



# A Design Example of a Wood Cantilever Diaphragm



**120 Union, San Diego, CA**  
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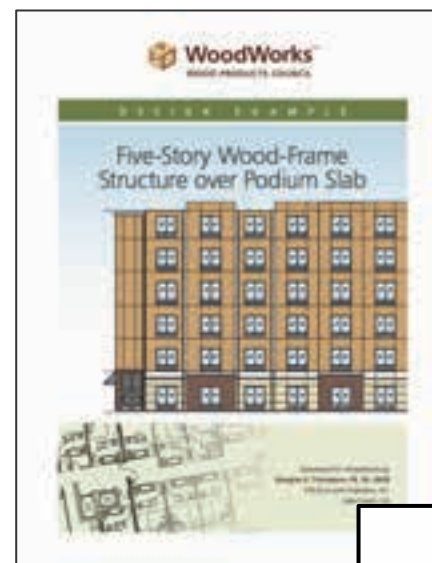
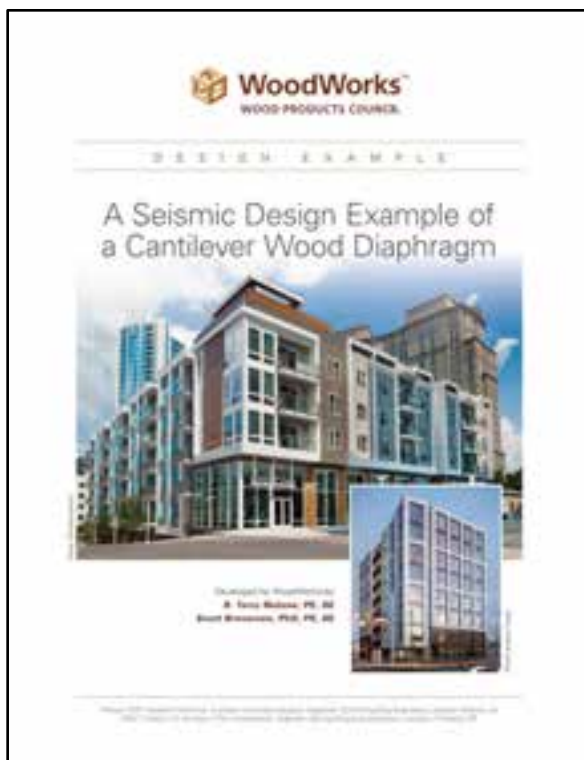
**Project Resources and Solutions Division**

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# In Your Folders



- Colored flow chart
- Wood Solutions paper: 5-Over-2 Podium Design, Parts 1 and 2.
- Wood Solutions paper: Shaft Wall Solutions for Wood-framed Buildings.
- Wood Solutions paper: Accommodating Shrinkage in Multi-story Wood-framed Structure
- The Analysis of Irregular Shaped Diaphragms

**Slides:** Education tab/Presentation Slide Archive/  
[http://www.woodworks.org/wp-content/uploads/presentation\\_slides-WW-Cantilever-Example-Workshop.pdf](http://www.woodworks.org/wp-content/uploads/presentation_slides-WW-Cantilever-Example-Workshop.pdf)

# Fasten Your Seatbelts



**5 out of 5 Calculators**



## **WoodWorks** Example and Method of Analysis:

- Currently, there are few, if any, examples or guidance available.
- No set path for design.
- Codes and standards only partially address open-front design issues.
- The method of analysis used in this example is based on our engineering judgement, experience, and interpretation of codes and standards as to how they might relate to open-front structures.

# Course Description: Open-Front Diaphragms



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

A variety of challenges often occur on projects due to:

- Fewer opportunities for shear walls at exterior wall lines
- Open-front diaphragm conditions
- Increased building heights, and
- Potential multi-story shear wall effects.
- **Can be very flexible structures subject to drift, irregularity and stiffness issues (seismic or wind).**

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

The goal of this presentation is to provide guidance on how to analyze a double open-front, or corridor only shear wall diaphragm, and help engineers better understand flexibility issues associated with these types of structures.



Codes and Standards

## Rigid Diaphragm Analysis

### Longitudinal Loading

Grid Line	K <sub>x</sub>	K <sub>y</sub>	d <sub>x</sub>	d <sub>y</sub>	k <sub>d</sub>	k <sub>d</sub> <sup>2</sup>	F <sub>x</sub>	F <sub>y</sub>	F <sub>x</sub> +F <sub>y</sub>
2	43.54		3		130.63	391.89	8874.0		-527.1
3	43.54		3		130.63	391.89	8874.0		527.1
A		25.14		20	502.74	10854.73		2028.5	-2028.5
B		25.14		20	502.74	10854.73		-2028.5	2028.5
Σ	87.09	50.27			J=	20893.23	17748		

### Transverse Loading

Grid Line	K <sub>x</sub>	K <sub>y</sub>	d <sub>x</sub>	d <sub>y</sub>	k <sub>d</sub>	k <sub>d</sub> <sup>2</sup>	F <sub>x</sub>	F <sub>y</sub>	F <sub>x</sub> +F <sub>y</sub>
2	43.54		3		130.63	391.89		277.4	277.4
3	43.54		3		130.63	391.89		-277.4	-277.4
A		25.14							
B		25.14							
Σ	87.09	50.27							

Loads	δ <sub>sw</sub>	R <sub>sw</sub>	1
	0.192	A <sub>w</sub>	1.25
	0.216		
	0.0907	F <sub>yw</sub>	17748
	-0.081	Em <sub>sw</sub>	4.75
		T=F <sub>x</sub>	84303

Use this load combination for defining No.

Expected Dead + Seismic 10-Q<sub>E</sub> (code)

Grid Line	SW	G <sub>sw</sub>	R <sub>sw</sub>	V
Calculate Stiffness of Walls on A & B using Table				
A	37	1.0		
B	37	1.0		
Calculate Stiffness of Walls on 2 & 3 using Table				
2	30	1.0		
3	30	1.0		

V equal to revised wall

### Longitudinal Analysis

#### Shear Walls LC7

Grid Line	SW	G <sub>sw</sub>	R <sub>sw</sub>	V
A & B	A,B	37	1.0	
2	2	30	1.0	
3	3	30	1.0	

#### Shear Walls LC6

Grid Line	SW	G <sub>sw</sub>	R <sub>sw</sub>	V
A & B	A,B	37	1.0	
2	2	30	1.0	
3	3	30	1.0	

### Diaphragm Deflection (in.)

Splice Forces (lbs.)			
F15	F23	F35	
1061.3	1158.3	3529.3	
4.70	5.13	15.62	
Use Nails =	8	16	24
Slip =	0.023	0.012	0.025
EA=28050000, (2)24#			
Includes effects of sw's along chord			

### Diaphragm Deflection (in.)

250.3	1930.1	3621.4
1.11	8.54	16.09
8	16	24
0.005	0.021	0.026

The analysis techniques provided are intended to demonstrate one but not the only means of analysis examples shown here are provided information for designers to construct techniques.

- The workshop is a basic. It won't always follow the
- The paper and workshop review and refinement by practicing engineers like

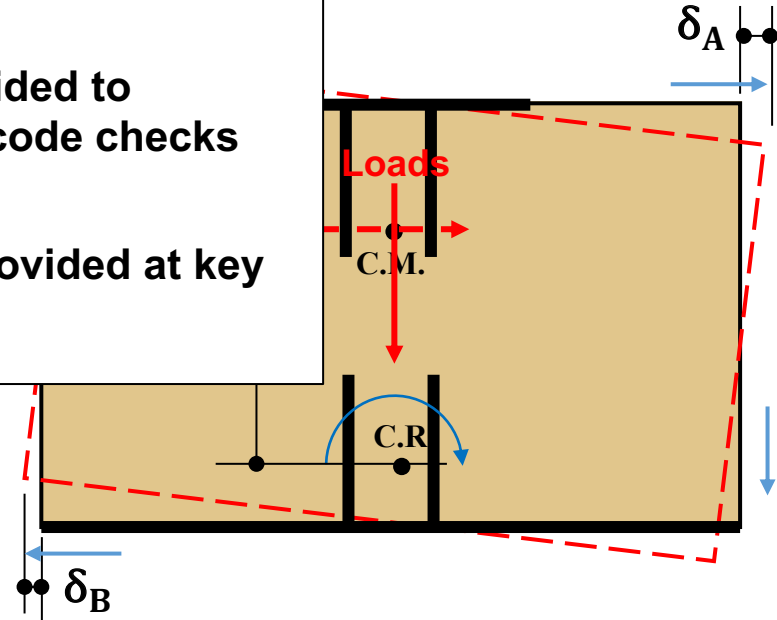
The analysis techniques provided in this presentation are intended to demonstrate one method of analysis, but not the only means of analysis. The techniques and examples shown here are provided as guidance and information for designers to consider to refine their own techniques.

- The workshop is a basic summary of the paper. It won't always follow the paper flow exactly.
- The paper and workshop are open to further review and refinement by **task groups** and practicing engineers like you.
- Only partial calculations are provided to demonstrate how certain design/code checks are performed.
- Example page numbers will be provided at key points of this presentation.

$$= \sum k d_x^2 + k d_y^2$$

$$\delta_{\text{Diaph Unif}} = \frac{3vL^3}{EAW'} + \frac{v}{Gvtv}$$

$$F_T = T \frac{\sum k d_x^2 + k d_y^2}{\sum k d_x^2 + k d_y^2}$$





# Workshop Content

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## Part 1-Background:

- Introduction
- Horizontal distribution of shear and stiffness issues
- Questions needing resolution
- 2015 SDPWS open-front requirements-review
- Introduction to open-front example
- Preliminary design assumptions

15 minute break

## Part 2-Design Example :

- Calculation of seismic forces and distribution
- Preliminary shear wall design
- Nominal shear wall stiffness
- Verification of shear wall design
- Diaphragm design

15 minute break

## Part 3-Design Example (cont.):

- Maximum diaphragm chord force
- Diaphragm flexibility
- Story drift
- Torsional irregularity

Lunch

## Part 4-Design Example (cont.):

- Amplification of accidental torsion
- Redundancy
- Transverse direction design
- Miscellaneous plan layouts and multi-story effects

## **Part 1-Background:**

- **Introduction**
- **Horizontal distribution of shear and stiffness issues**
- **Questions needing resolution**
- **2015 SDPWS open-front requirements-review**
- **Introduction to open-front example**
- **Preliminary design assumptions**

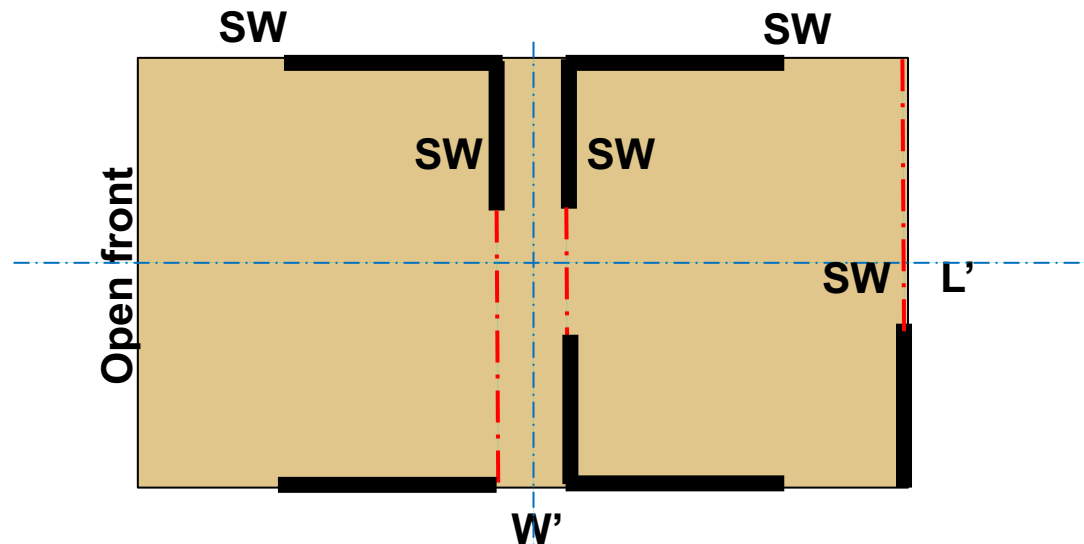
# Questions

1. When does a loss in stiffness in the exterior walls cause an open-front diaphragm condition?
2. What is the deflection equation for open-front/cantilever diaphragms?
3. How is diaphragm flexibility defined for open-front/cantilever diaphragms vs. ASCE 7-16, Figure 12.3-1?
4. What are the available methods of distributing torsional forces into the diaphragm?
5. Do shear walls located along diaphragm chord lines affect the diaphragm chord forces?
6. Will the in-plane lateral forces of the exterior walls located at the ends of the cantilever increase chord forces, or is it acceptable to include these as part of the PSF lateral load?
7. How are torsional irregularities determined and addressed for open-front/cantilever diaphragms?



# Horizontal Distribution of shear and Stiffness Issues

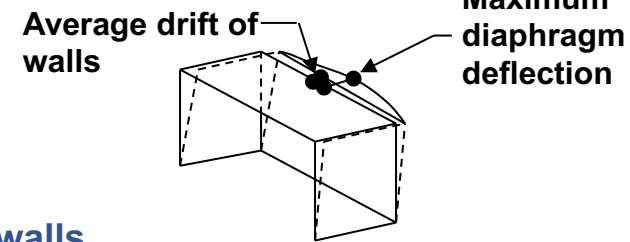
- Horizontal Distribution of shear
- Diaphragm/SW Stiffness Issues
- **Question 1:** Example-Changes in exterior wall stiffness
- 2015 SDPWS Open-front Diaphragm Requirements



# Horizontal Distribution of Shear

Distribution of shear to vertical resisting elements shall be based on an analysis where the diaphragm 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 and 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.

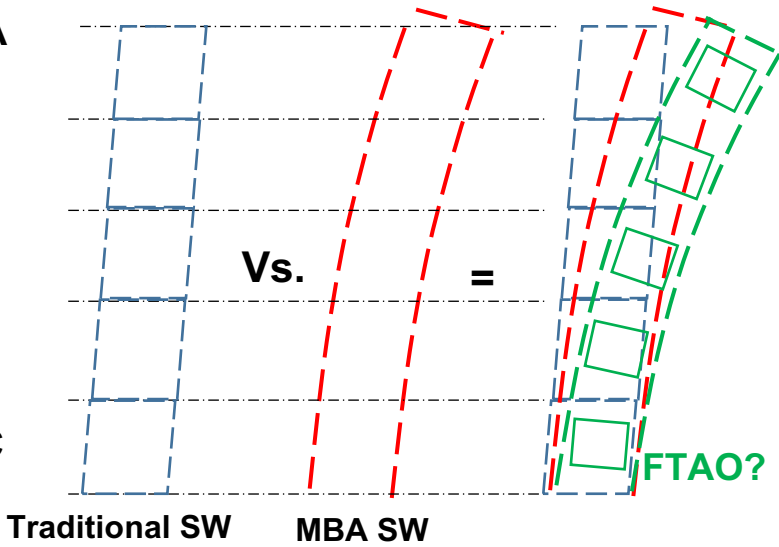
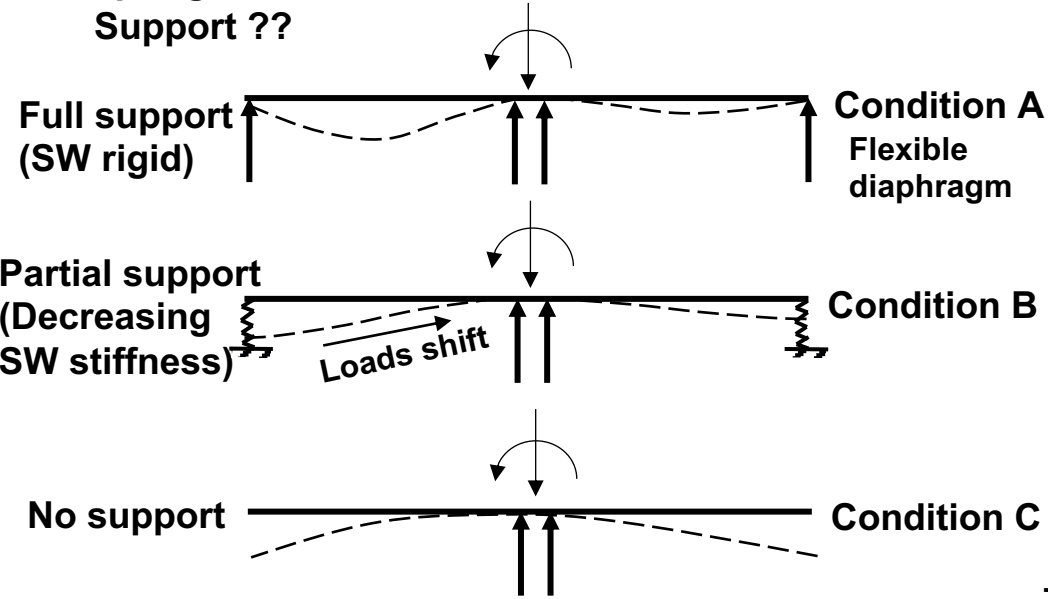
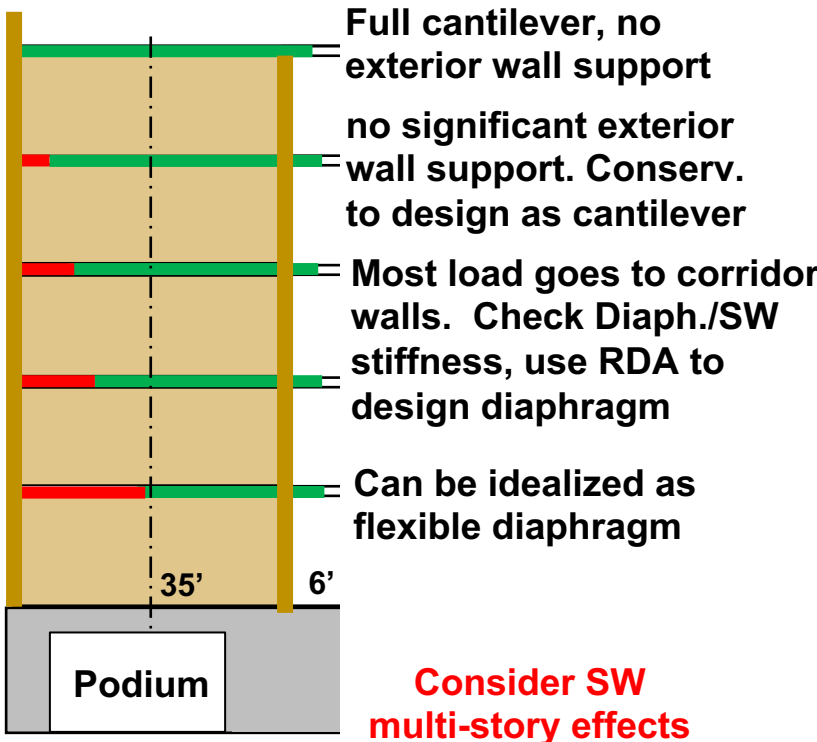
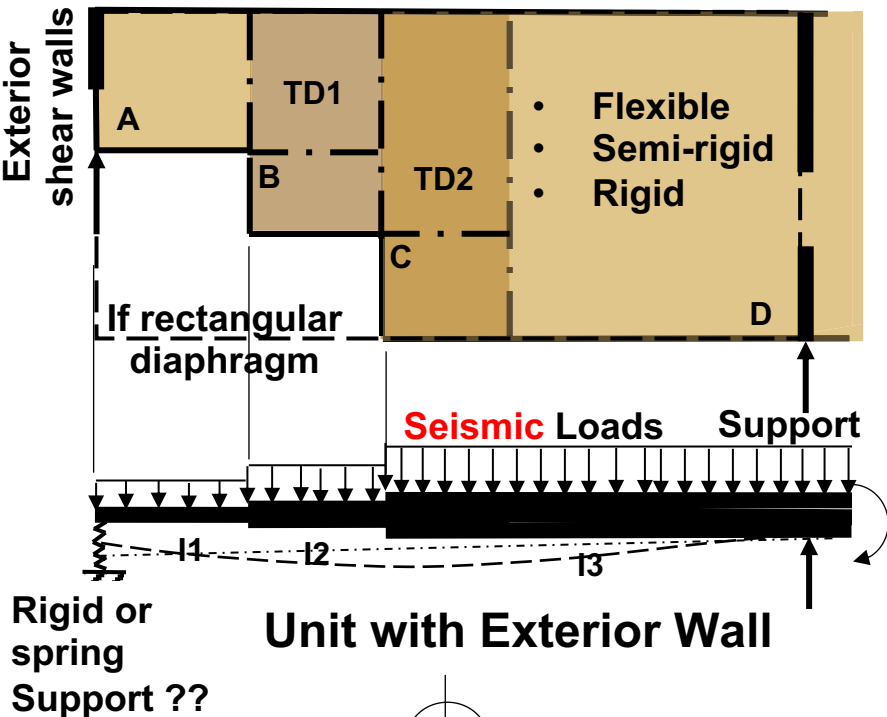


