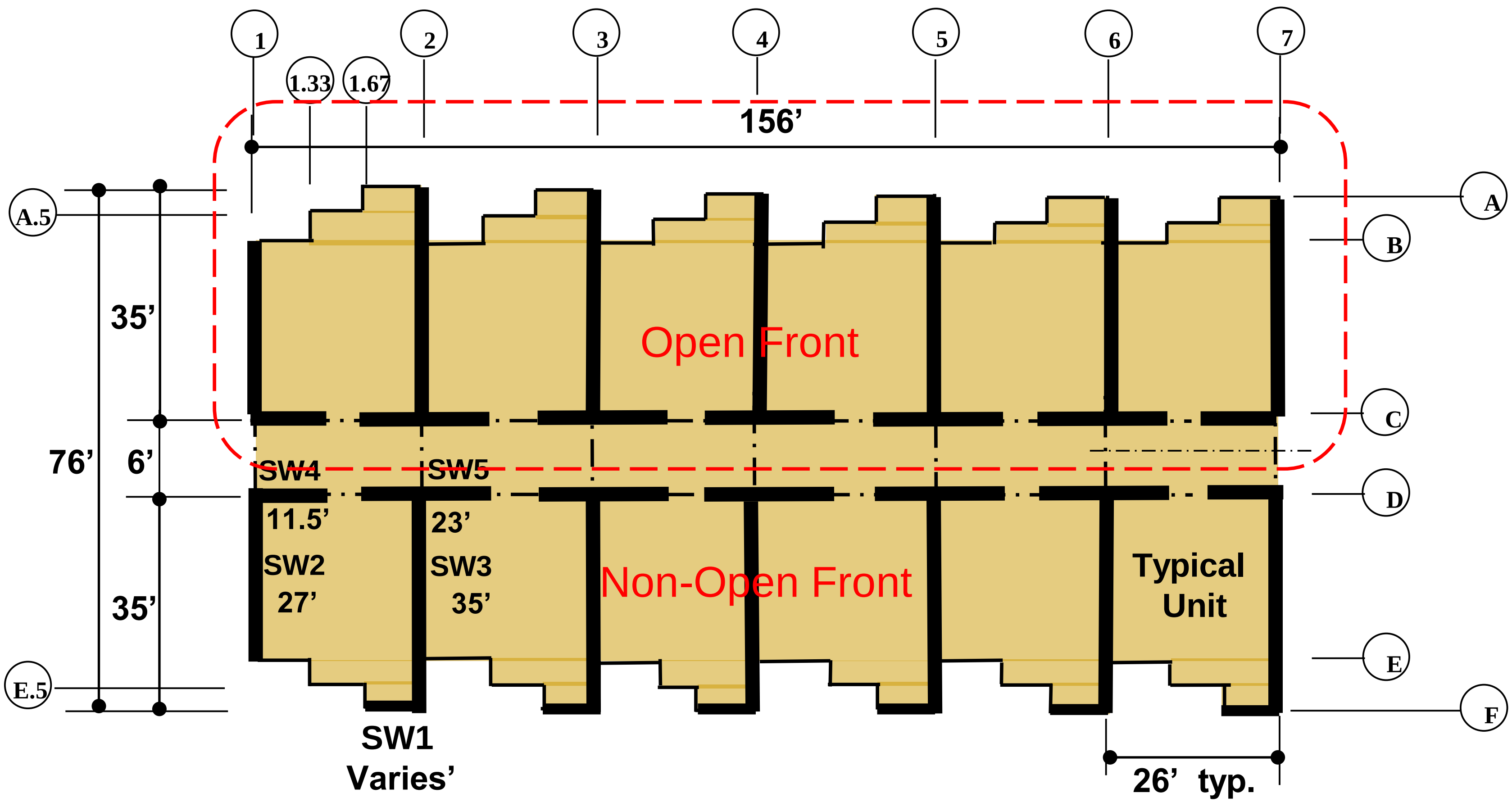
Maximum drift

Story Drift Check Transverse + Torsion

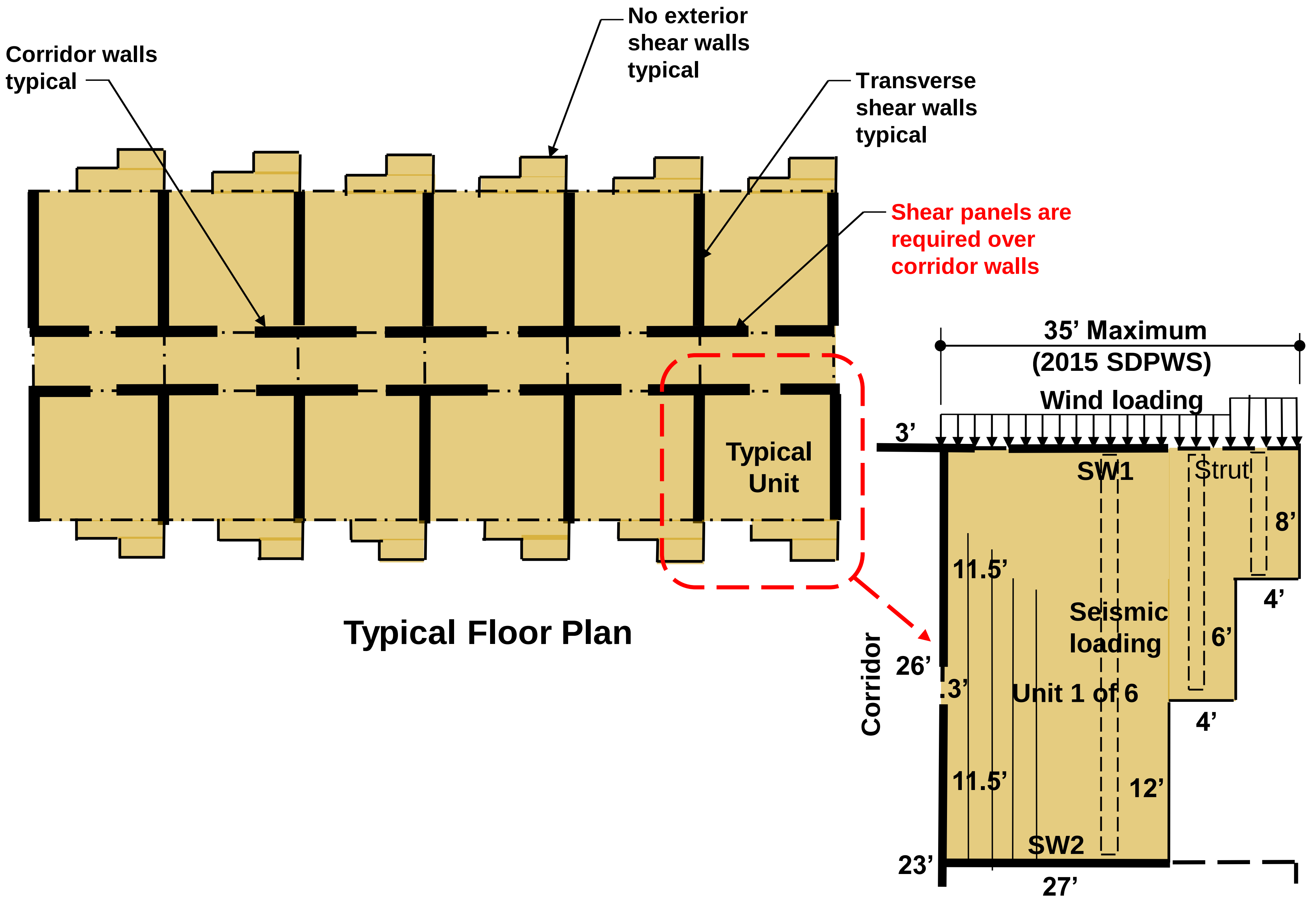


Case Study-Open Front Floor Plan With Offsets

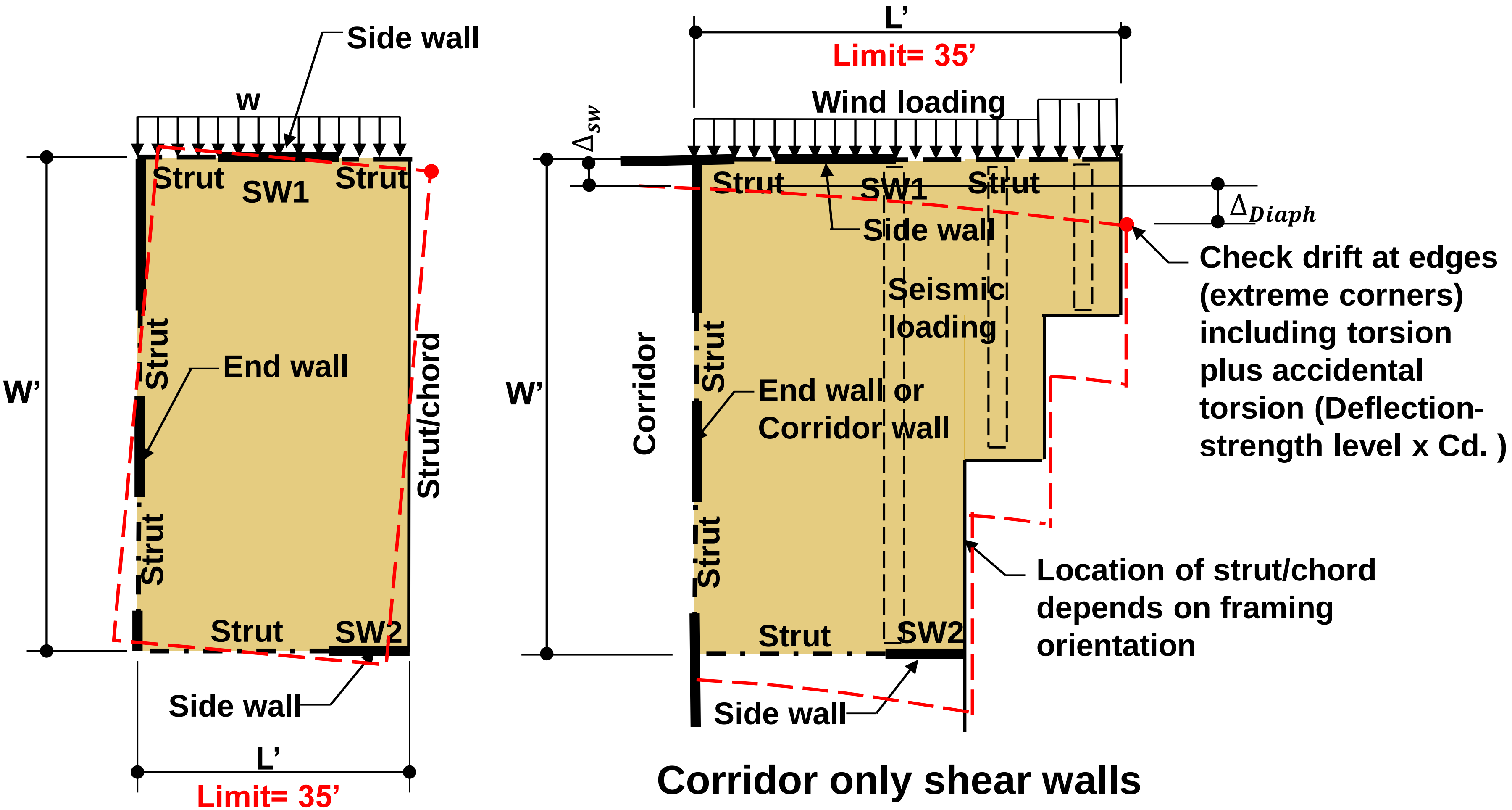
- Rigid analysis
- Semi-rigid (Envelope)



Open Front / Cantilever Diaphragm-Corridor Shear Walls Only



Example-Open Front Diaphragms



Corridor only shear walls

4.2.6.1 Framing requirements:

- Diaphragm boundary elements shall be provided to transmit design tension, compression and shear forces.
- Diaphragm Sheathing shall not be used to splice boundary elements.