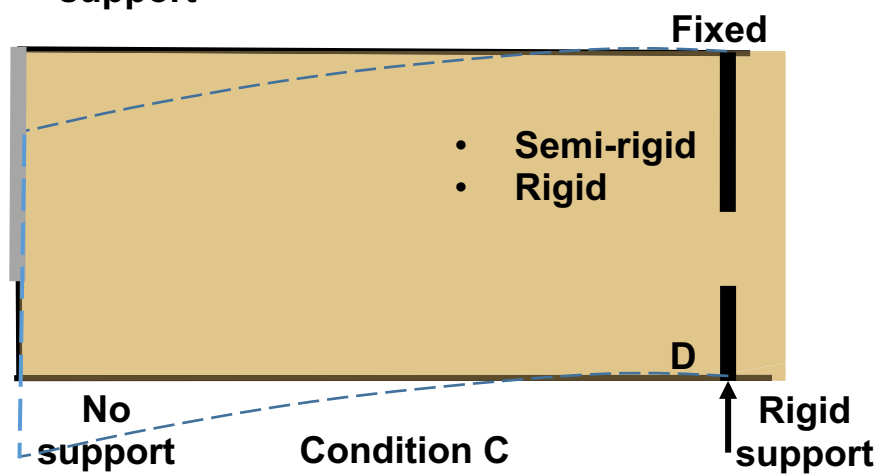
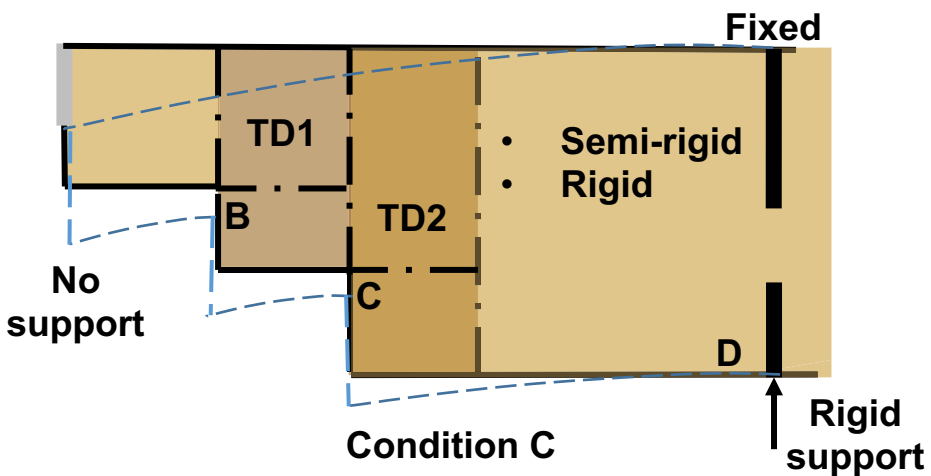
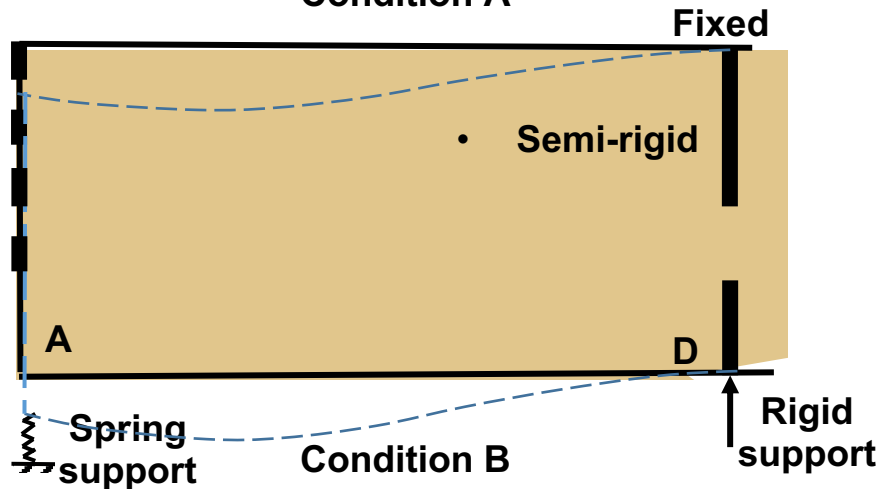
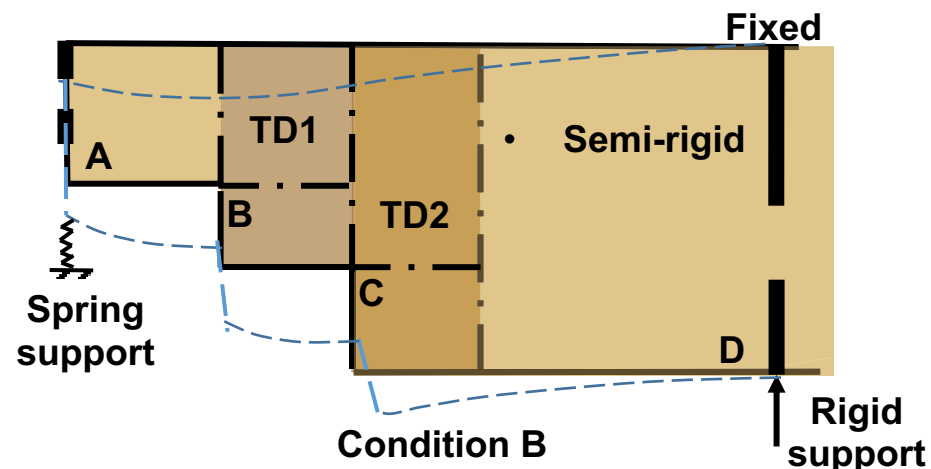
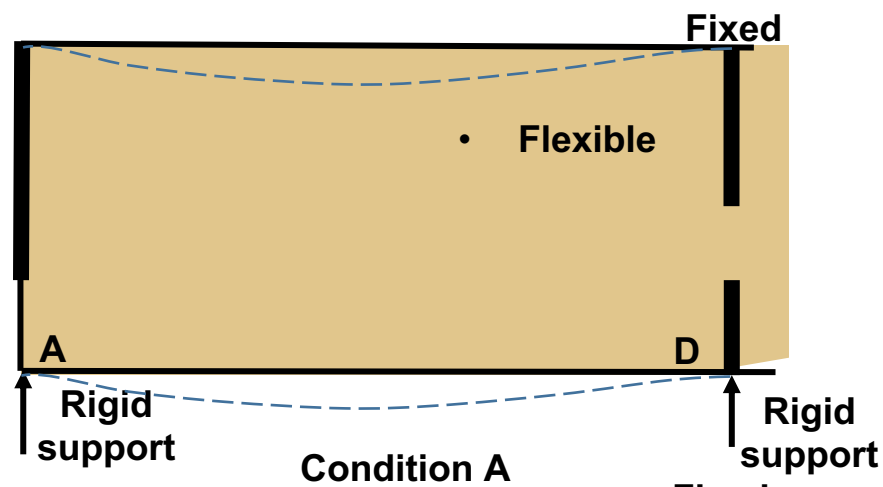
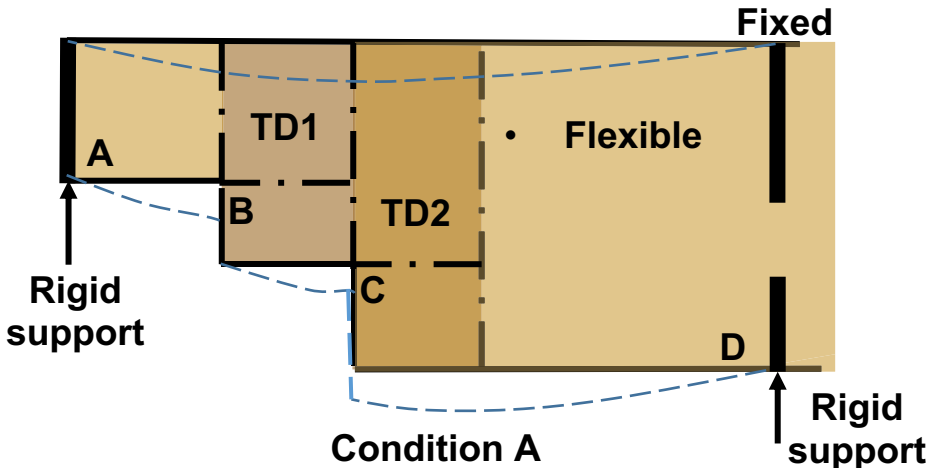
Maximum diaphragm deflection (MDD) >2x average story drift of vertical elements, using the ELF Procedure of Section 12.8?

Calculated as Flexible

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

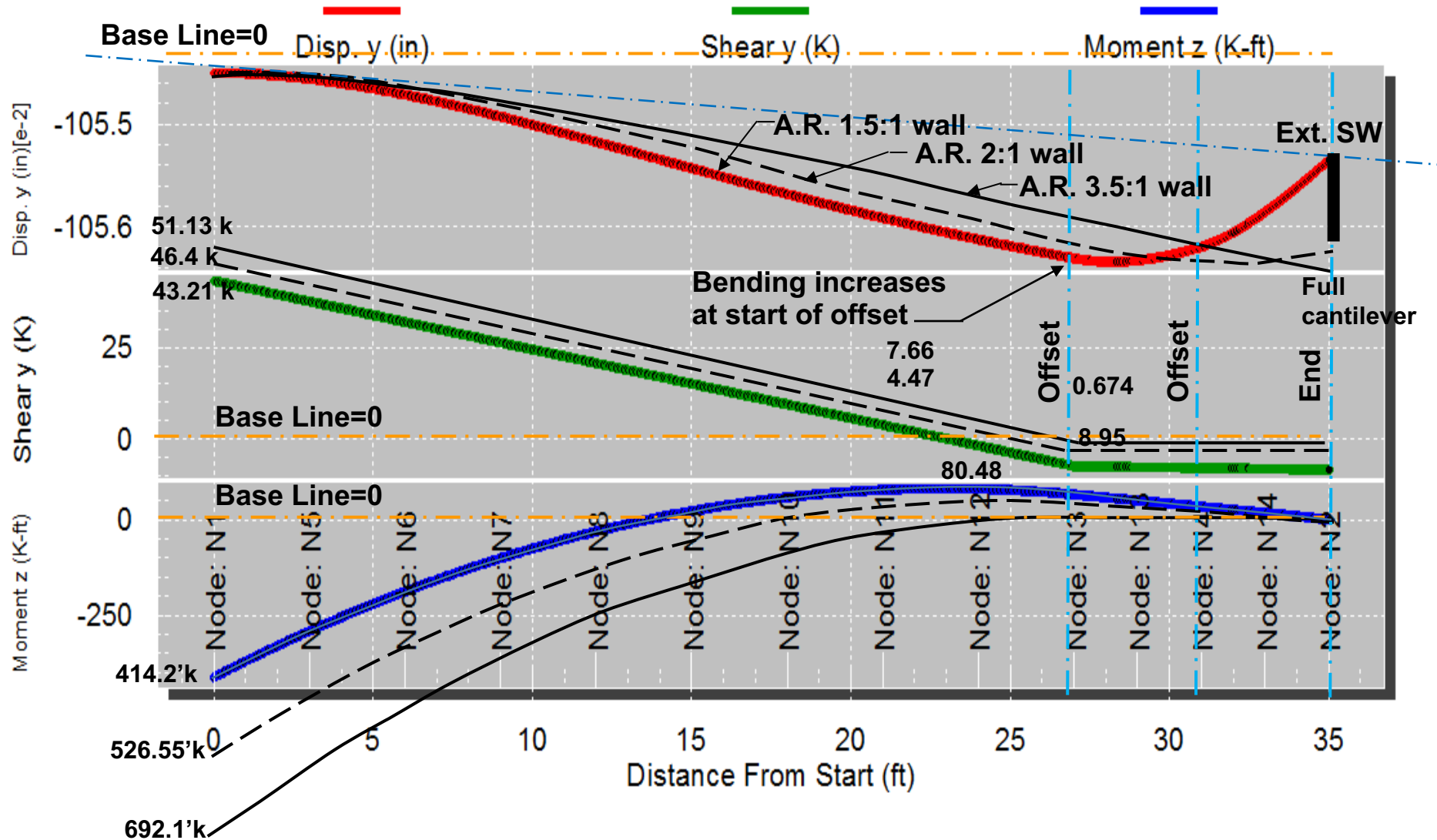
# Force Distribution Due to Diaphragm/SW stiffness





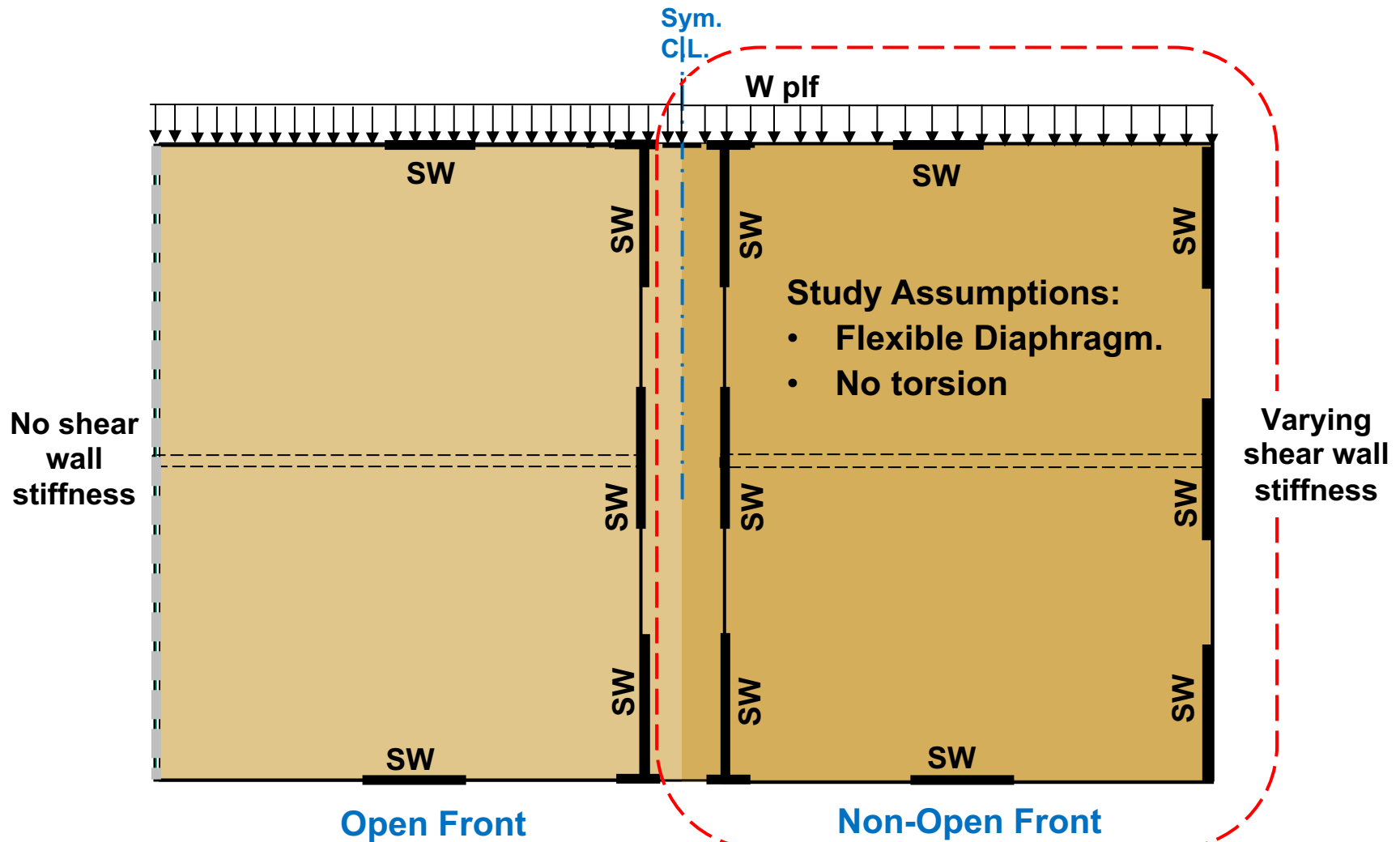
# Review Stiffness at Offsets

## Longitudinal Loads- Shear Wall A.R.=1.5:1



# Example-Exterior Wall Stiffness- Not in paper

**Question 1-**When Does a Loss in Stiffness in the Exterior Walls Cause an Open-front Diaphragm Condition? **No magic bullet answer!**



**Starting point-**Exterior shear walls same number, length, stiffness and construction as corridor walls.

# Study to Determine Open-front condition - 35' Span

Objective is to determine point where loss of shear wall stiffness at exterior wall line causes an open-front condition

- Force distribution to walls based on nominal wall stiffness
- 2D FEA model used to **visualize** diaphragm displacement curves and force distribution
- Diaphragm 15/32" WSP w/ 10d@6" o.c.
  - Modelled as flexible
  - Continuous chords at corridor walls
- Shear walls with 15/32"WSP
  - Wall height=10'
  - Hold down anchors same for all walls
  - No gravity loads
  - Corridor walls (3)10' w/ 10d@4" o.c.- constant through-out study (**basis of design**)

## 10d nails

### **L=(3)10' walls**

- 10d@3" o.c., **Ga=37**
- 10d@4" o.c., **Ga=30**
- 10d@6" o.c., **Ga=22**

### **L=(3)8' walls**

- 10d@3" o.c.
- 10d@4" o.c.
- 10d@6" o.c.

### **L=(3)6' walls**

- 10d@3" o.c.
- 10d@4" o.c.
- 10d@6" o.c.

### **L=(3)4' walls**

- 10d@3" o.c.
- 10d@4" o.c.
- 10d@6" o.c.

### **L=(3)3' walls**

- 10d@3" o.c.
- 10d@4" o.c.
- 10d@6" o.c.



# 35' RDA Force Distribution-SW displ.

- Diaphragm stiffness flexible
- Shear wall stiffness-variable
- Seismic STR. Forces
- No torsion
- No gravity loads

- $V$ =Shear to wall line
- $k$ =Stiffness of wall line
- $\%$ =SW stiffness at exterior wall vs. corridor wall line

Fixed support

If flexible, trib. Reaction force  $R=3810$  lbs.

Open-front effect

$V=3.81k$ ,  $k=40$   
 $V=4.15k$ ,  $k=40.71$   
 $V=3.81k$ ,  $k=40$ ,  $\%=100$   
 $V=3.45k$ ,  $k=33.86$ ,  $\%=85$

(3) 10' ext. walls  
 $\Sigma L_{sw}=30'$ , A.R.=1:1

$V=4.35k$ ,  $k=41.06$   
 $V=4.53k$ ,  $k=41.36$   
 $V=4.82k$ ,  $k=41.8$

Forces shifting

$V=3.25k$ ,  $k=30.66$ ,  $\%=77$   
 $V=3.07k$ ,  $k=28.05$ ,  $\%=70$   
 $V=2.78k$ ,  $k=24.08$ ,  $\%=60$

(3) 8' ext. walls  
 $\Sigma L_{sw}=24'$   
A.R.=1.25:1

$V=5.3k$ ,  $k=42.43$   
 $V=5.42k$ ,  $k=42.58$   
 $V=5.63k$ ,  $k=42.81$

Forces shifting

$V=2.31k$ ,  $k=18.42$ ,  $\%=46$   
 $V=2.18k$ ,  $k=17.07$ ,  $\%=43$   
 $V=1.97k$ ,  $k=14.96$ ,  $\%=37$

(3) 6' ext. walls  
 $\Sigma L_{sw}=18'$   
A.R.=1.67:1

$V=6.39k$ ,  $k=43.56$   
 $V=6.45k$ ,  $k=43.61$   
 $V=6.55k$ ,  $k=43.7$

$V=1.21k$ ,  $k=8.2$ ,  $\%=21$   
 $V=1.15k$ ,  $k=7.74$ ,  $\%=19$   
 $V=1.05k$ ,  $k=6.98$ ,  $\%=17$

(3) 4' ext. walls  
 $\Sigma L_{sw}=12'$   
A.R.=2.5:1

All open-front Diaph.

10d nails

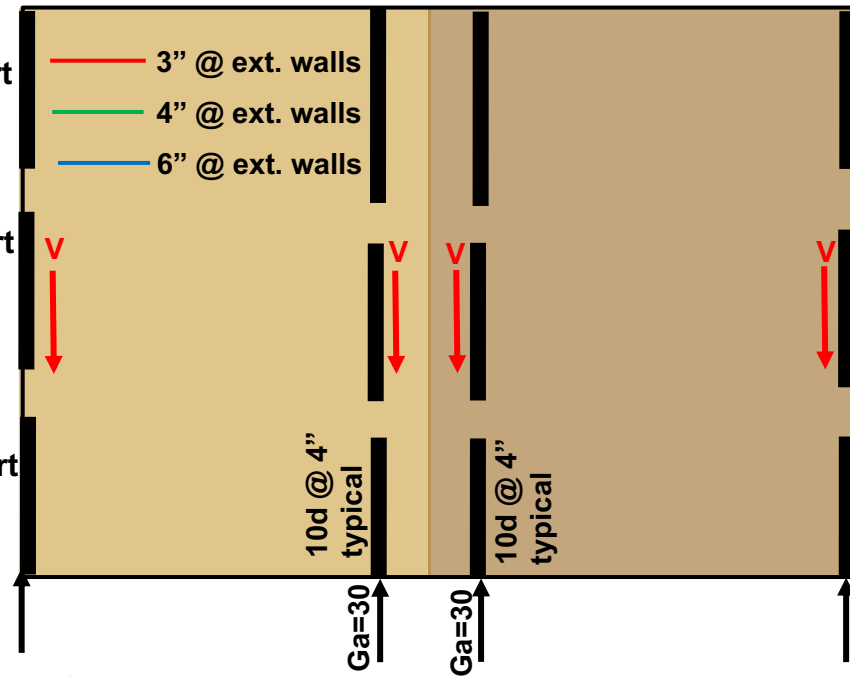
Rigid support

Partial support

All partial support

No support

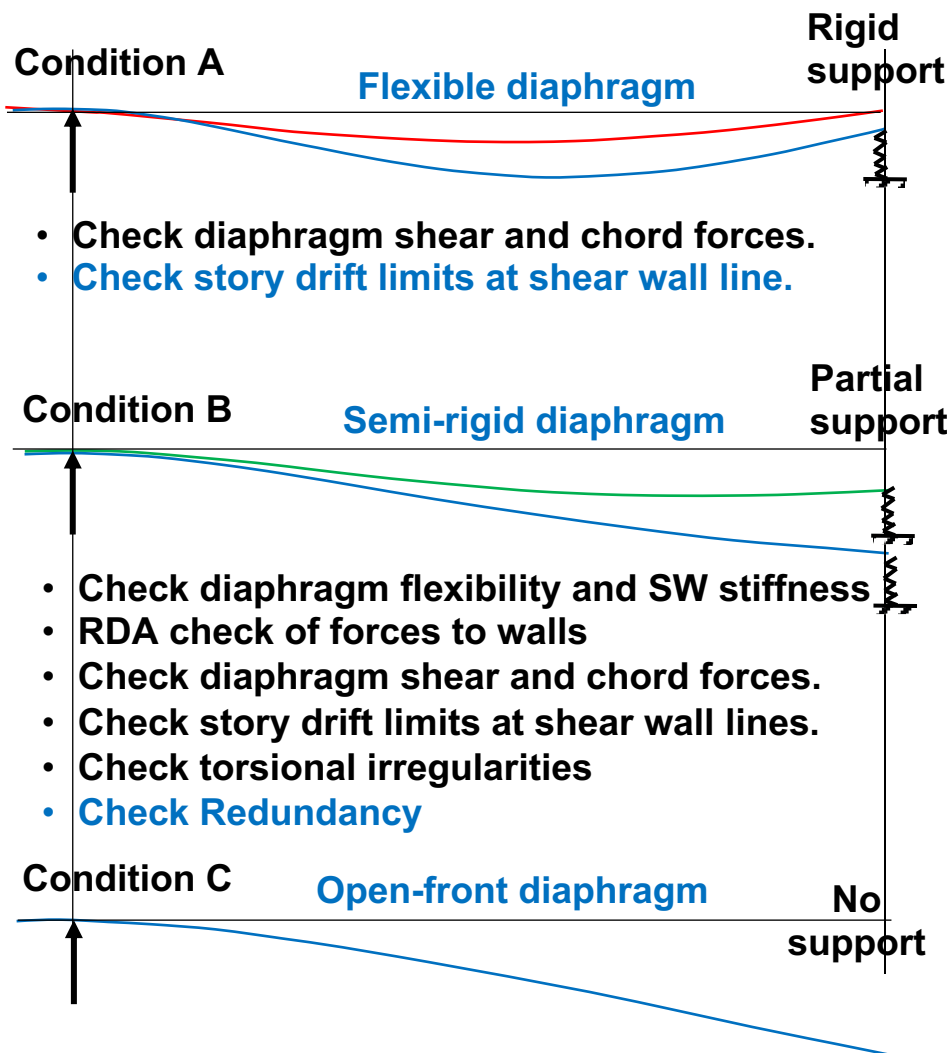
Exterior Walls



Prelim conclusion (This example only):

- If walls near 44% or if  $k \leq 20$  consider open-front
- Magic 20' SW

Corridor Walls



Can happen when loss of wall support occurs, diaphragm flexibility changes, or story drift cannot be met

Flexible diaphragm

### Transition Stage

There comes a point when: SW's don't significantly contribute to lateral resistance, provide economical solutions, or become less constructible

**Areas of partial support-Requires engineering judgement**

Conservative to design as open-front.