Relevant 2015 SPDWS Sections-Open Front Structures

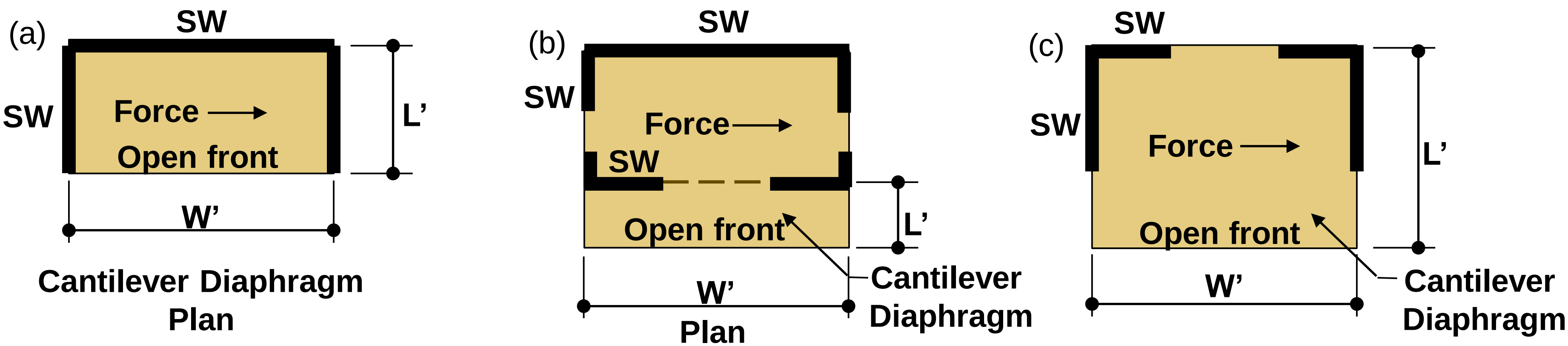


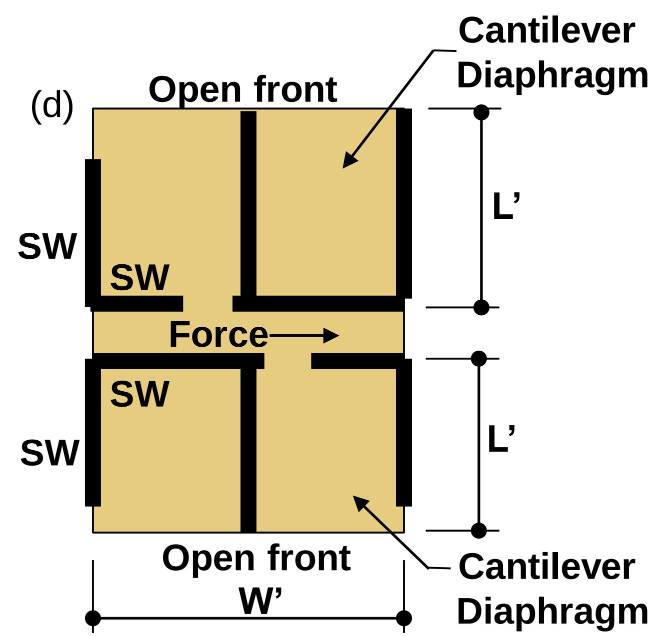
Figure 4A Examples of Open Front Structures

New definitions added:

- Open front structures
- Notation for L' and W' for cantilever Diaphragms
- Collectors

Relevant Revised sections:

- 4.2.5- Horizontal Distribution of Shears
- 4.2.5.1-Torsional Irregularity
- 4.2.5.2- Open Front Structures



4.2.5.2 Open Front Structures: (Figure 4A)

For resistance to seismic loads, wood-frame diaphragms in open front structures shall comply with all of the following requirements:

1. **The diaphragm conforms to:**
   a. **WSP-L'/W' ratio ≤ 1.5:1 4.2.7.1**
   b. **Single layer-Diag. sht. Lumber- L'/W' ratio ≤ 1:1 4.2.7.2**
   c. **Double layer-Diag. sht. Lumber- L'/W' ratio ≤ 1:1 4.2.7.3**

2. **The drift at edges shall not exceed the ASCE 7 allowable story drift ratio when subject to seismic design forces including torsion, and accidental torsion (Deflection-strength level amplified by Cd.).**

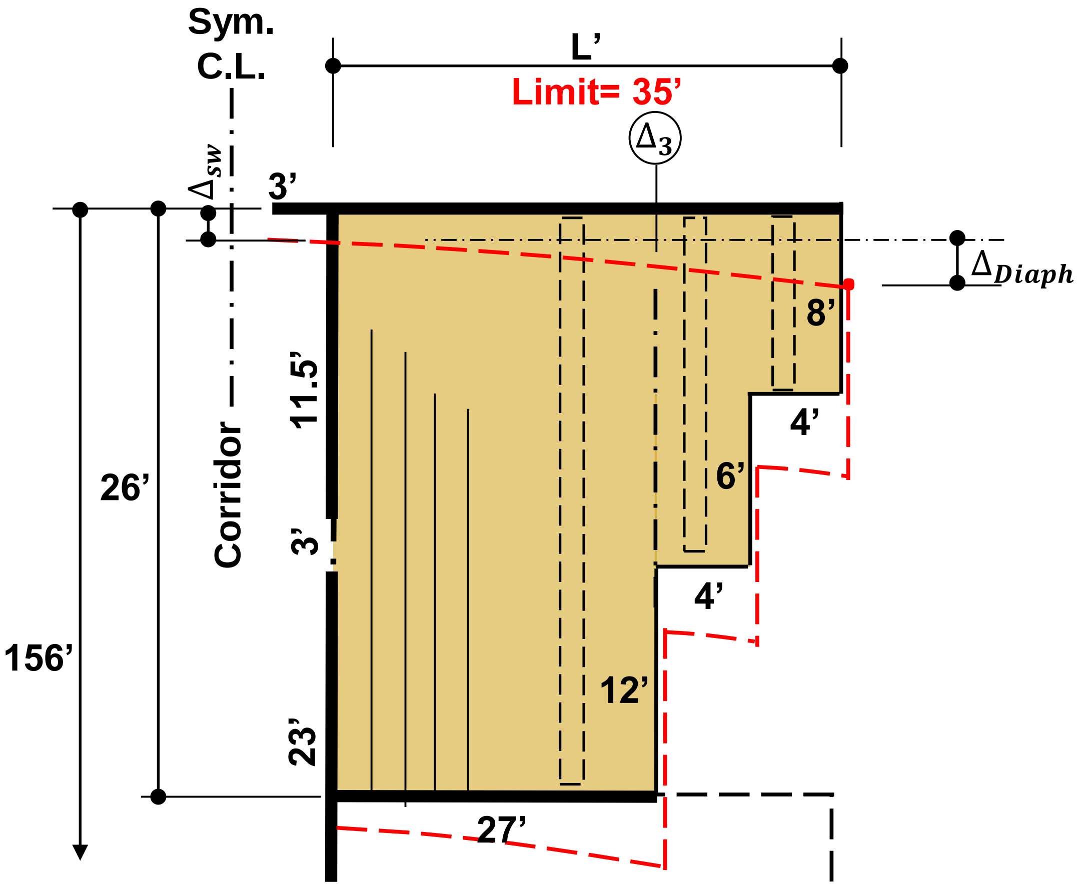
3. **For open-front-structures that are also torsionally irregular as defined in 4.2.5.1, the L'/W' ratio shall not exceed 0.67:1 for structures <u>over one</u> story in height, and 1:1 for structures <u>one</u> story in height.**