### Open-front condition **SDPWS Section 4.2.5.2**

- Check diaphragm flexibility
- Check shear wall deflection, stiffness
- RDA check of forces to walls
- Check diaphragm shear and chord forces.
- Check story drift limits at edges
- Check torsional irregularities
- Check redundancy
- Check amplification of accidental torsion

## Minimum Design Check Considerations

(You make the judgement call)

# Structures Are Also Susceptible to Wind Damage



- Too much flexibility?
- Lack of adequate shear walls
- Soft / Weak story issues?
- Insufficient load paths?
- Lack of proper connections?



**Possible Soft Story**



**Possible Soft Story**  
(Not enough shear walls across front)





**Possible Soft Story**



**An Engineered Structure?**



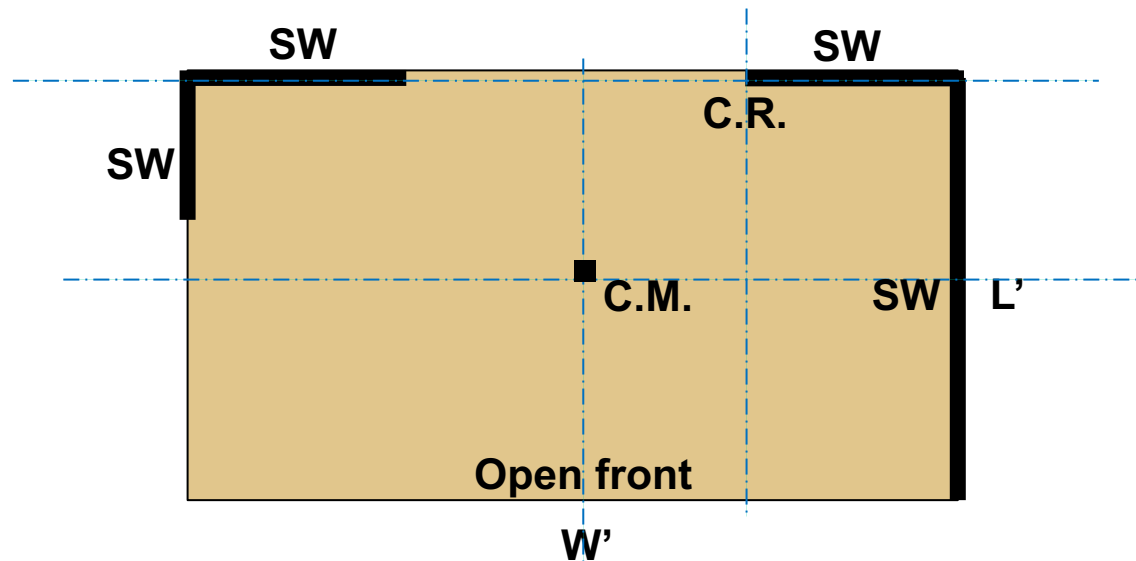
No shear walls

**Possible Soft Story**

# 2015 SDPWS Open-front Diaphragm Requirements

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## Open-Front Diaphragms



# Relevant 2015 SDPWS Sections

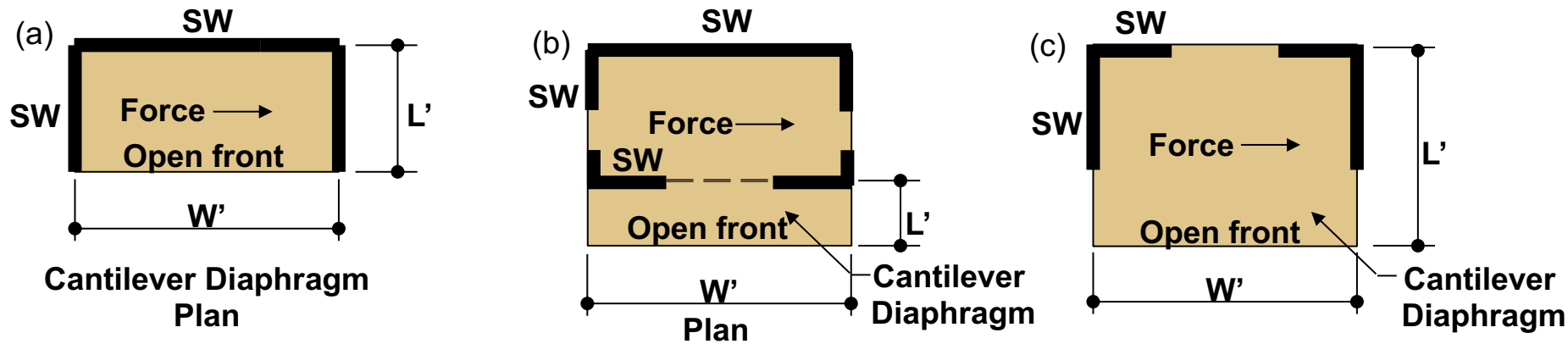


Figure 4A Examples of Open Front Structures

## 4.2.5.2 Open Front Structures:

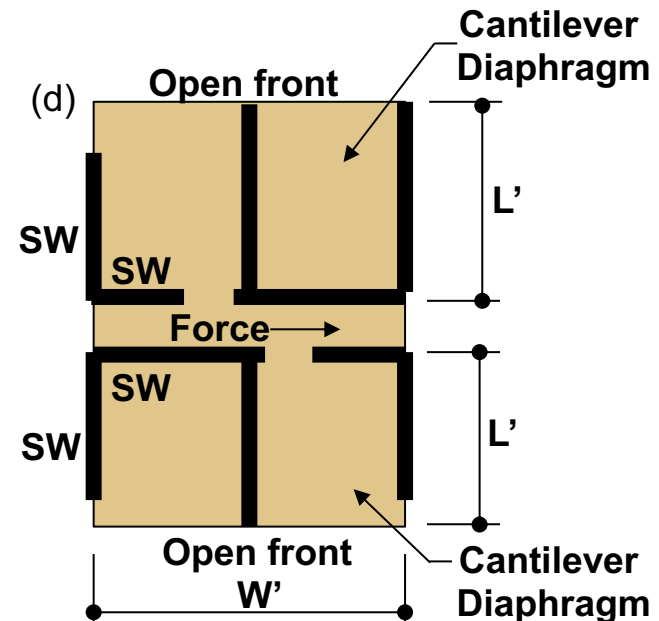


### New definitions added:

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

### Relevant Revised sections:

- 4.2.5- Horizontal Distribution of Shears
- 4.2.5.1-Torsional Irregularity
- 4.2.5.2- Open Front Structures
- Combined open-front and cantilever diaphragms



Similar to MS-MF structures

## SDPWS 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:

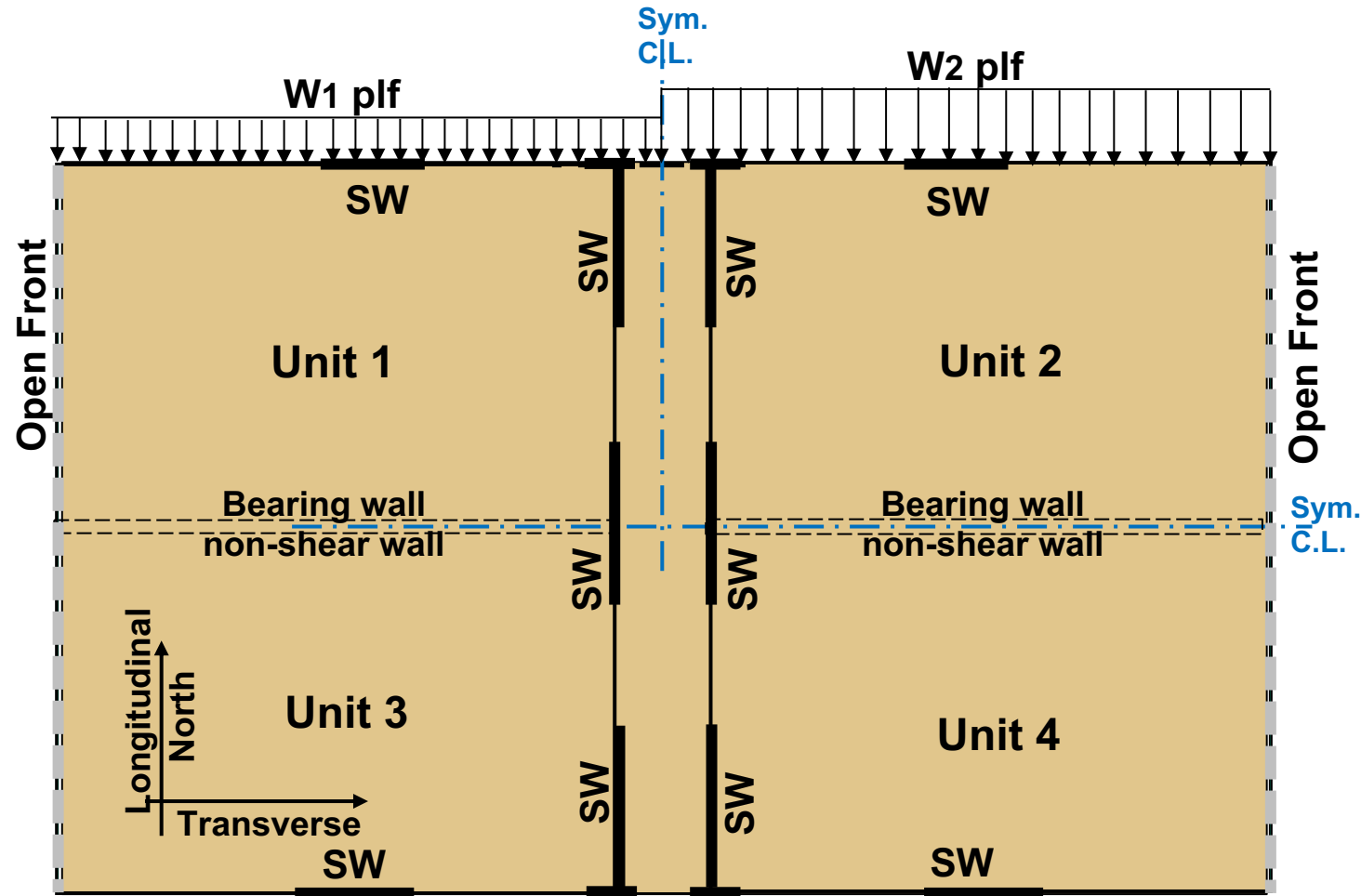
1. The diaphragm conforms to:
  - a. WSP-L'/W' ratio  $\leq$  **1.5:1** 4.2.7.1
  - b. Single layer-Diag. sht. Lumber- L'/W' ratio  $\leq$  **1:1** 4.2.7.2
  - c. Double layer-Diag. sht. Lumber- L'/W' ratio  $\leq$  **1:1** 4.2.7.3
2. The drift at edges shall not exceed the ASCE 7 allowable story drift 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 over one story in height, and **1:1** for structures one 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. (2008 SDPWS: L'<sub>max</sub>=25'. Exception-if drift can be tolerated, L' can be increased by 50%). Could use an Alternative Materials, design and Methods Request (AMMR) to exceed 35'.

**Currently no deflection equations or guidance on determination of diaphragm flexibility.**



# Design Example- Longitudinal Direction

Example plan selected to provide maximum information on design issues



## Disclaimer:

The following information is an open-front diaphragm example which 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 the only method of analysis available.

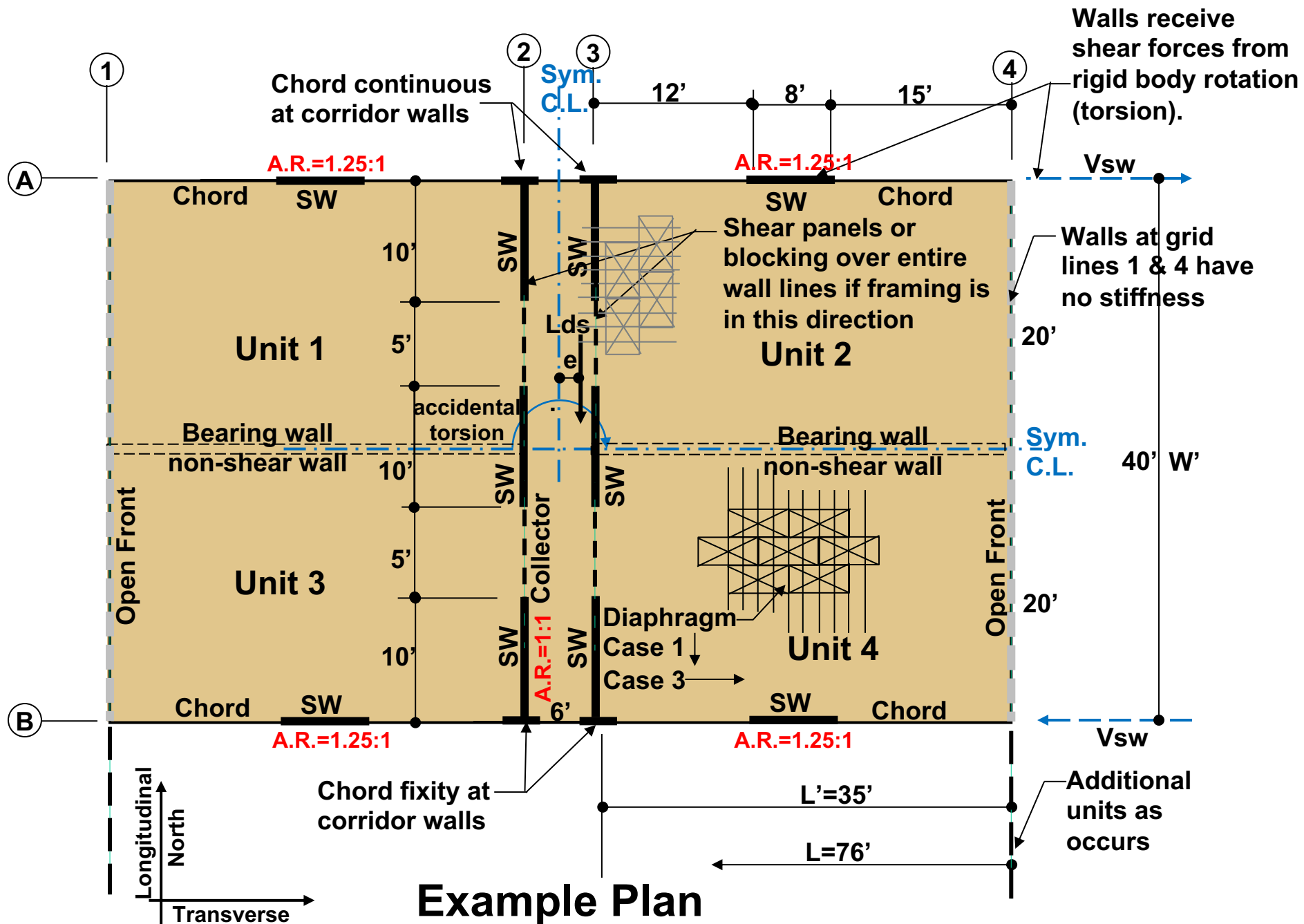
# Open Front Structures Code Checks:

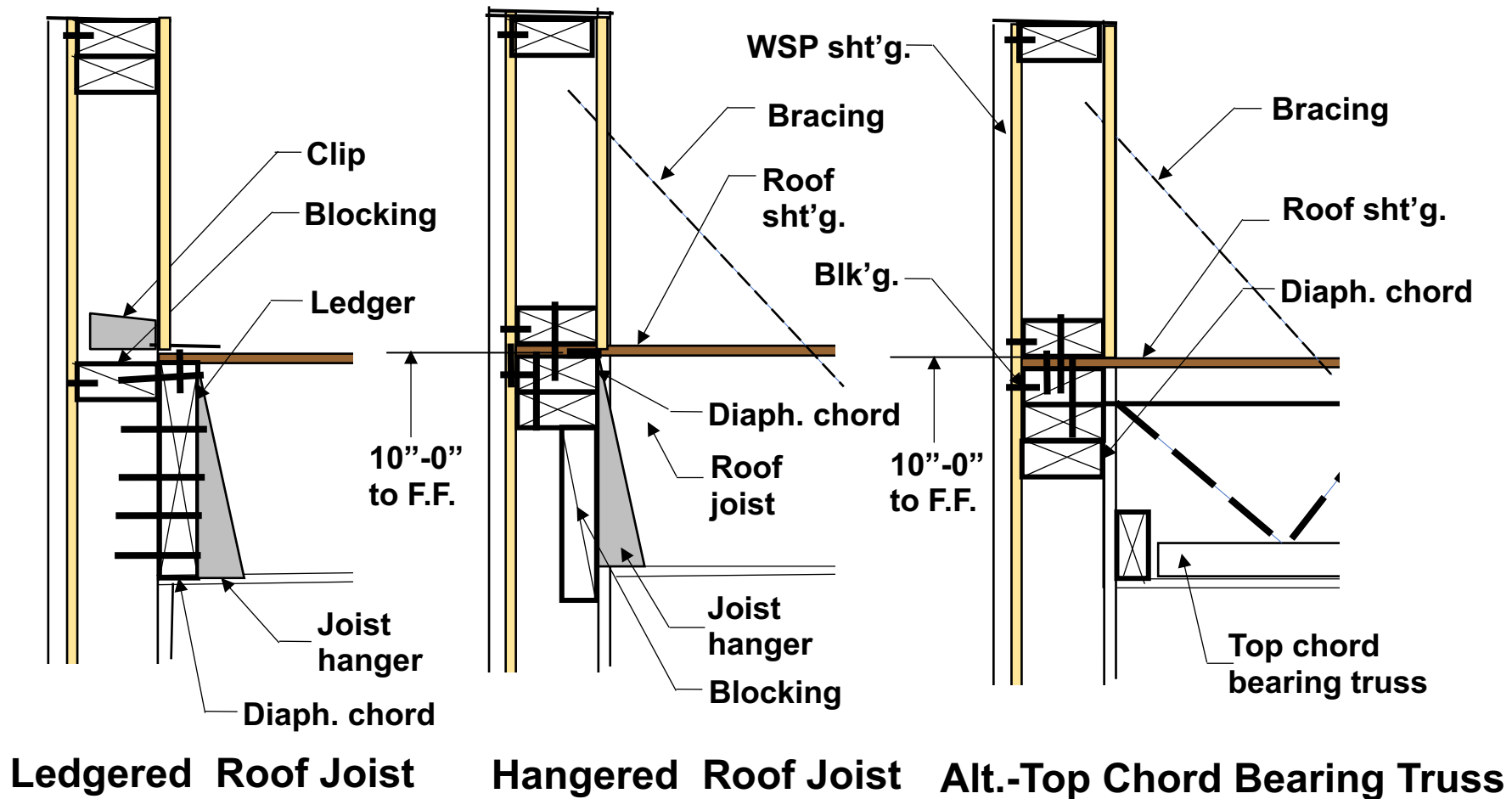
For resistance to **seismic** loads, wood-frame diaphragms in open front structures **should** comply with **all** of the following requirements:

- |   |                                  |
|---|----------------------------------|
| 1. Check stiffness of diaphragm and shear walls | ASCE 7 12.3.1, SDPWS 4.2.5.2 (3) |
| 2. Verify aspect ratio                          | SDPWS 4.2.7.1- 4.2.7.3           |
| 3. Check drift at <u>edges</u>                  | ASCE 7 12.12.1, SDPWS 4.2.5.1    |
| 4. Check for torsional irregularity             | ASCE 7 12.3.2, SDPWS 4.2.5.1     |
| • Inherent torsion                              | ASCE 7 12.8.4.1                  |
| • Accidental torsion                            | ASCE 7 12.8.4.2                  |
| • Amplification of accidental torsion           | ASCE 7 12.8.4.3                  |
| 5. Check diaphragm flexibility                  | ASCE 7 12.3, SDPWS 4.2.5.2 (3)   |
| 6. Verify diaphragm length, $L'$                | SDPWS 4.2.5.2(4)                 |
| 7. Assume or verify redundancy                  | ASCE 7 12.3.4                    |

For resistance to **Wind** loads:

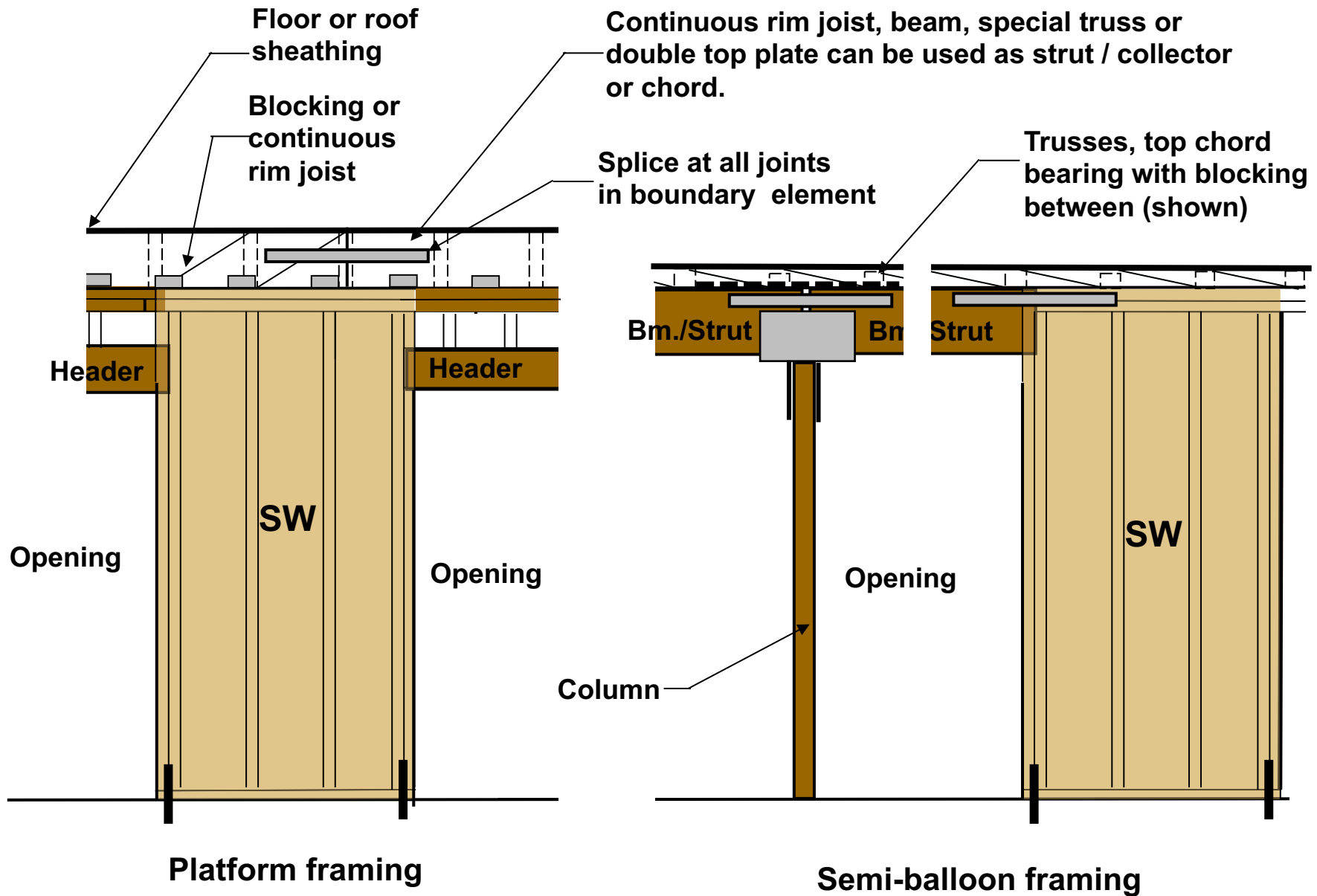
1. **ASCE 7-16 Section 27.4.5-Diaphragm flexibility-The structural analysis shall consider the stiffness of diaphragms and vertical elements of the MWFRS**
2. Recommend Following SDPWS 4.2.5.2 **(not required by code)**. Considered good engineering practice.
3. Show that the resulting drift at the edges of the structure can be tolerated.



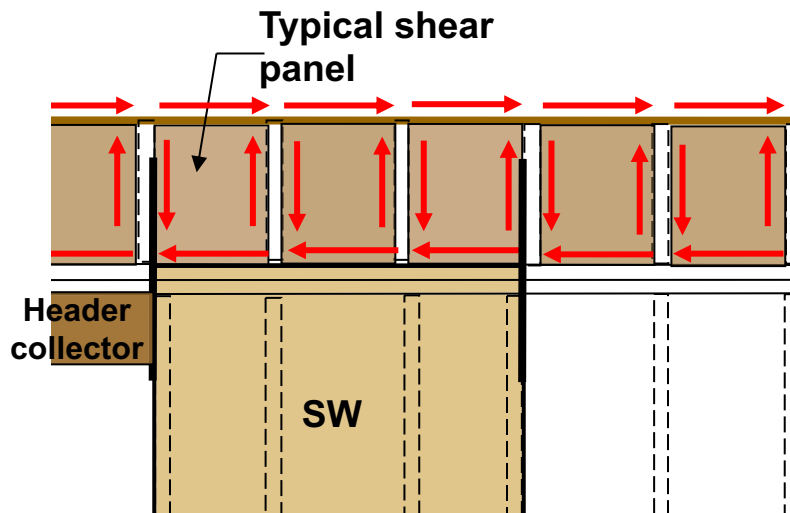


(Platform framing not shown)

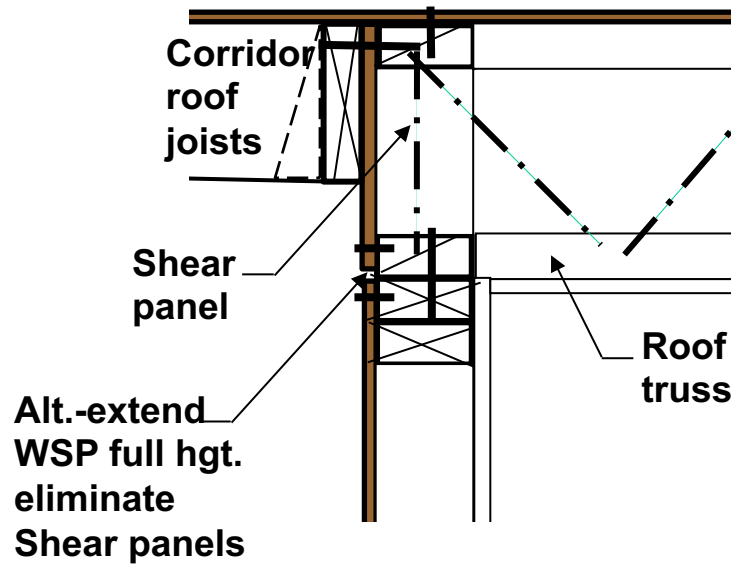
## Typical Exterior Wall Sections



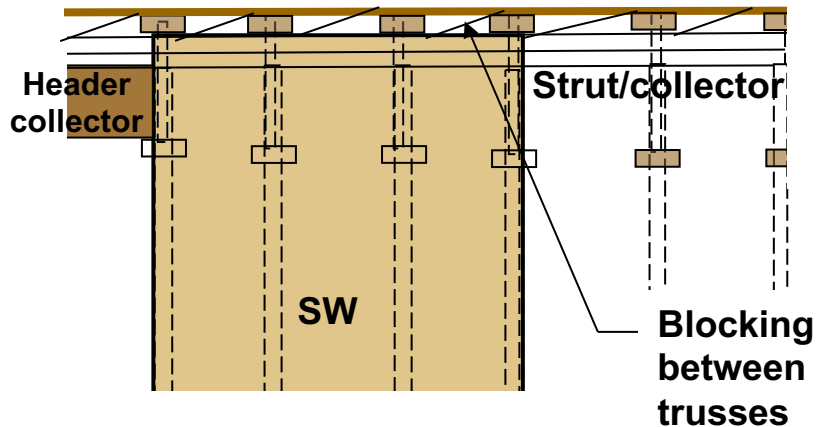
**Typical Exterior Wall Sections at Grid Lines A and B**



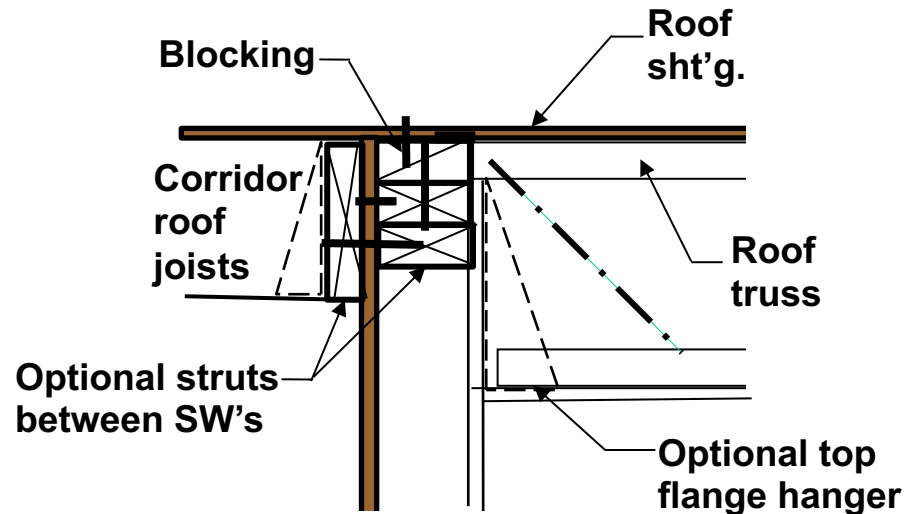
**Platform Framing at Corridor**



**Section at Corridor**



**Semi-balloon Framing at Corridor**



**Section at Corridor  
(Similar to example)**

## Typical Wall Sections at Corridor Walls

# Preliminary Assumptions

1. LFRS Layout -efficient / marginal / **scary**
2. Diaphragm Flexibility
3. Redundancy
4. Accidental torsion
5. Torsional Irregularities

## Options: Pros and Cons of Assumptions

- Assume conservative values upfront:
  1. Design is conservative, leave as is
  2. Design is conservative, revise to reduce forces
- Assume minimum values upfront:
  1. Design meets demand, leave as is
  2. Design meets demand but is marginal, change to improve performance
  3. Design unconservative, revise design to meet demand



## **2. Diaphragm Flexibility-12.3.1**

NEHRP Seismic Design Brief 10 and ASCE 7-16 commentary-"The diaphragms in **most** buildings braced by wood light-frame shear walls are **semi-rigid**".

- The diaphragm stiffness relative to the stiffness of the supporting vertical seismic force-resisting system is **important to define**.

ASCE 7, C12.3.1.1 Flexible Diaphragm Condition is allowed provided:

- All light framed construction
- 1 ½" or less of non-structural concrete topping
- Each line of LFRS is less than or equal to allowable story drift

Compliance with story drift limits along each line of shearwalls is intended as an **indicator** that the shearwalls are substantial enough to share load on a tributary area basis and do not require torsional force redistribution.

### **3. Redundancy**

Assume  $\rho=1.3$  unless conditions of ASCE 7-16 Section 12.3.4.2 are met to justify  $\rho=1.0$ .

### **4. Accidental Torsion 12.8.4.2**

Accidental torsion shall be applied to all structures for determination if a horizontal irregularity exists as specified in Table 12.3-1.

- Applies to non-flexible diaphragms
- Design shall include the inherent torsional moment ( $M_t$ ) plus the accidental torsional moments ( $M_{ta}$ )
- Accidental torsional moment ( $M_{ta}$ ) = assumed displacement of the C.M. equal to 5% of the dimension of the structure perpendicular to the direction of the applied forces.

## 5. Accidental Torsion 12.8.4.2 (Cont.)

Accidental torsion moments ( $M_{ta}$ ) **need not be included** when determining:

- Seismic forces  $E$  in the design of the structure, or
- Determination of the design story drift in Sections 12.8.6, 12.9.1.2, Chapter 16, or drift limits of Section 12.12.1.

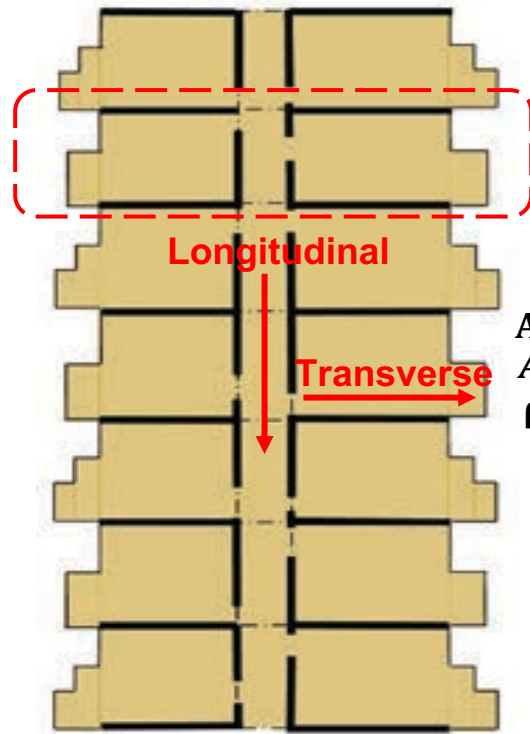
### Exceptions:

- Structures assigned to Seismic Category B with Type 1b horizontal structural irregularity.
- Structures assigned to Seismic Category C, D, E, and F with Type 1a or Type 1b horizontal structural irregularity.

Structures assigned to SDC **C, D, E, or F**, where **Type 1a or 1b** torsional irregularity shall have the effects accounted for by **multiplying  $M_{ta}$  at each level by a torsional amplification factor ( $A_x$ )**

For our example, C.M = C.R. No inherent torsion. Only accidental torsion is applied.

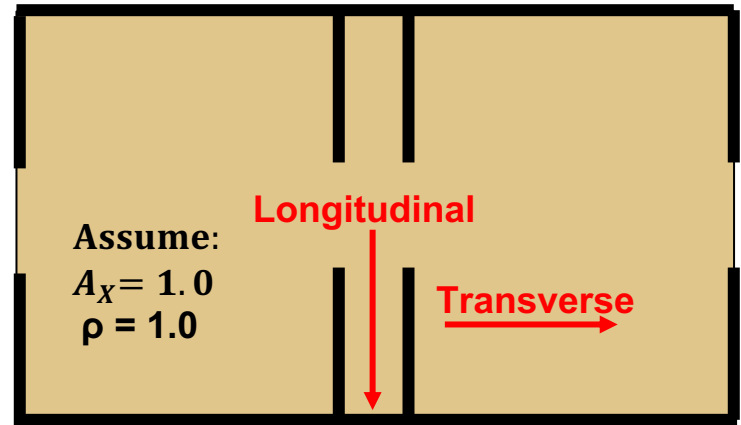
## Preliminary Assumptions-Redundancy / Irregularity Issues



Plan A

Assume:  
 $A_X = 1.0$   
 $\rho = 1.0$

Similar →

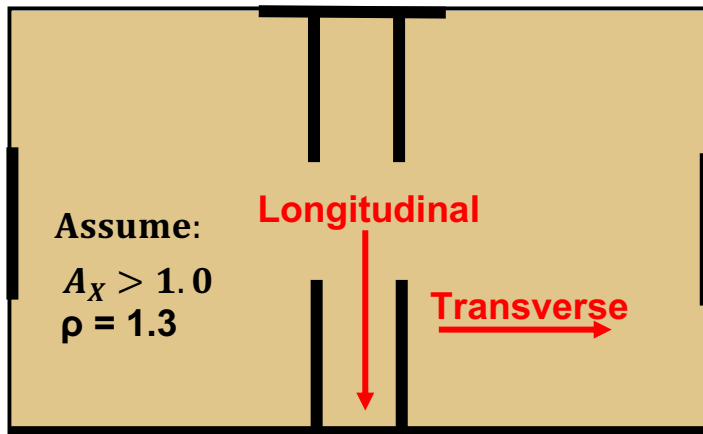


Plan B

### Regular Plans

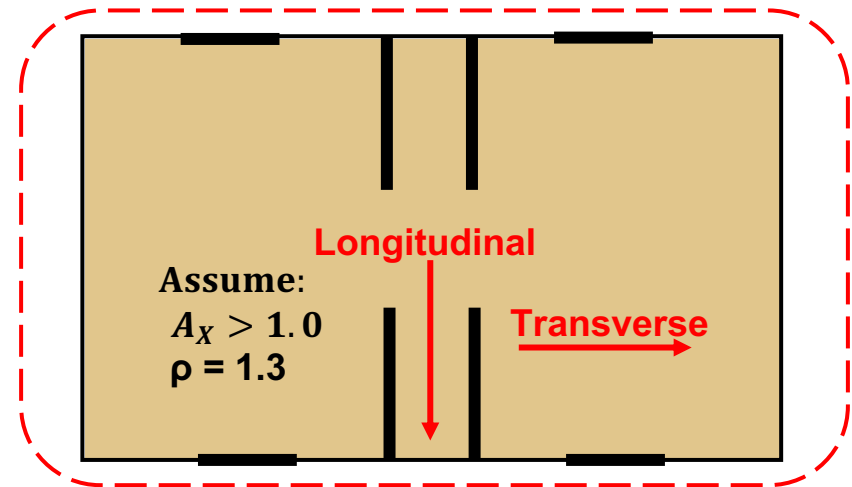
$A_X$  = Amplification of accidental torsion if torsional irregularity exists  
 $\rho$  = Redundancy

### Questionable Plans



Plan C

Assume:  
 $A_X > 1.0$   
 $\rho = 1.3$

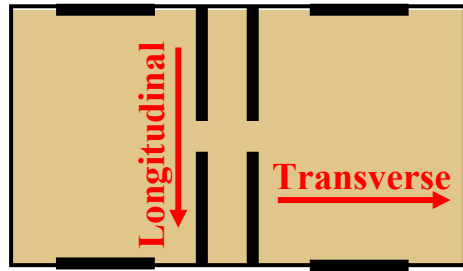


Plan D-Example

Assume:  
 $A_X > 1.0$   
 $\rho = 1.3$

# Analysis Flow- **Not in paper**

## Longitudinal Design

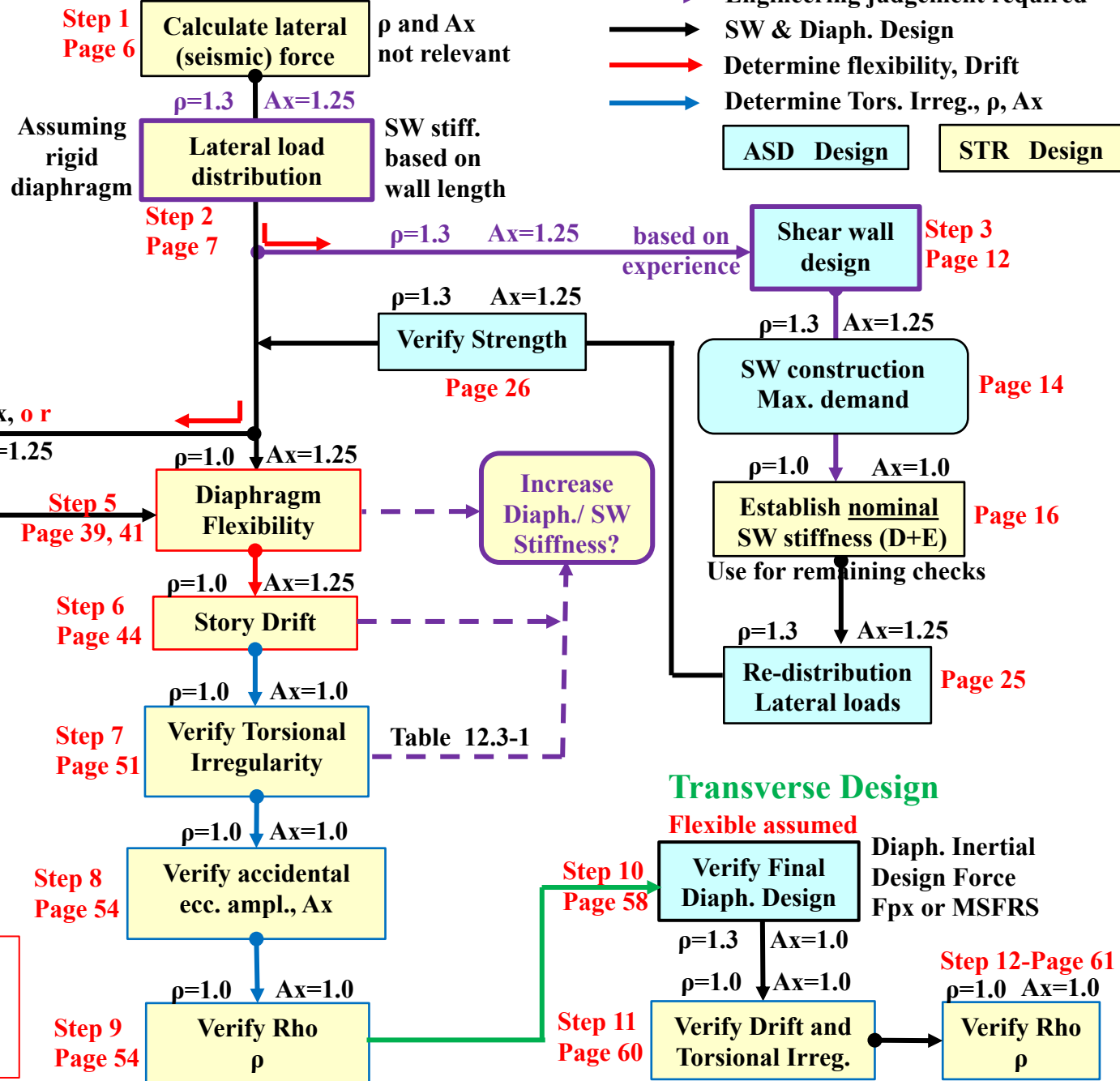


## Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



# Typical Spreadsheet

## Rigid Diaphragm Analysis

### Longitudinal Loading

Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv	Ft	Fv+Ft
2	43.54		3		130.63	391.89	8874.0	-527.1	8346.9
3	43.54		3		130.63	391.89	8874.0	527.1	9401.1
A		25.14		20	502.74	10054.73		2028.5	2028.5
B		25.14		20	502.74	10054.73		-2028.5	-2028.5
Σ	87.09	50.27			J= 20893.23		17748		

### Transverse Loading

Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv	Ft	Fv+Ft
2	43.54		3		130.63	391.89		277.4	277.4
3	43.54		3		130.63	391.89		-277.4	-277.4
A		25.14		20	502.74	10054.73	8874.0	1067.6	9941.6
B		25.14		20	502.74	10054.73	8874.0	-1067.6	7806.4
Σ	87.09	50.27			J= 20893.23		17748.0		

Loads

$\delta_{SW}$   
0.192  
0.216  
0.0807  
-0.081

Rho= 1  
Ax= 1.25  
Fy= 17748  
Cmin= 4.75  
T=Fc= 84303

Requires Input

Input p, Ax

Input or calculate  
base shear

Loads

0.006  
-0.006  
0.396  
0.311

Fx= 17748  
Cmin= 2.5  
T=Fc= 44370

Shear wall p=1.3, Ax=1.25  
Torsion, Ax p=1.0, Ax=1.0  
Flex/Drift p=1.0, Ax=1.25  
Redundancy p=1.0, Ax=1.0

Use this load combination for defining Nominal Stiffness values, Keff. Then use those Keff values for all other analyses.