4. **For loading parallel to open side:**
   a. **Model as semi-rigid (min.), shall include shear and bending deformation of the diaphragm, or idealized as rigid.**

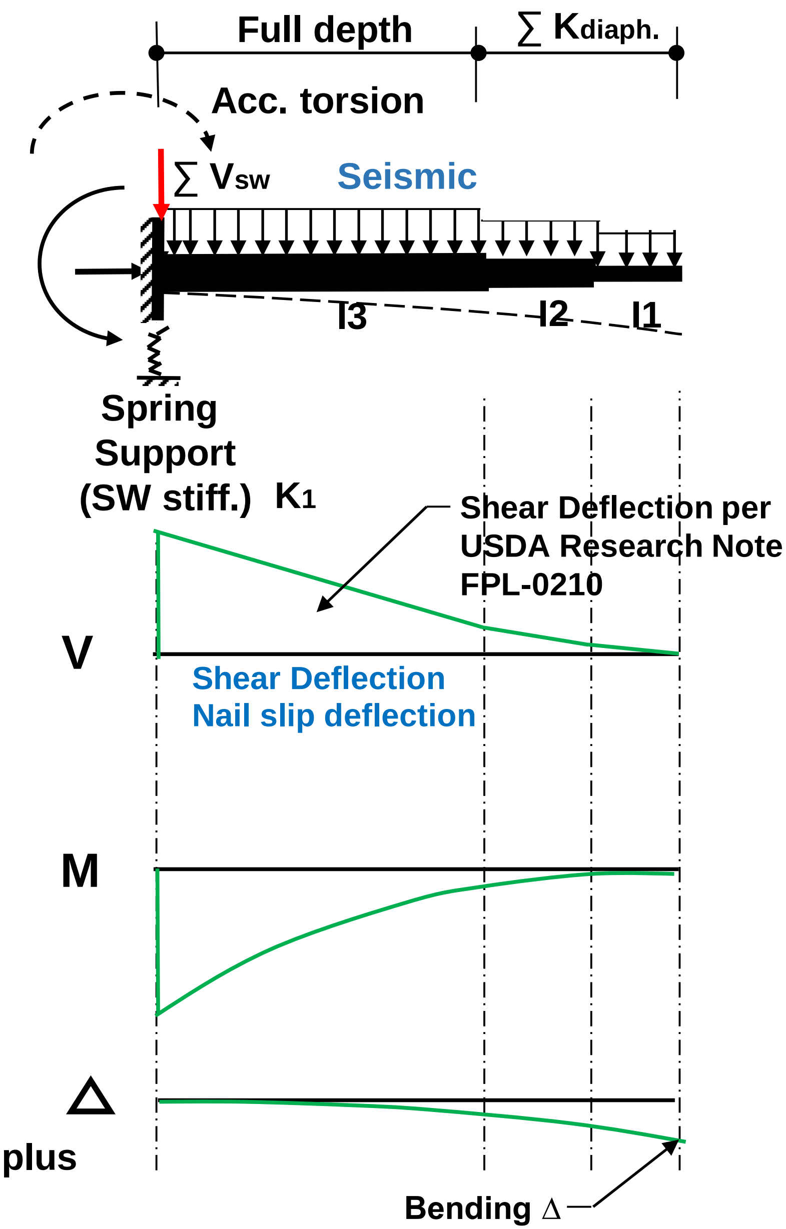
5. **The diaphragm length, L', (normal to the open side) does not exceed 35 feet.**
   <u>Exception:</u>
   Where the diaph. edge cantilevers no more than 6' beyond the nearest line of vertical elements of the LRFS.

4.2.5.2.1 For open front structures of 1 story, where L'≤25' and L'/W' ≤1:1, the diaphragm shall be permitted to be idealized as rigid for the purposes of distributing shear forces through torsion.

Determination of Open Front Diaphragm Flexibility



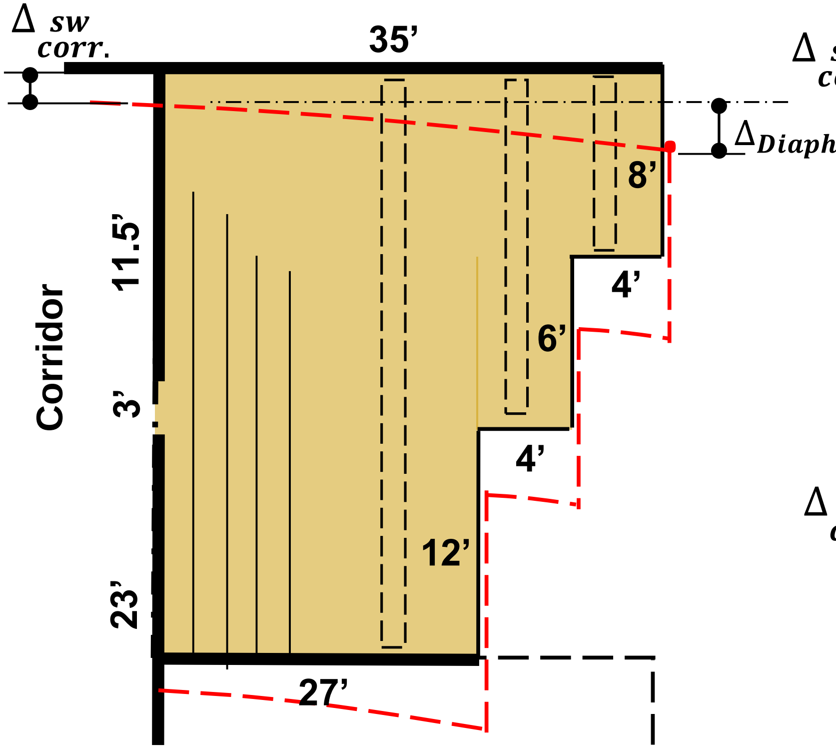
Corridor only shear walls

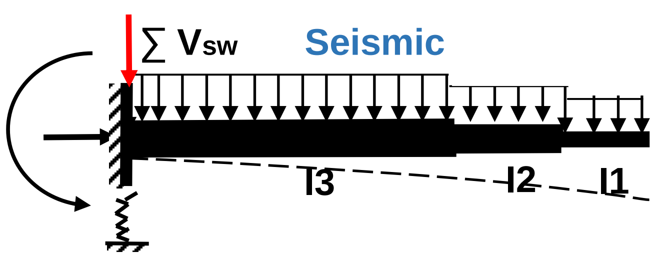


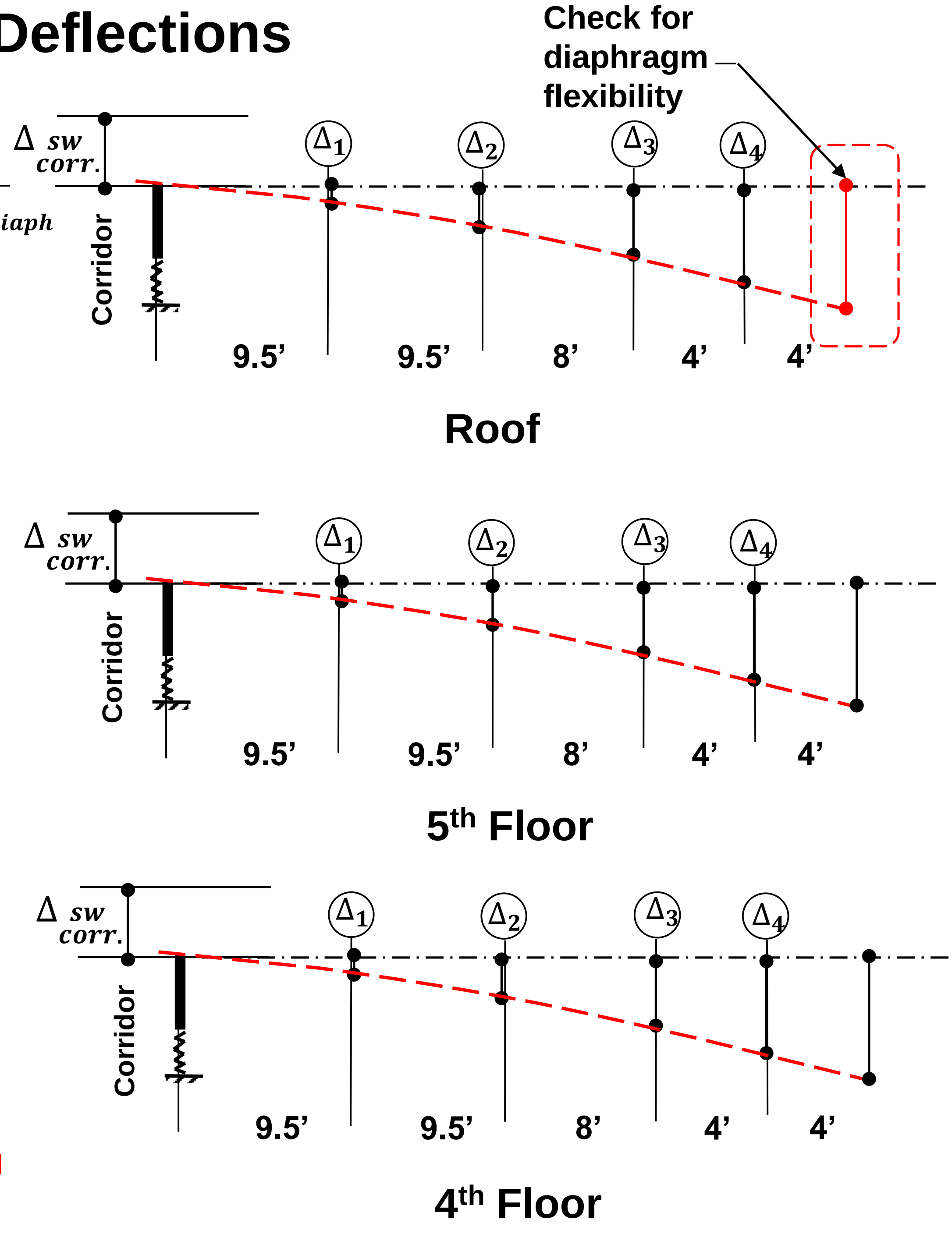
Computer Model

- Check drift at edges (extreme corners) including torsion plus accidental torsion (Deflection-strength level x Cd.)
- Seismic-meet ASCE7 drift
- Wind-deflections can be tolerated.

Final Diaphragm (Total) Deflections







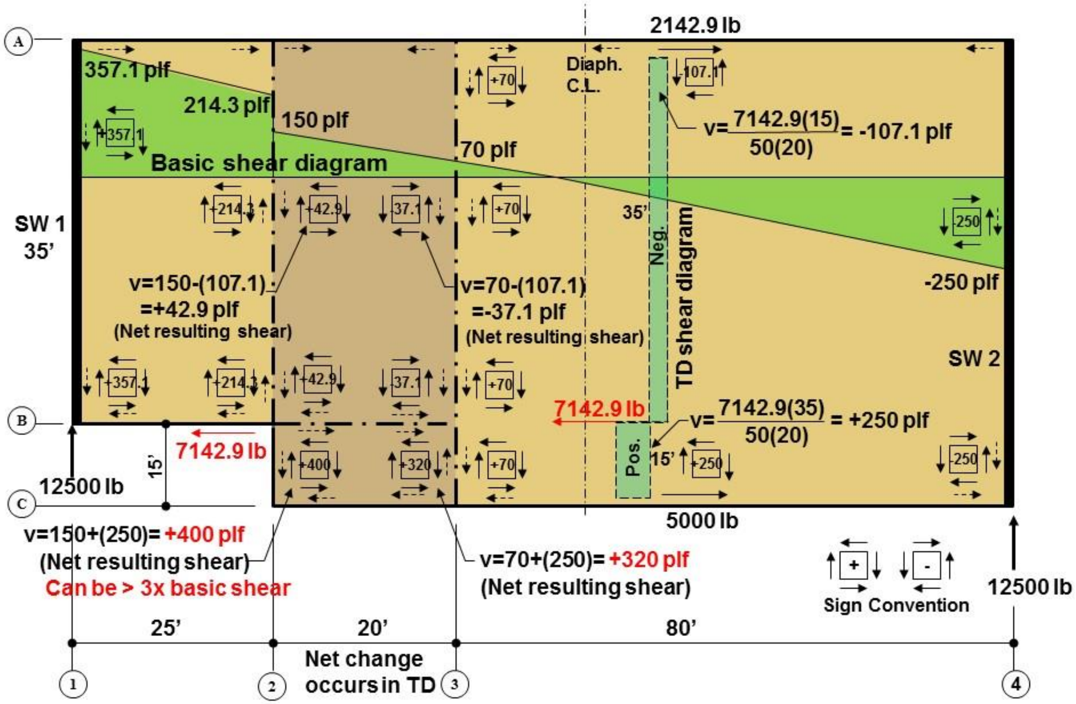
Verify Diaphragm Flexibility:

- **If flexible, add blocking**
- **If still flexible, decrease nail spacing to stiffen up**

In summary:

- **There are a number of shear wall framing systems available that can be used to resisting lateral forces.**
- **Simple calculation methods and software are often available to analyze and design most shear wall framing types.**
- **Boundary framing elements and their connections are required to maintain complete load paths to the shear walls and must be fully detailed on the drawings.**
- **Mid-rise structures require the consideration of stiffnesses of the diaphragms and shear walls to determine the horizontal distribution of shears within the diaphragm and to the walls.**