Expected Dead + Seismic D+Qe (other terms if "expected" gravity loads as per ASCE 41-13, equation 7-3) p=1.0, Ax=1.0

Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\Delta_a$	$F_c$	Crush.	Shrink	$\delta_B$	$\delta_S$	$\delta_{Rot}$	$\delta_{SW}$	K (k/in)
Calculate Stiffness of Walls on A & B using Transverse loading																
A		37	1.0	7308.0	913.5	6390.85	13769.85	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581	A 25.14
B		37	1.0	7308.0	913.5	6390.85	13769.85	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581	B 25.14
Calculate Stiffness of Walls on 2 & 3 using Longitudinal loading																
2		30	1.0	7022.0	702.2	6391.13	8340.73	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484	2 43.54
3		30	1.0	7022.0	702.2	6391.13	8340.73	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484	3 43.54
V equal to revised wall force based on HD STR (design) capacity										625 Max.	Add stud					

Longitudinal Analysis

Shear Walls LC7

LC7=0.726D+pQe

Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\delta_{SW}=F/K_{eff}$
A & B	A,B	37	1.0	1014.3	126.8	-1229.16	4127.99	0.081
2	2	30	1.0	2782.3	278.2	2202.41	3617.82	0.192
3	3	30	1.0	3133.7	313.4	2572.30	3987.71	0.216

Calculate nominal stiffness  
by 3-term or 4-term  
equation.  $K=F/\delta$

$\delta_{sw}=F/K$

Shear Walls LC6

LC6=1.374D+pQe+0.2S

Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\delta_{SW}=F/K_{eff}$
A & B	A,B	37	1.0	1014.3	126.8	-4085.04	7128.71	0.081
2	2	30	1.0	2782.3	278.2	1477.15	4305.90	0.192
3	3	30	1.0	3133.7	313.4	1847.03	4675.78	0.216

## Diaphragm Deflection (STR)

Splice Forces (Lbs.)			Σδ slip in.	v unif. plf	v conc. plf	Ga k/in.	L' Ft.	W' Ft.	δDiaph Unif in.	δDiaph conc in.	Total δ in.
F 15	F23	F35									
1063.3	1158.3	3529.3	0.075	232.94	0.00	25.0	35.00	40.00	0.265	0.00	0.265
4.70	5.13	15.62									
8	16	24									
0.023	0.012	0.025									

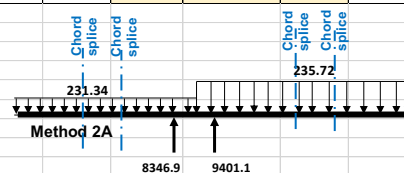
Nails Req'd=

Use Nails =

Slip=

EA= 28050000, (2)2x6

includes effects of sw's along chord line



## Diaphragm Deflection (STR)

250.3	1930.1	3622.4	0.073	229.11	0.00	25.0	35.00	40.00	0.259	0.00	0.259
1.11	8.54	16.03									
8	16	24									
0.005	0.021	0.026									

$$\delta_{diaphragm} = \frac{3pL^3}{8EI} + \frac{0.5p_{unif}L^3}{1000G_c} + \frac{EA_c E_c}{W}$$

Rt. Cantilever

W2	W1
233.53	233.53
2.19	-2.19
235.72	231.34

Lft. Cantilever

# Let's Take a 15 Minute Break



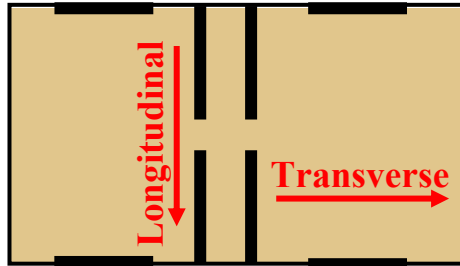


### **Part 2-Design Example :**

- **Calculation of seismic forces and distribution**
- **Preliminary shear wall design**
- **Nominal shear wall stiffness**
- **Verification of shear wall design**
- **Diaphragm design**

# Analysis Flow

## Longitudinal Design



Example Plan

### Legend

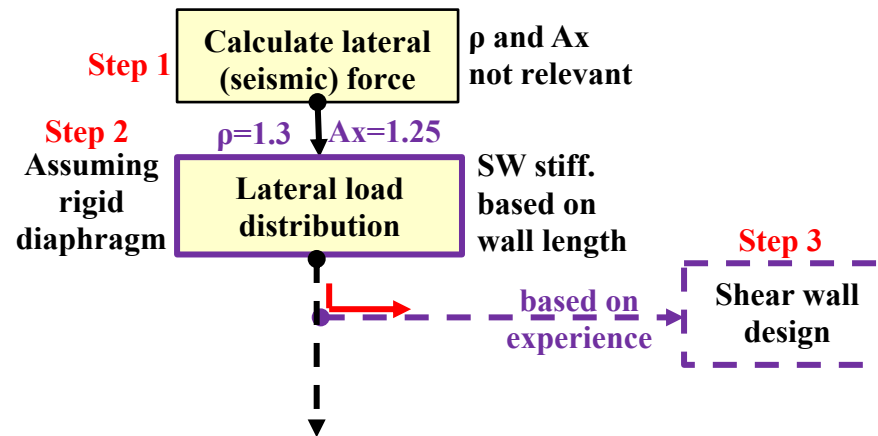
- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design

### Assumptions Made: Page 8

- Diaphragm is rigid or semi-rigid in both directions
- Torsional irregularity Type 1a occurs in longitudinal direction, but not transverse,  $A_x=1.25$ .
- Horizontal irregularity Type 1b does not occur in either direction.
- No redundancy in both directions,  $\rho=1.3$



Force Distribution to Shear Walls

Seismic-  $\rho=1.3$ ,  $A_x=1.25$

# Basic Project Information

- **Structure-Occupancy B, Office, Construction Type VB-Light framing:**
  - **Wall height=10'-Single story**
  - **L=76', total length**
  - **W'=40', width/depth**
  - **L'=35', cantilever length (max.)**
  - **6' corridor width**
- **Roof DL (seismic)= 35.0 psf including wall/ partitions**
- **Wall DL = 13.0 psf (in-plane)**
- **Roof snow load = 25 psf > required roof LL=20 psf**
- **Roof (lateral)= roof + wall H/2 plus parapet**

# **Lateral Load Calculations-Seismic**

**Calculate Seismic Forces -ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure,  $F_x$**

- **Risk category II**
- **Importance factor,  $I_e = 1.0$**

**Using USGS Seismic Design Map-Tool, 2015 NEHRP, 2016 ASCE 7-16:**

- **Location-Tacoma, Washington**
- **Site class D-stiff soil**
- **$S_s = 1.355$  g,  $S_1 = 0.468$  g**
- **$S_{Ds} = 1.084$  g,  $S_{D1} = 0.571$  g**
- **Seismic Design Category (SDC) = D**

**ASCE 7-16 Table 12.2-1, Bearing Wall System, A(15) light framed wood walls w/ WSP sheathing.  $R = 6.5$ ,  $\Omega_0=3$ ,  $C_d=4$ , Maximum height for shear wall system=65'.**

# Seismic Force Calculation results:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.167 \text{ short period controls}$$

12.8-2

Basic lateral force MSFRS

$$V = C_s W = 0.167(35)(76)(40) = 17769 \text{ lbs. STR}$$
$$17769(0.7) = 12438 \text{ lbs. ASD}$$

Rigid Diaphragm Analysis-  $\rho=1.3$ ,  $A_x=1.25$

Initial wall stiffness will be based on wall length.

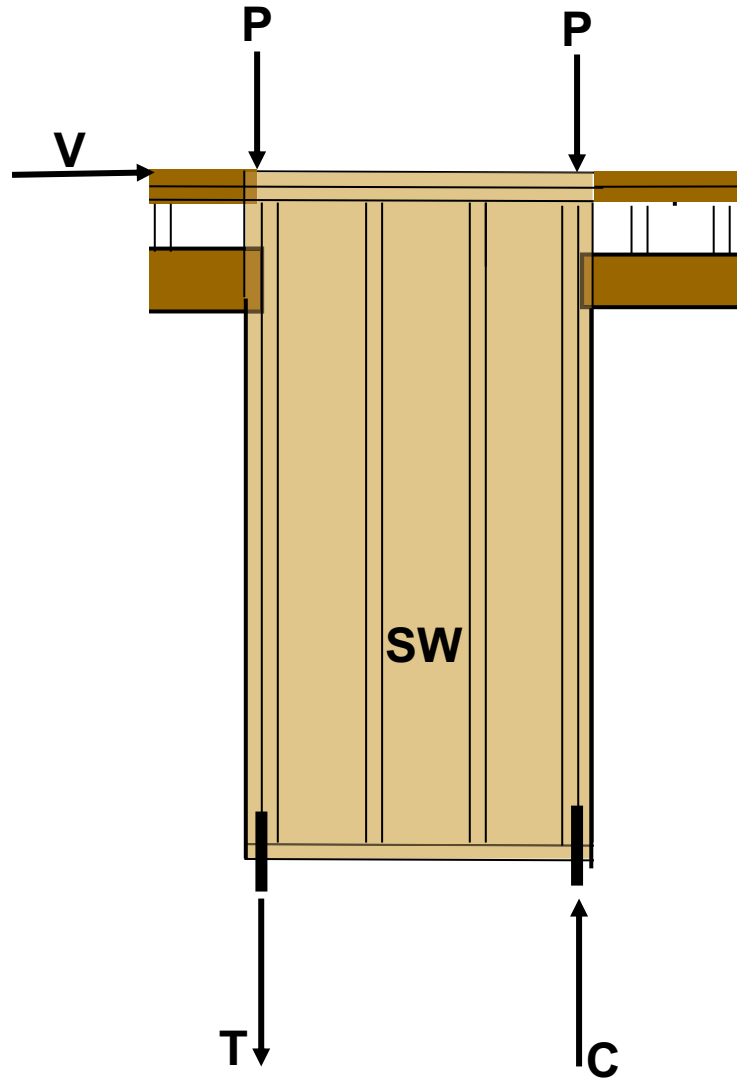
The final wall Nominal stiffness's are used for all final analysis checks.

## RDA Equations

$$T = V(e)(A_x)(\rho) \text{ ft. lbs.}$$
$$F_T = T \frac{kd}{\sum kd_x^2 + kd_y^2}$$
$$F_{sw} = F_V + F_T$$

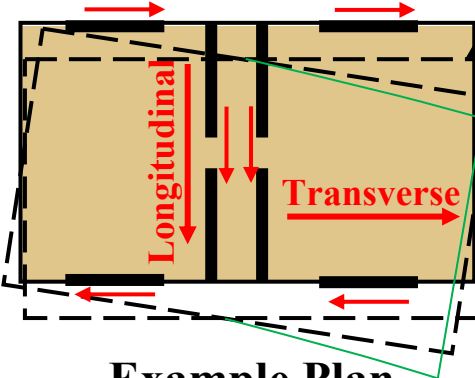
$$J = \sum kd_x^2 + kd_y^2$$
$$F_V = F_x \frac{k}{\sum k}$$

# Preliminary Shear Wall Design



# Analysis Flow

## Longitudinal Design



## Example Plan

- Drift
- Torsional Irreg.

Translation  
Rotation  
Diaphragm deflection

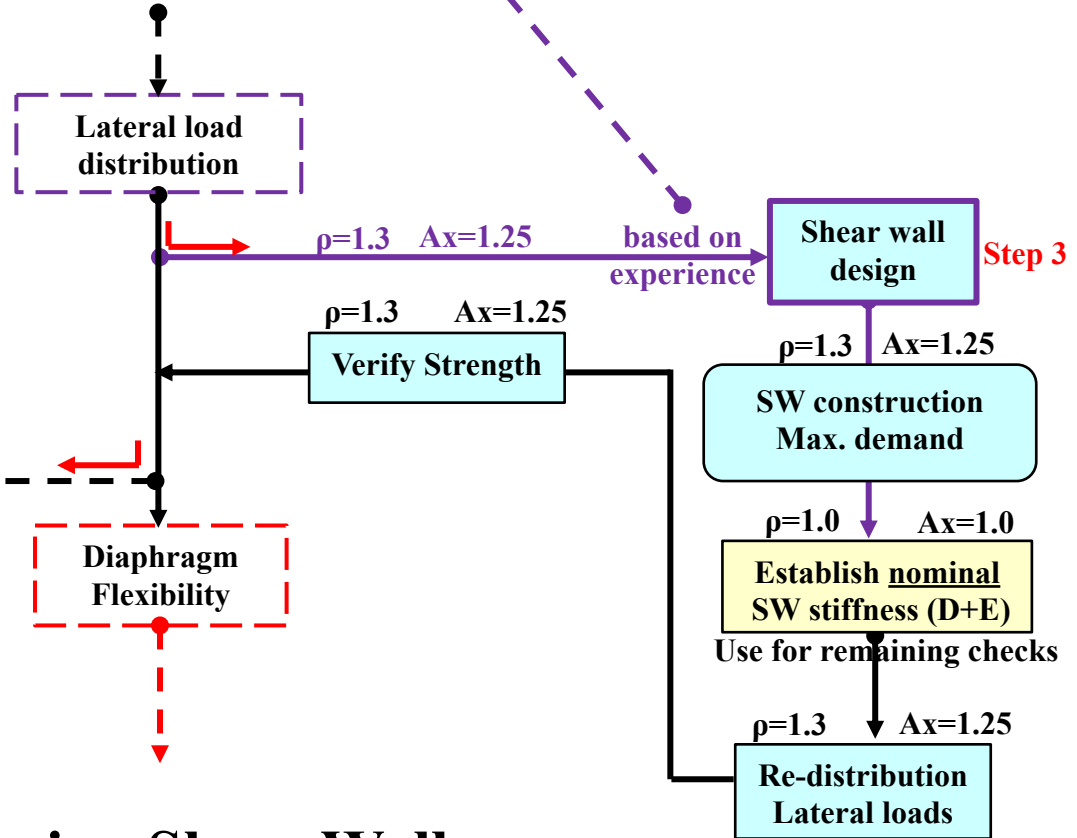
Displacements effected by wall stiffness

## Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



**Design Shear Walls**  
**Seismic-  $\rho=1.3$ ,  $A_x=1.25$**

# Preliminary Shear wall Design (**ASD**): ASCE 7-16 Section 2.3.6-Seismic

---

- Determine shear wall chord properties:

2x6 DF-L no. 1 framing used throughout.

$E = 1,700,000$  psi, wall studs @ 16" o.c.

$EA = 42,075,000$  lbs. at grid line **A,B** = (3)2x6 D.F., KD, studs @16" o.c. boundary elem.

$EA = 28,050,000$  lbs. at grid line **2,3** = (2)2x6 D.F., KD, studs @16" o.c. boundary elem.

- Check aspect ratio, If  $A.R. > 2:1$ , reduction is required per SDPWS Section 4.3.4.
- Hold downs = pre-manufactured bucket style with screw attachments **Same H.D used at all SW locations**
  - Manuf. table gives Allowable ASD hold down capacity and displacement at capacity (ESR Reports)
  - Displacement at hold down =  $\frac{T(Allow.Displ)}{ASD\ Capacity}$
  - Min. wood attachment thickness = 3" per table



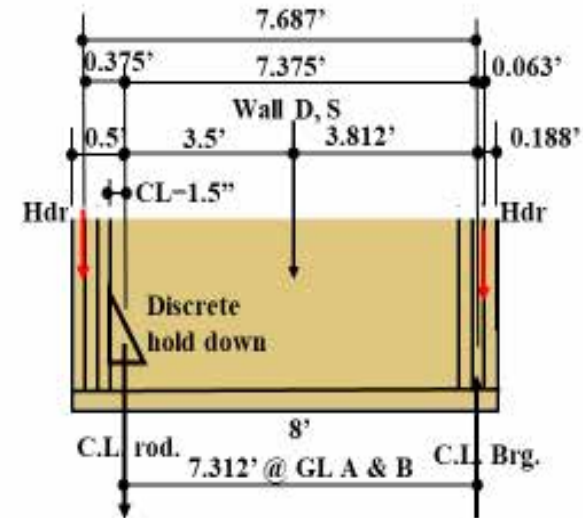
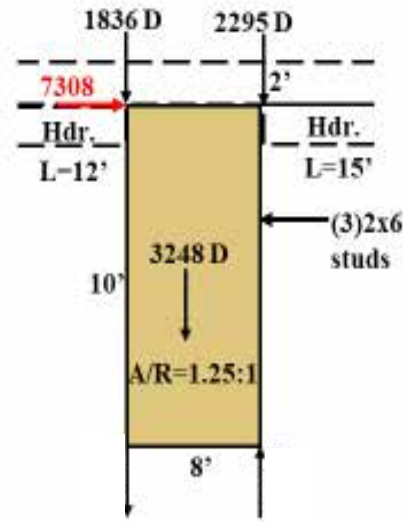
## Load Combinations (ASD):

$$LC8 = 1.152D + 0.7pQ_E$$

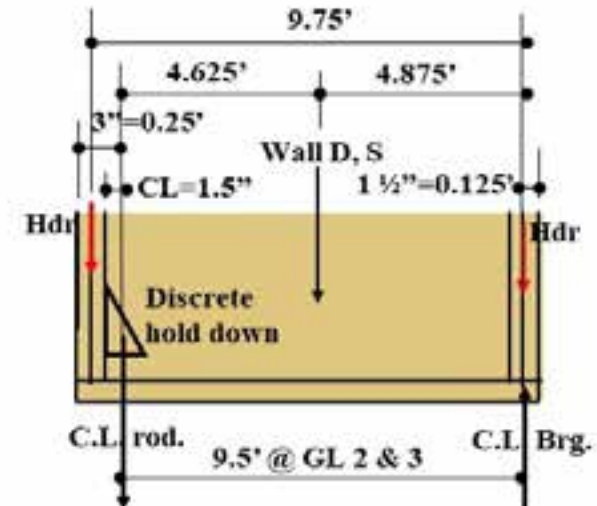
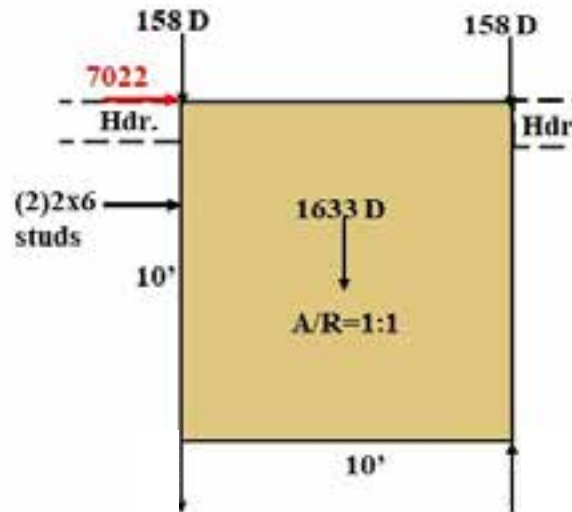
$$LC9 = 1.114D + 0.525pQ_E + 0.75S$$

$$LC10 = 0.448D + 0.7pQ_E$$

Full dead loads shown, 1.0D



Shear Walls Along Grid Lines A and B  
Design Dimensions



Shear Walls Along Grid Lines 2 and 3  
Design Dimensions

Based on initial Relative Wall Stiffness's, **ASD**,  $\rho=1.3$ ,  $A_x=1.25$  –by wall lengths

Longitudinal Direction,  $e=4.75'$ ,  $T = 76806.5$  ft. lbs.

SW Line	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd <sup>2</sup>	Fv Lbs.	F <sub>T</sub> Lbs.	Total Lbs.
A	-----	16	-----	20	320	6400	0	1842.4	1842.4
B	-----	16	-----	20	320	6400	0	-1842.4	-1842.4
2	30	-----	3	-----	90	270	8084.9	-518.2	7566.7
3	30	-----	3	-----	90	270	8084.9	518.2	8603.1

$\Sigma K_y=60$   $\Sigma K_x=32$

J=16169.8

Corridor Walls at Grid  
Walls lines A & B

Transverse Direction,  $e=2.5'$ ,  $T = 40424.5$  ft. lbs.

SW Line	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd <sup>2</sup>	Fv Lbs.	F <sub>T</sub> Lbs.	Total Lbs.
A	-----	16	-----	20	320	6400	8084.9	969.7	9054.6
B	-----	16	-----	20	320	6400	8084.9	-969.7	7115.2
2	30	-----	3	-----	90	270	0	-272.7	-272.7
3	30	-----	3	-----	90	270	0	272.7	272.7

$\Sigma K_y=60$   $\Sigma K_x=32$

J=16169.8

Corridor Walls at Grid  
Walls lines A & B

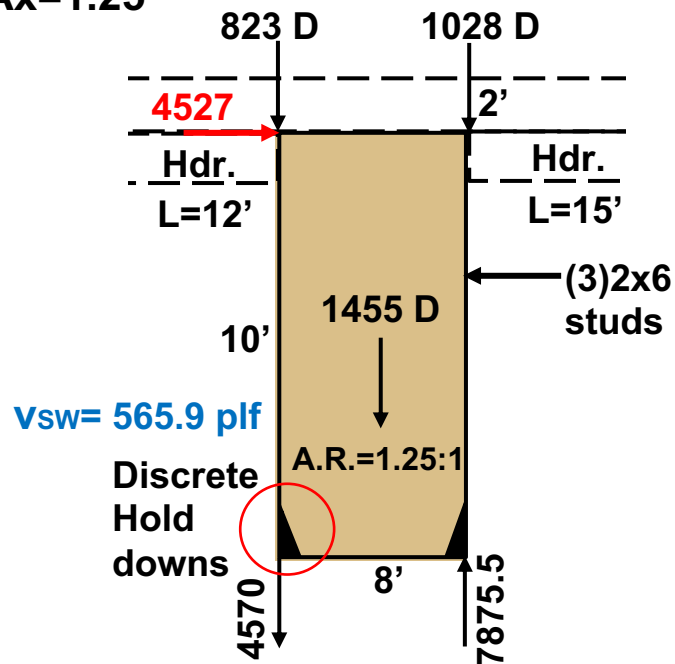
# Preliminary Shear Wall Design-Distribution based on wall lengths

## Adding Gravity Loads to Shear Walls

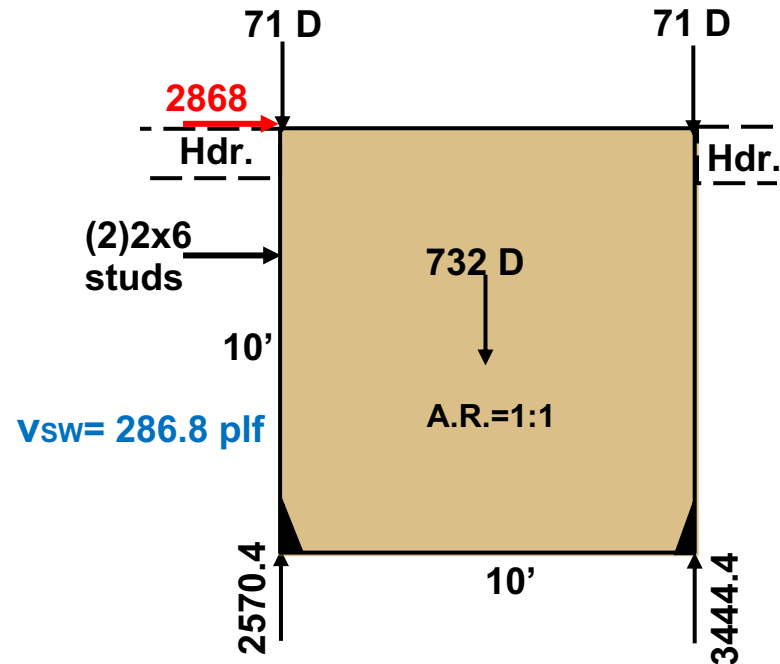
- Can have a significant impact on horizontal shear wall deflections and stiffness.
- Results in wall stiffness ( $K = F/\delta$ ) relationships which are non-linear with the horizontal loading applied.

**ASD Load Combination:  $LC10 = 0.448D + 0.7\rho Q_E$**

$\rho = 1.3$ ,  $A_x = 1.25$



Shear Walls Along Grid Lines A and B  
**Transverse Loading**



Shear Walls Along Grid Lines 2 and 3  
**Longitudinal Loading**

# Calculated results by wall length

**V<sub>SW A,B</sub> = 565.9 plf**

**V<sub>SW 2,3</sub> = 286.8 plf**

## Shear Wall Capacity-Wood Based Panels

**Blocked**

**Table 4.3A Nominal Unit Shear Capacities for Wood-Framed Shear Walls**

Wood Based Panels <sup>4</sup>											
Sheathing Material	Minimum Nominal Panel Thickness (in.)	Minimum Fastener Penetration In Framing Member or Blocking (in.)	Fastener Type & Size Nail (common or Galvanized box)	A Seismic				B Wind			
				Panel Edge Fastener Spacing (in.)				Panel Edge Fastener Spacing (in.)			
				6	4	3	2	6	4	3	2
				(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)	(plf) (kips/in.)
Wood <sup>4,5</sup> Structural Panels- Sheathing	15/32	1-3/8	8d	Vs Ga OSB PLY 520 13 10	Vs Ga OSB PLY 760 19 13	Vs Ga OSB PLY 980 25 15	Vs Ga OSB PLY 1280 39 20	Vw 730	Vw 1065	Vw 1370	Vw 1790
	<b>15/32</b>			<b>620 22 14</b>	<b>920 30 17</b>	<b>1200 37 19</b>	<b>1540 52 23</b>	<b>870</b>	<b>1290</b>	<b>1680</b>	<b>2155</b>
	19/32	1-1/2	10d	680 19 13	1020 26 16	1330 33 18	1740 48 28	950	1430	1860	2435

**Increasing stiffness to account for drift, torsion, etc. requires engineering judgement.**

**SW<sub>A,B</sub>: Use 15/32" OSB w/ 10d@3" o.c., v<sub>s</sub>= (1200)/2 = 600 plf, Ga=37**

**SW<sub>2,3</sub>: Use 15/32" OSB w/ 10d@4" o.c., v<sub>s</sub>= (920)/2 = 460 plf, Ga=30**

**Maximum tension force, T= 4570 lbs.- Use HD=4565 lbs. (0.1% under-check later)**

**ASD, Δa=0.114" @ capacity**

**STR, Δa=0.154" @ capacity**

# Determination of Nominal Wall Stiffness

---

Combining Rigid Diaphragm Analysis & shear wall deflection calculations is problematic due to non-linearities. Whenever changing:

- Load combinations
- Vertical or lateral loads,
- Direction of loading
- Redundancy, or
- Accidental torsion

...it can effect the distribution of loads to the shear walls which will effect the shear wall deflections. This can lead to a different set of stiffness values that may not be consistent.

**Requires an Iterative search for the point of convergence, which is not practical for multi-story structures.**

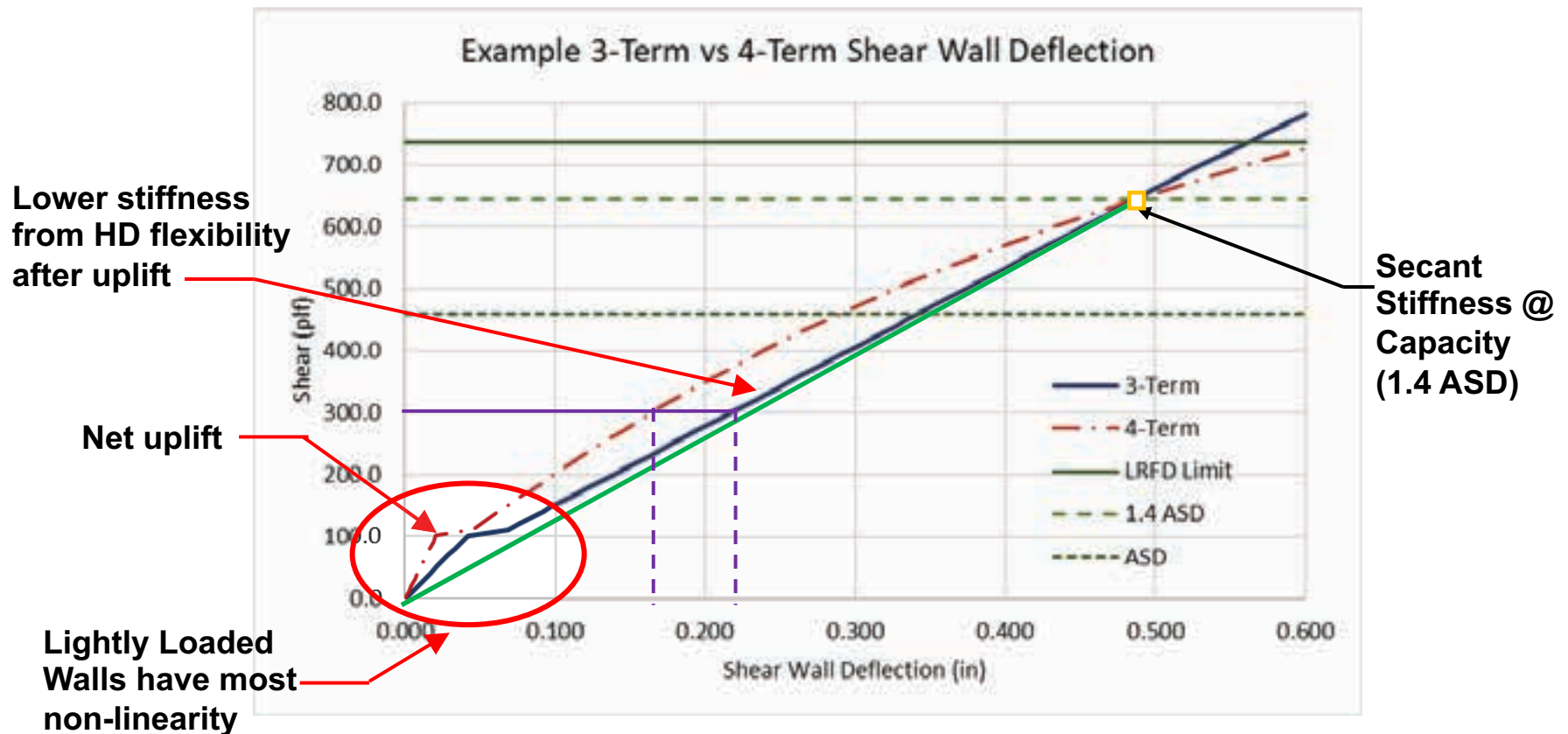
**Sources of non-linearities:**

- Hold-down slip at uplift (e.g. shrinkage gap)
- Hold-down system tension and elongation
- Compression crushing. Non-linear in NDS
- Shrinkage
- 4-term deflection equation

Since deflection is “non-linear”.... the stiffness can vary with the loading, even when using 3-term deflection equation.

## LATERAL Load for Shear Wall Deflection & Stiffness Calculations

- 3-term equation is a linear simplification of the 4-term equation, calibrated to match the applied load at 1.4 ASD.
- This simplification removes the non-linear behavior of  $e_n$ .
- Similar approach can be used to remove non-linear effects of  $\Delta_a$  by calculating the wall stiffness at **strength level capacity of the wall**, not the applied load.



**Method allows having only one set of nominal stiffness values.**

## Objective:

Use a single rational vertical and lateral load combination to calculate deflections and **Nominal** shear wall stiffness.

## Gravity Loads:

A simplification of gravity loads are applied similar to nonlinear procedures in ASCE 41-13 in ASCE 41-13 Eq. 7-3.

For this *Single-Story* Example we used **1.0D**, using  $\rho = 1.0$  and  $A_x = 1.0$ . Vertical seismic loading not included. ( $E_v = 0.2S_{DS}D$ )

For multi-story buildings, suggest **1.0D +  $\alpha L$**  as in ASCE 7-16 Section 16.3.2- Nonlinear analysis

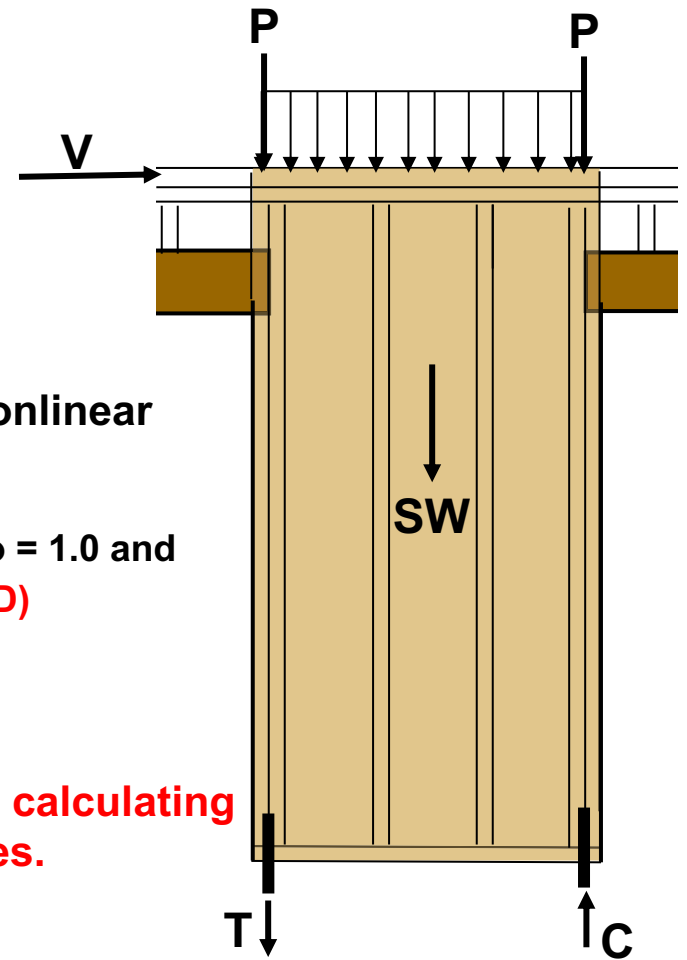
**Results in single vertical loading condition to use when calculating shear wall deflections and nominal shear wall stiffnesses.**

## Proposing:

1. Stiffness calculated using 3-term eq. and LC **1.0D+Qe**, with  $\rho=1.0$  and  $A_x=1.0$ .
2. Use stiffness calculated at 100% Maximum Seismic Design Capacity of the Wall for all Load Combinations and Drift Checks from RDA using 3 term equation.
3. Use nominal stiffness for all other analysis checks, calculating wall deflection,

$$\delta_{SW} = \frac{F}{K}$$

4. Maximum wall capacity = max. allow. Shear (nailing) or HD capacity whichever is less.



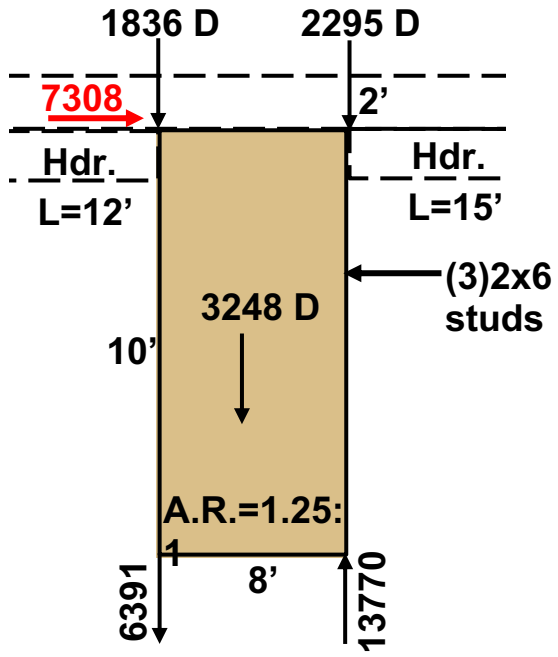


# Nominal Shear Wall Stiffness's (STR) $\rho=1.0, A_x=1.0$

Load Combination:  $1.0D + Q_E$

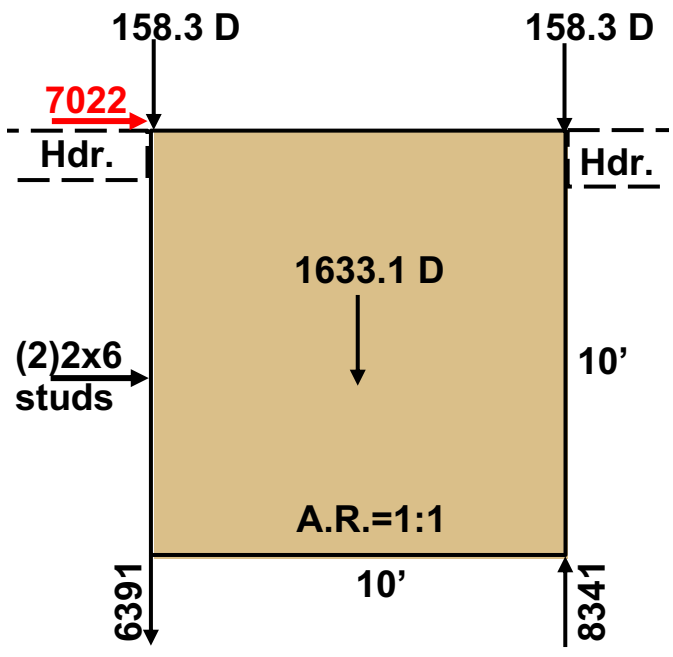
Grid Line	Ga	V on wall	v	T	C	$\Delta_a$	$F_c$	Crush.	Shrink	$\delta_B$	$\delta_S$	$\delta_{Rot}$	$\delta_{SW}$
Calculate Stiffness of Walls on A & B using LRED Capacity													
A	37	7308.0	913.5	6391	13770	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581
B	37	7308.0	913.5	6391	13770	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581
Calculate Stiffness of Walls on 2 & 3 using LRFD Coading													
2	30	7022.0	702.2	6391	8341	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484
3	30	7022.0	702.2	6391	8341	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484

Wall Capacity based on hold down



Shear wall Grid A and B  
Trib. = 10'

Transverse Loading  
Nominal Strength



Shear wall Grid 2 and 3  
Trib. = 2'

Longitudinal Loading  
Nominal Strength

	K (k/in)
A	25.14
B	25.14
Aver.=	25.14
2	43.54
3	43.54
Aver.=	43.54

Max. capacity check (STR):

$Shear_{A,B} = 0.8(1200)(8) = 7680 \text{ lbs.}$   
 $Shear_{2,3} = 0.8(920)(10) = 7360 \text{ lbs.}$

$H.D._{A,B,2,3} = 6391 \text{ lbs. (STR)}$   
 $\Delta a = 0.154''$

Set tension force=H.D. cap. and solve for allowable V.

$V_{allow. A,B} = 7308 \text{ lbs. controls}$   
 $V_{allow. 2,3} = 7022 \text{ lbs. controls}$

# Verification of Wall Strength (ASD)

Based on selected wall construction and Nominal Wall Stiffness

Longitudinal Direction,  $e=4.75'$ ,  $T = 76806.5$  ft. lbs.  $\rho=1.3$ ,  $A_x=1.25$

SW Line	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd <sup>2</sup>	Fv Lbs.	F <sub>T</sub> Lbs.	Total Lbs.	Corridor Walls  Walls at Grid lines A & B
A	-----	25.14	-----	20	502.8	10056	0	1848.1	1848.1	
B	-----	25.14	-----	20	502.8	10056	0	-1848.1	-1848.1	
2	43.54	-----	3	-----	130.62	391.86	8084.9	-480.1	7604.8	
3	43.54	-----	3	-----	130.62	391.86	8084.9	480.1	8565.0	
ΣKy=87.08   ΣKx=50.28						J=20895.72				

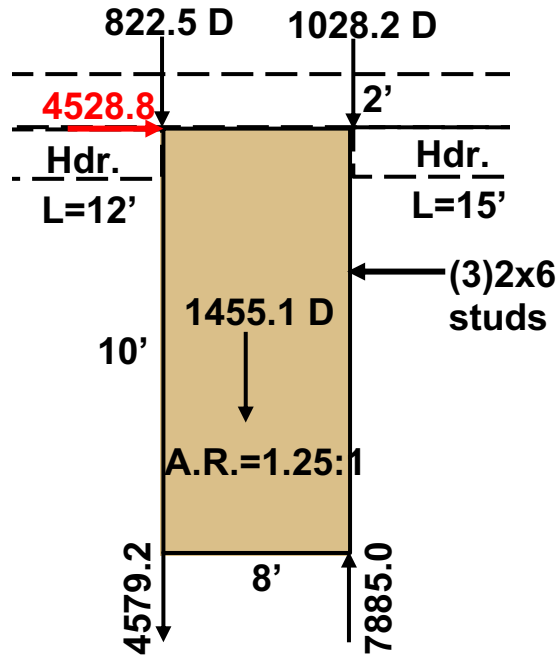
Transverse Direction –  $e=2.5'$ ,  $T = 40424.5$  ft. lbs.  $\rho=1.3$ ,  $A_x=1.25$

SW	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd <sup>2</sup>	Fv Lbs.	F <sub>T</sub> Lbs.	Total Lbs.	Corridor Walls at Grid Walls      lines A & B
A	-----	25.14	-----	20	502.8	10056	8084.9	972.7	9057.6	
B	-----	25.14	-----	20	502.8	10056	8084.9	-972.7	7112.2	
2	43.54	-----	3	-----	130.62	391.86	0	252.7	252.7	
3	43.54	-----	3	-----	130.62	391.86	0	-252.7	-252.7	
ΣKy=87.08   ΣKx=50.28						J=20895.72				

Nominal stiffness values used

**ASD Load Combination: LC10 0.448D + 0.7pQE**

$\rho=1.3$ ,  $A_x=1.25$

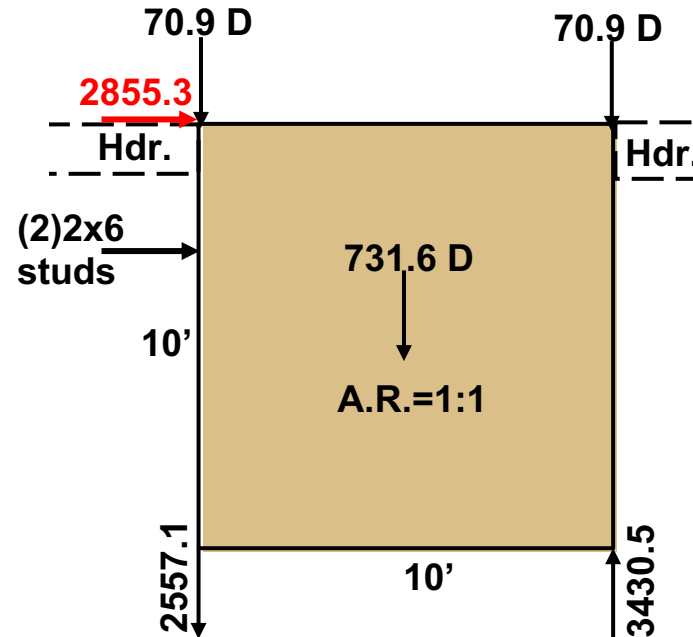


**Shear wall Grid A and B**

**Shear Walls Along Grid Lines A and B**  
**Transverse Loading-** Nominal Strength

$$v_s = \frac{4528.8}{8} = 566.1 \text{ plf} < 600 \text{ plf allowed} \therefore \text{o.k.}$$

$T = 4579.2 \text{ lbs.} \approx 4565 \text{ lbs. allowed, } 0.3\% \text{ over}$   
 $\therefore$  hold down o.k. –check later



**Shear wall Grid 3**

**Shear Walls Along Grid Lines 2 and 3**  
**Longitudinal Loading-** Nominal Strength

$$v_s = \frac{2855}{10} = 285.5 \text{ plf.} < 460 \text{ plf allowed} \therefore \text{o.k.}$$

$T = 2557.1 \text{ lbs.} < 4565 \text{ lbs. allowed}$   
 $\therefore$  hold down o.k.