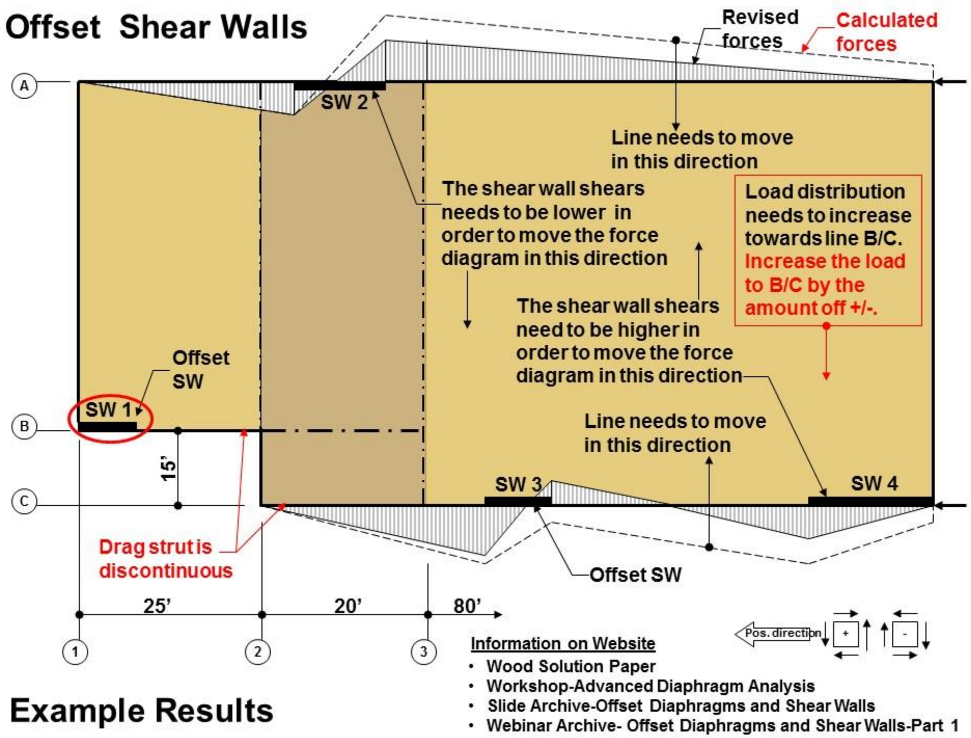
Example Offset Diaphragms



Information on Website

- Wood Solution Paper
- Slide Archive-Workshop-Advanced Diaphragm Analysis
- Slide Archive-Offset Diaphragms and Shear Walls
- Webinar Archive- Offset Diaphragms -Part 1

Example Offset Shear Walls

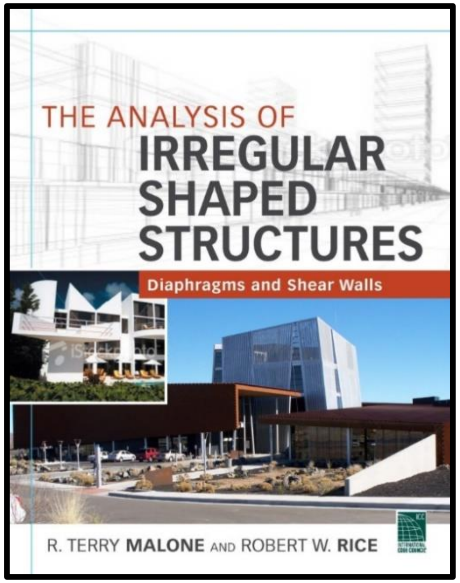


Information on Website

- Slide Archive-Workshop-Advanced Diaphragm Analysis
- Slide Archive-Offset Diaphragms and Shear Walls
- Webinar Archive- Offset Shear Walls-Part 2

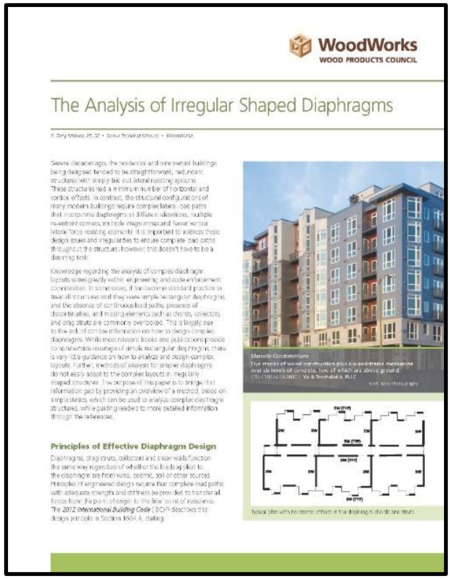
Additional Reference Materials

- The Analysis of Irregular Shaped Structures: Diaphragms and Shear Walls-Malone, Rice - McGraw-Hill, ICC



- **Woodworks Presentation Slide Archives-Workshop-Advanced Diaphragm Analysis**
- **NEHRP (NIST) Seismic Design Technical Brief No. 10-Seismic Design of Wood Light-Frame Structural Diaphragm Systems: A Guide for Practicing Engineers**
- **SEAOC Seismic Design Manual, Volume 2**
- Paper-The Analysis of Irregular Shaped Diaphragms-**Complete Example with narrative and calculations**

  http://www.woodworks.org/wp-content/uploads/Irregular-Diaphragms_Paper1.pdf



QUESTIONS?

This concludes Our Workshop Presentation on

Advanced Diaphragm Analysis

WoodWorks

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