# SW Design Checks

- A.R. = 1.25:1 < 3.5:1 Since the A.R. does not exceed 2:1, no reduction is required per SDPWS Section 4.3.4.

- Wall shear:  $V_{sw A, B} = \frac{V_{wall line}}{2}$  Lbs. each wall segment,  $v_s = \frac{V_{wall}}{L_{wall}}$  plf

- Check anchor Tension force  $\leq$  Allowable.  $\therefore$  okay?

Calculate actual anchor slip,  $slip = \frac{\text{Max slip at capacity}(T)}{\text{Strength capacity}}$

- Sill plate shrinkage:

Dimensional change = 0.0025 inches per inch of cross-sectional dimension for every 1 percent change in MC.

Shrinkage = (0.0025)(D)(Starting MC - End MC)

Where: D is the dimension of the member in the direction under consideration, in this case the thickness of a wall plate.

- Sill plate crushing:**

$F'_{c\perp}$  values in AWC 2018 NDS section 4.2.6 are based on 0.04" deformation/crushing limit for a steel plate bearing on wood.

Adjustment factor = 1.75 for parallel to perpendicular grain wood to wood contact.

Boundary values for bearing perpendicular to grain stresses and crushing-D.F.

$$F_{c\perp 0.02} = 0.73 F'_{c\perp} = 0.73(625) = 456.3 \text{ psi}$$

$$F_{c\perp 0.04} = F'_{c\perp} = 625 \text{ psi}$$

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

$$\Delta_{crush} = 0.02 \left( \frac{f_{c\perp}}{F_{c\perp 0.02}} \right)$$

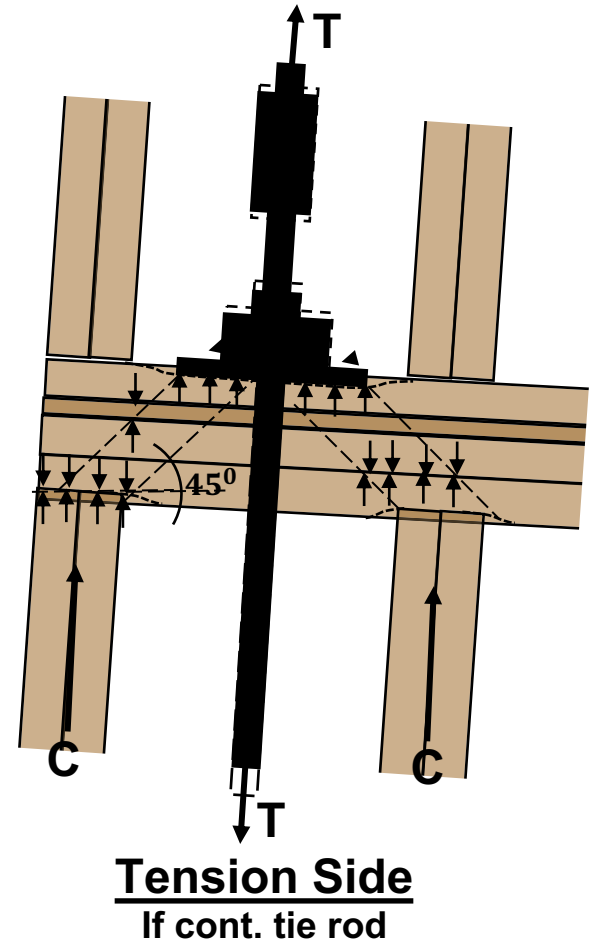
When  $F_{c\perp 0.02} < f_{c\perp} \leq F_{c\perp 0.04}$

$$\Delta_{crush} = 0.04 - 0.02 \left( \frac{1 - \frac{f_{c\perp}}{F_{c\perp 0.04}}}{0.27} \right)$$

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

$$\Delta_{crush} = 0.04 \left( \frac{f_{c\perp}}{F_{c\perp 0.04}} \right)^3$$

If  $f_{c\perp} = \left( \frac{C}{A_{chord}} \right) < 456.3 \text{ psi}$ ,  $\text{Crushing} = 0.02 \left( \frac{f_{c\perp}}{456.3} \right) (1.75)$

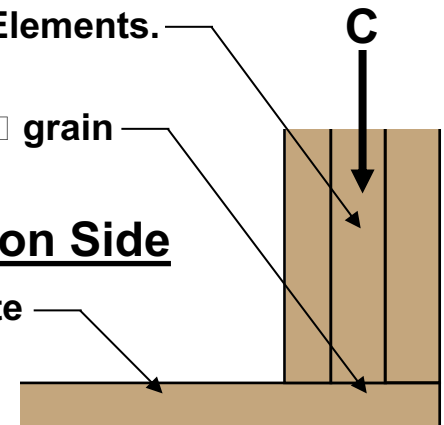


SW boundary Elements.  
A=24.75 in<sup>2</sup>

Crushing // to □ grain  
Factor = 1.75

**Compression Side**

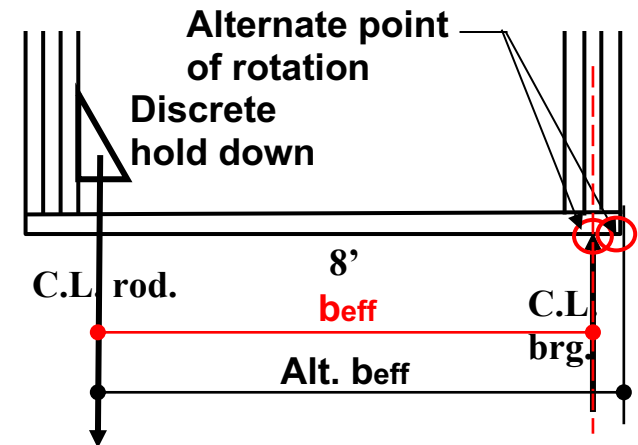
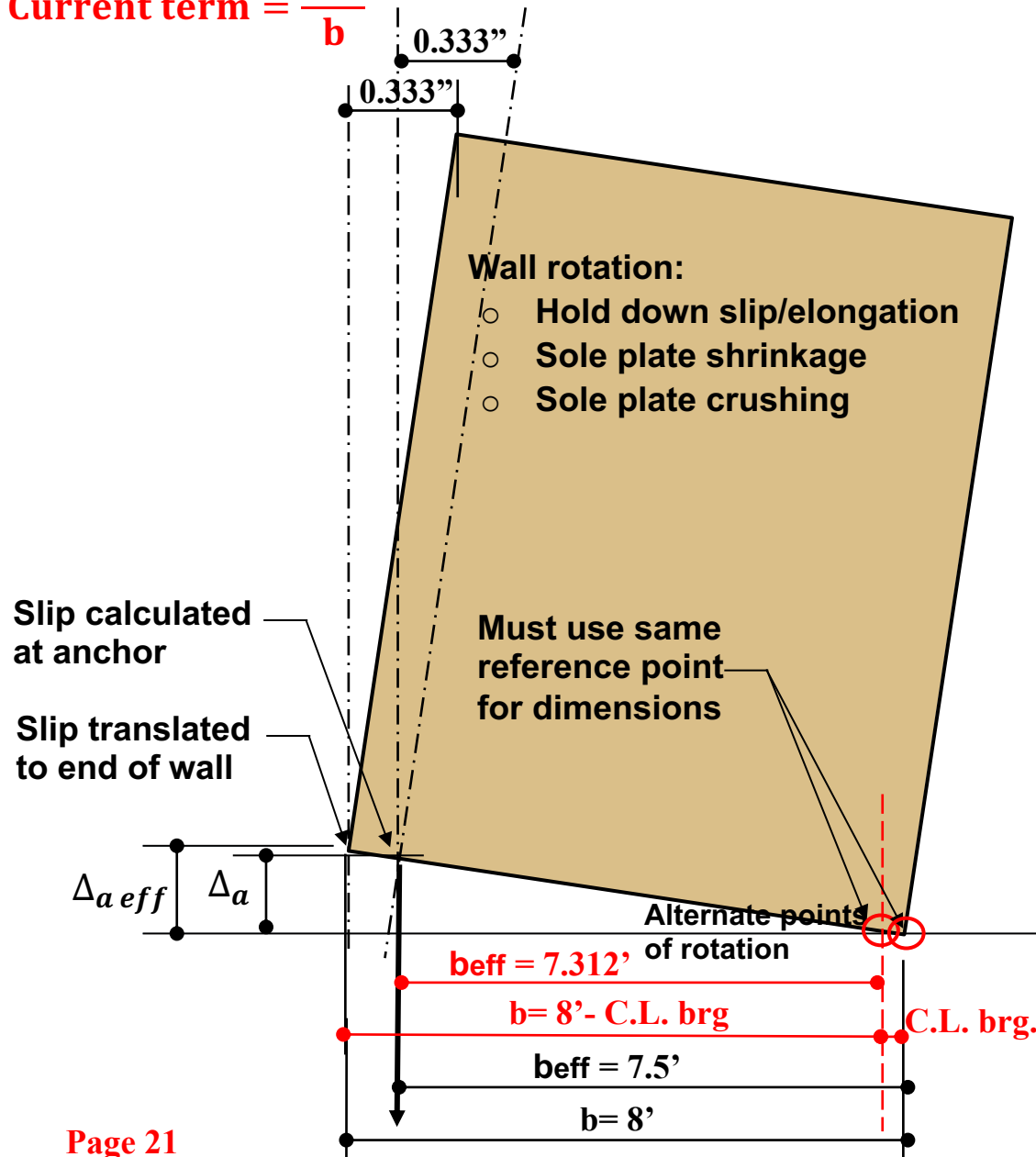
Sill plate



# Shear Wall Rotation

Proposed nomenclature of next edition of SDPWS

Current term =  $\frac{h\Delta_a}{b}$



$$SW_{rot} = \frac{h\Delta_a}{b_{eff}} \text{ or } SW_{rot} = \frac{h\Delta_{a\ eff}}{b}$$

Where

$h$ =wall height (ft.)

$b_{eff}$  =Wall rotation arm (ft.)

$b$ =Wall width (ft.)

$\Delta_{a\ eff}$ =Sum of vertical displacements at anchorage (in.)

$\Delta_a$ =Sum of vertical displacements at tension edge of wall

$$\Delta_a = 0.25''$$

$$\Delta_{a\ eff} = \frac{0.25(8)}{7.5} = 0.267''$$

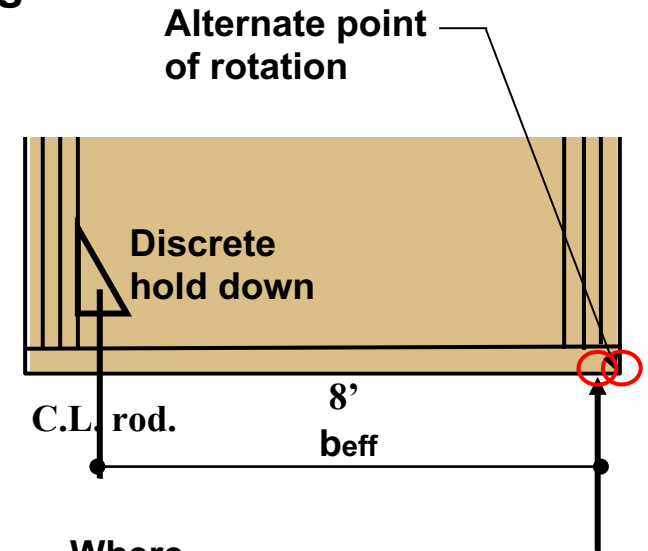
$$SW_{rot} = \frac{10(0.25)}{7.5} = 0.333''$$

$$SW_{rot} = \frac{10(0.267)}{8} = 0.333''$$

- Nominal Shear Wall Deflection-calculated using:  
Traditional 4 term deflection equation

$$\delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{G_v t_v} + \overbrace{+0.75he_n}^{\text{SDPWS combines}} + \frac{h\Delta_a}{b_{eff}} \quad \text{C4.3.2-1}$$

Bending                      Shear                      Nail slip                      Rod elongation (Wall rotation)



Where

$v$ =wall unit shear (plf)

$h$ =wall height (ft.)

$b_{eff}$ =Wall rotation width (ft.)

$b$ =Wall width (ft.)

$G_a$ =apparent shear stiffness (k/in.)

$\Delta_a$ =Sum of vertical displacements at anchorage and boundary members (in.)

### SDPWS 3 term deflection equation

$$\delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b_{eff}} \quad \text{4.3-1 Alt.}$$

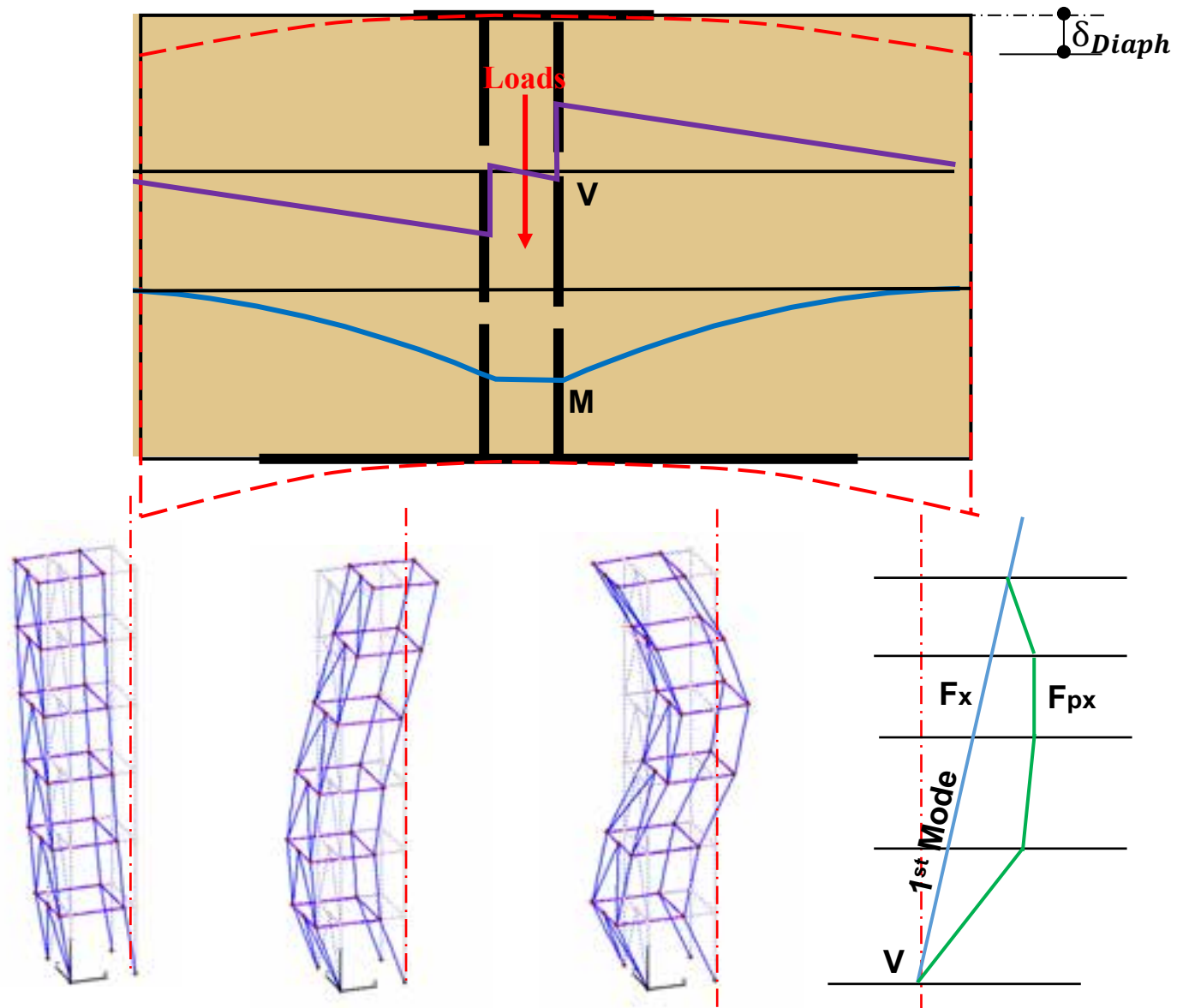
Bending                      Apparent shear stiffness                      Vertical elongation

- Nail slip
- Panel shear deformation
- Device elongation
- Rod elongation

Calculate deflection  $\delta_{sw A,B} = \frac{F}{k}$  to be used after

Nominal stiffness has been established

# Diaphragm Design

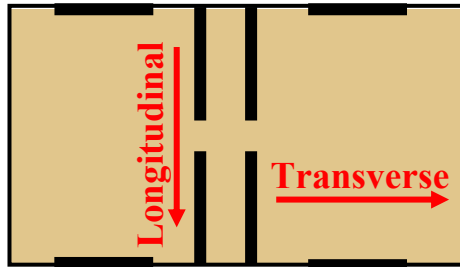


Diaphragm Design Forces: MSFRS or  $F_{px}$



# Analysis Flow

## Longitudinal Design



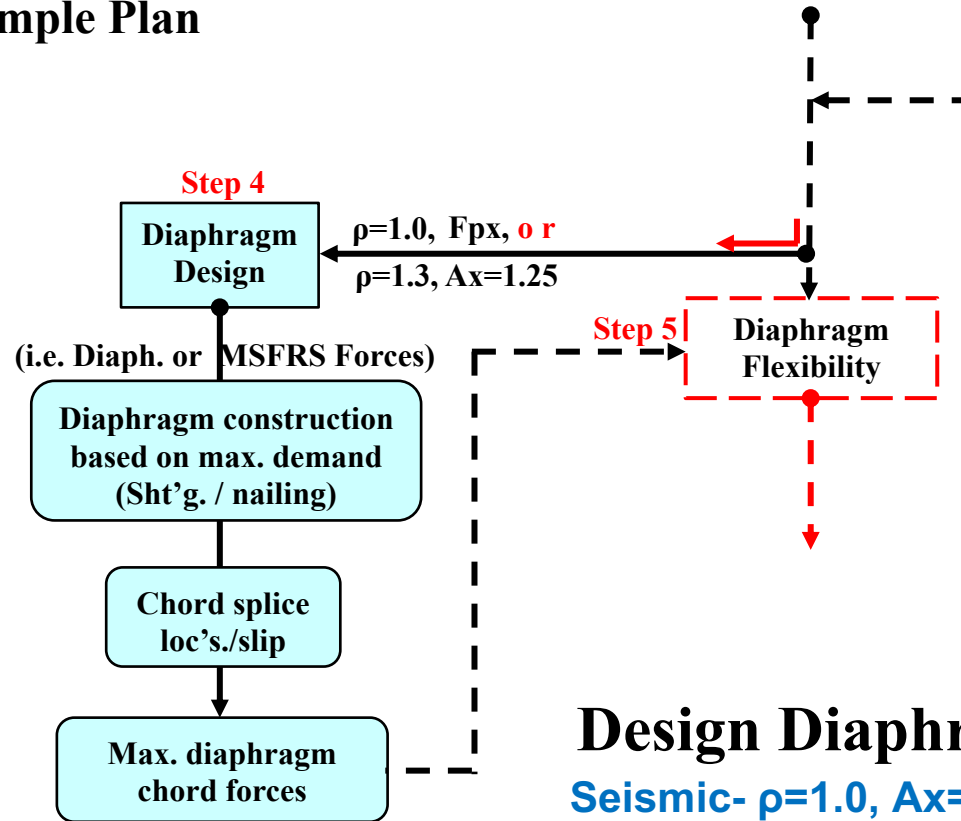
Example Plan

## Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



## Design Diaphragm

Seismic-  $\rho=1.0, A_x=1.25$  or  $\rho=1.3, A_x=1.25$

## Verification of Compliance with SDPWS 4.2.5.2-Open Front Structures:

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: WSP-L'/W' ratio  $\leq 1.5:1$  A.R. = 0.875 OK
2. For open-front-structures that are also **torsionally irregular** L'/W' ratio shall not exceed **1:1** for structures one story in height. A.R. = 0.875 OK
3. The drift at edges shall not exceed the ASCE 7 allowable story drift when subject to **seismic** design forces including torsion, and accidental torsion (Deflection-strength level amplified by Cd. ). **To be verified later**
4. For loading parallel to open side: Model as semi-rigid or rigid Assumed to be rigid both directions. **To be verified later**
5. The diaphragm length, L', (normal to the open side) does not exceed **35** feet.  
**L'max = 35' OK**

### 12.10.1.1 Diaphragm Design Forces.

The diaphragm must be designed to the maximum of these two:

- MSFRS Diaphragm (structure) Load ( $F_x$ ) or,
- Controlling Diaphragm inertial Design Load ( $F_{px}$ ) Per Eq. 12.10-1 as follows:

$$F_{px} = \frac{\sum_{i=x}^n F_i}{\sum_{i=x}^n w_i} w_{px} \quad (12.10-1)$$

where

$F_{px}$  = the diaphragm design force at level x

$F_i$  = the design force applied to level i

$w_i$  = the weight tributary to level i

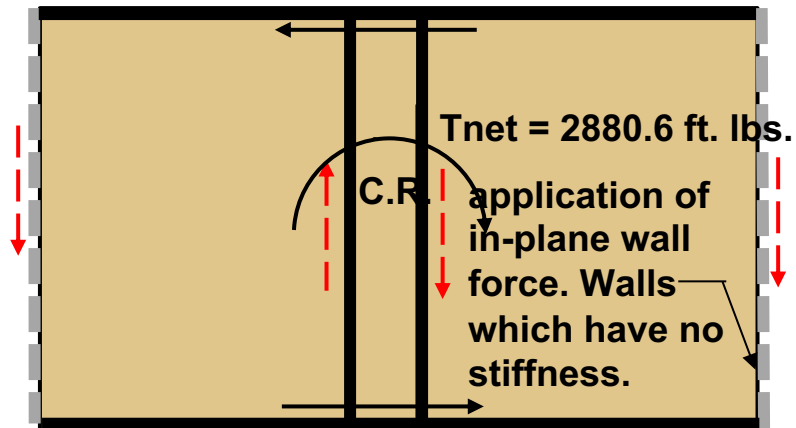
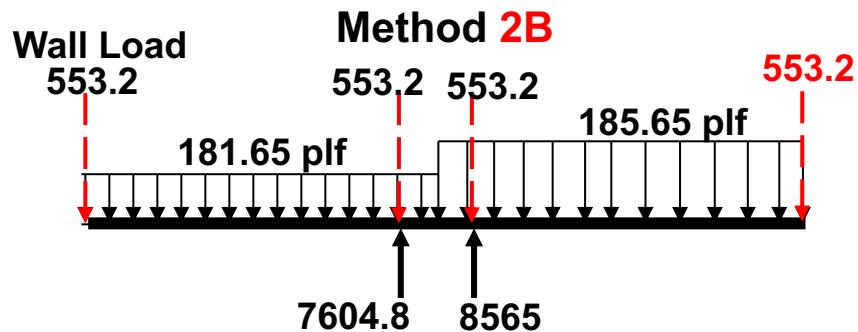
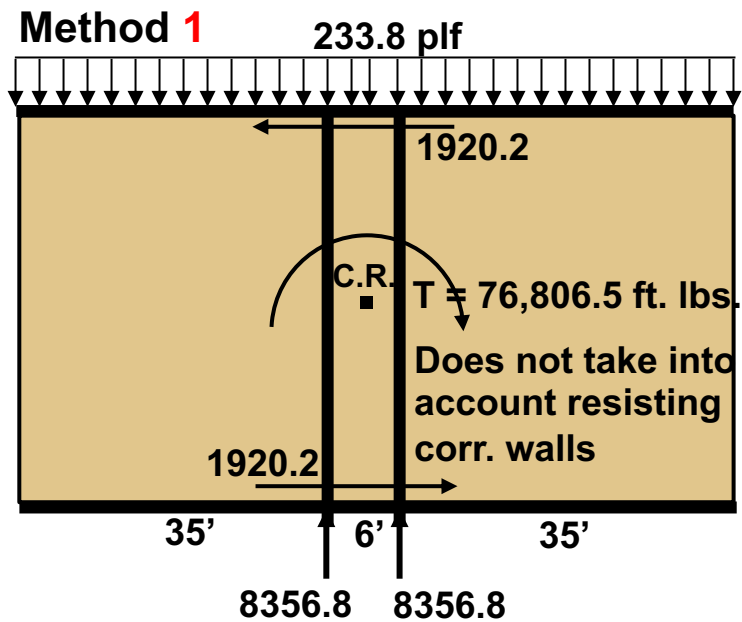
$w_{px}$  = the weight tributary to the diaphragm at level x

$$\text{The force shall not be less than } F_{px} = 0.2S_{DS}w_{px} \quad (12.10-2)$$

$$\text{The force need not exceed } F_{px} = 0.4S_{DS}w_{px} \quad (12.10-3)$$

For inertial forces calculated in accordance with Eq. 12.10-1,  $p=1.0$  per ASCE 7-16 Section 12.3.4.1, Item 7.

$$\text{For a single story structure } F_x = F_{px} = \frac{S_{DS}I_e}{R} w_{px}$$



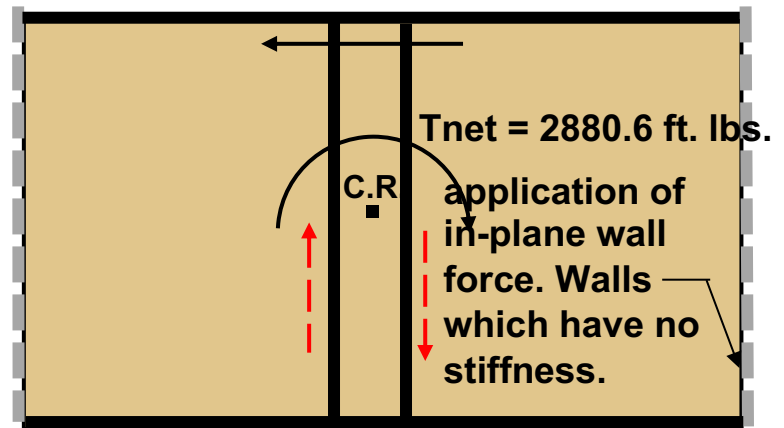
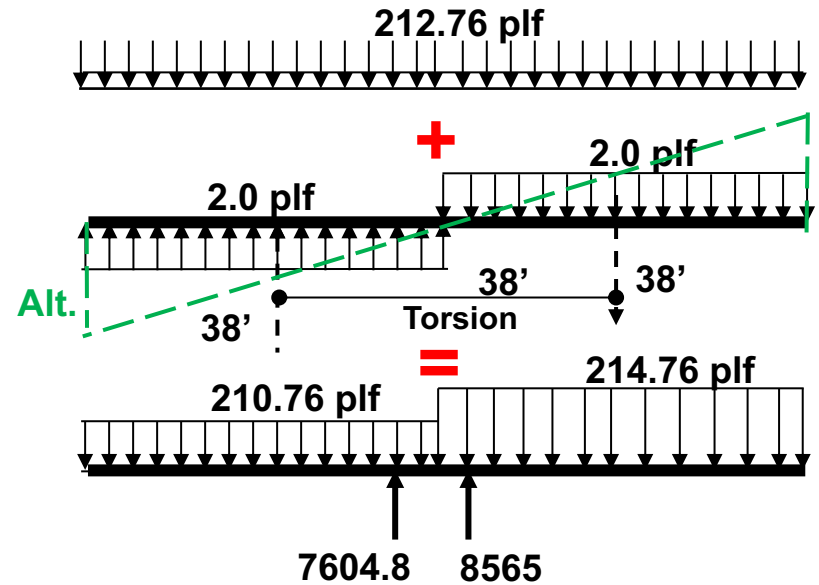
# Torsional Distribution-Not mandatory

(Question 4)  $p=1.3$ ,  $A_x=1.25$

Method **2B** will be used for diaphragm design  
(To answer questions 5 and 6)

Method **2A** will be used for all other checks

## Method 2A



## Using method 2B- $\rho=1.3$ , $A_x=1.25$ :

$F_T$  = Torsion forces only at corridor walls, gridlines 2 and 3

$M_{net} = 480.1(6 \text{ ft.}) = 2880.6 \text{ ft. lbs.}$  **Net moment**

The in-plane forces of the longitudinal walls applied at grid lines 1, 2, 3 and 4 are calculated:

$$F_{1,2,3,4} = 0.167(0.7)(1.3)(13 \text{ psf}) \left( \frac{10}{2} + 2 \right) (40) = 553.2 \text{ lbs.}$$

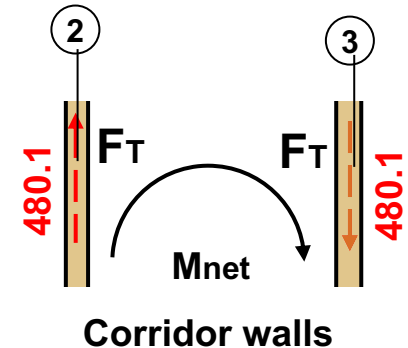
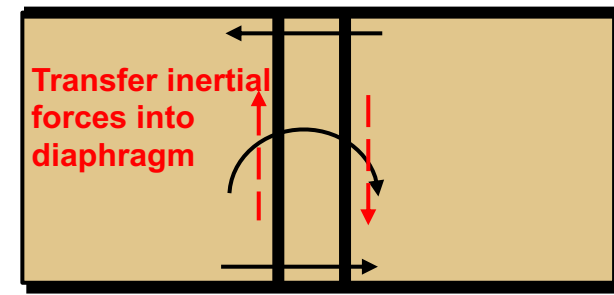
$$V_{net} = V_{base} - F_{1,2,3,4} = 12438.3(1.3) - 4(553.2) = 13957 \text{ lbs.}$$

$$W = \frac{13957}{76} = 183.65 \text{ plf uniform load}$$

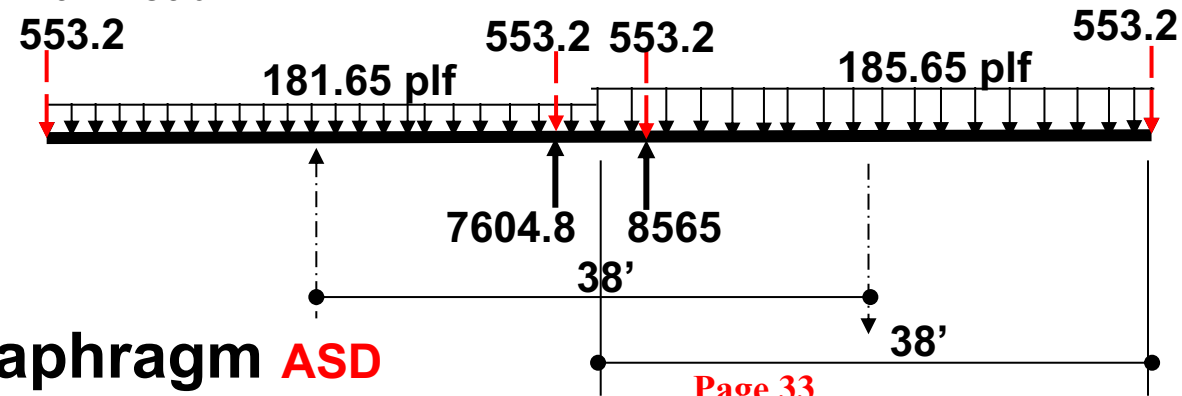
$$W_T = \frac{2880.6}{38(38)} = 2.0 \text{ plf: equivalent uniform torsional load acting as } M_{net}$$

$W_1 = 183.65 - 2.0 = 181.65 \text{ plf: uniform load minus torsional load=net uniform load left cantilever}$

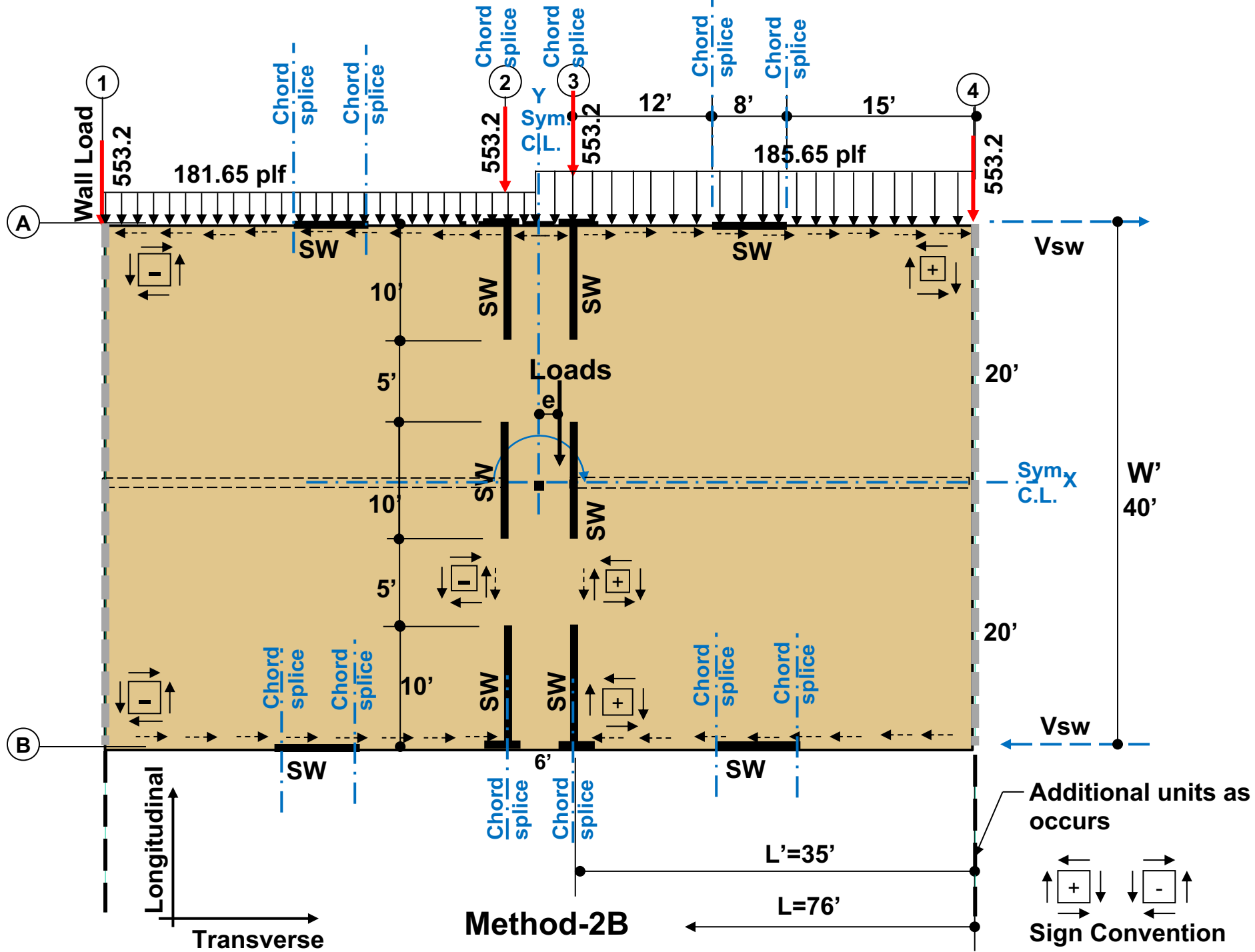
$W_2 = 183.65 + 2 = 185.65 \text{ plf}$   
Right cantilever

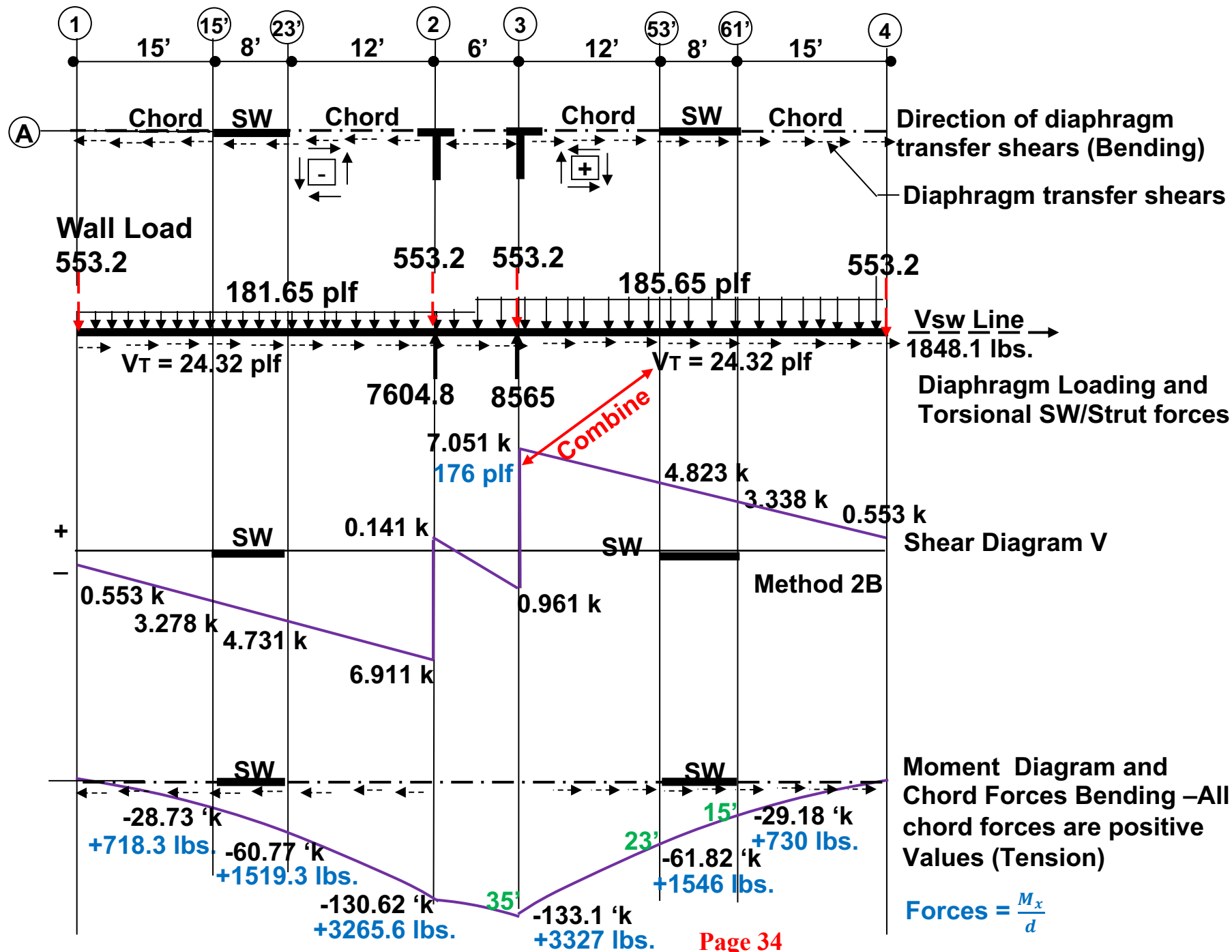


Wall Load



**Calculate Loads to Diaphragm ASD**





# Diaphragm Capacity-Wood Structural Panels

**Blocked**

**Table 4.2A Nominal Unit Shear Capacities for Wood-Framed Diaphragms**<sup>1,3,6,7</sup>

Sheathing Grade	Common nail Size	Minimum Fastener Penetration In Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal width Of nailed face At adjoining Panel edges and boundaries (in.)	A								B			
					Seismic								Wind			
					Nail spacing (in.) at boundaries (all cases), at continuous panel edges parallel to load (cases 3 & 4), and at all panel edges (cases 5 & 6).								Panel Edge Fastener Spacing (in.)			
					6	4	2 ½	2	6	4	2 ½	2	6	4	2 ½	2
					Nail spacing (in.) at other panel edges(cases 1, 2, 3 & 4)								Vw (plf)	Vw (plf)	Vw (plf)	Vw (plf)
					6	6	4	3	6	6	4	3				
					Vs (plf)	Ga (kips/in.)	Vs (plf)	Ga (kips/in.)	Vs (plf)	Ga (kips/in.)	Vs (plf)	Ga (kips/in.)				
					OSB	PLY	OSB	PLY	OSB	PLY	OSB	PLY				

Sheathing and Single floor	8d	1-3/8	7/16	3	570	11	9	760	7	6	1140	10	8	1290	17	12	800	1065	1595	1805
			15/32	2	540	13	9.5	720	7.5	6.5	1060	11	8.5	1200	19	13	755	1010	1485	1680
				3	600	10	8.5	800	6	5.5	1200	9	7.5	1350	15	11	840	1120	1680	1890
	10d	1-1/2	15/32	2	580	25	15	770	15	11	1150	21	14	1310	33	18	810	1080	1610	1835
				3	650	21	14	860	12	9.5	1300	17	12	1470	28	16	910	1205	1820	2060
			19/32	2	640	21	14	850	13	9.5	1280	18	12	1460	28	17	895	1190	1790	2045
				3	720	17	12	960	10	8	1440	14	11	1640	24	15	1010	1345	2015	2295

Roof framing-D.F. 1, E = 1,700,000 psi, roof joists @ 16" 0.c.

Unit torsional shear = 24.32 plf

V<sub>Max diaph</sub> = 176.3 + 24.3 = 200.6 plf.

200.6 plf < vs = 0.5(580) = 290 plf. o.k.

G<sub>a</sub> = 25, blocked



# Let's Take a 15 Minute Break



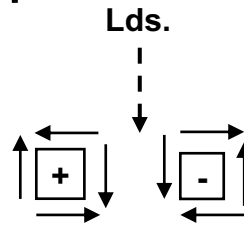
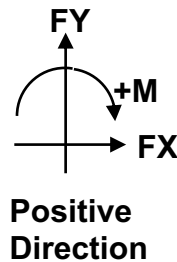
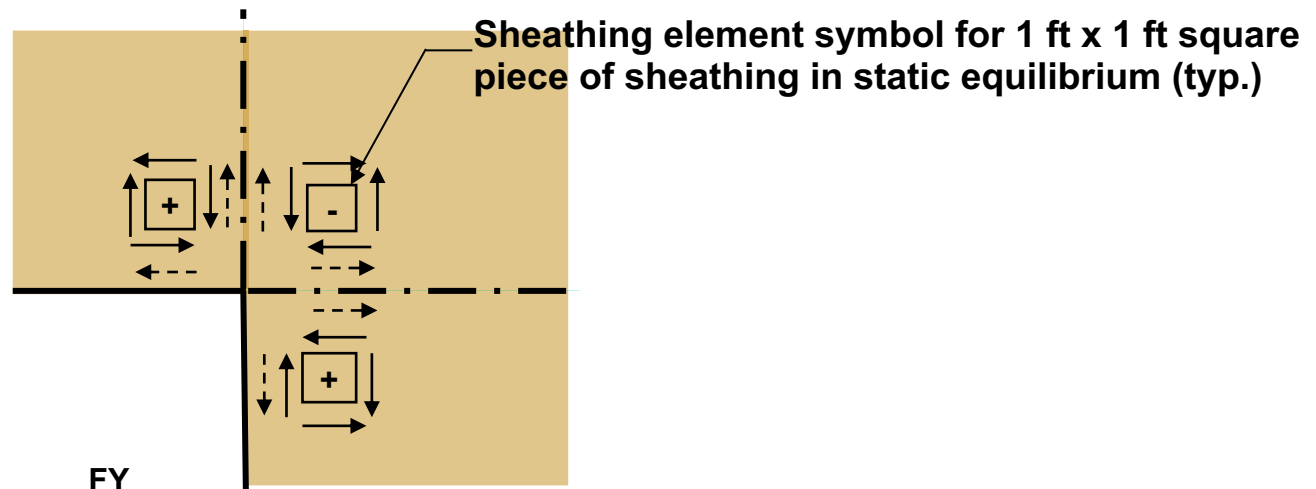
## Mass Timber Project

### **Part 3-Design Example (cont.):**

- **Maximum diaphragm chord force**
- **Diaphragm flexibility**
- **Story drift**
- **Torsional irregularity**

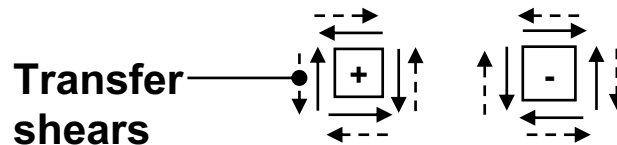
# Visual Aid-Shear

Page 36



Longitudinal Direction (shown)

## Shears Applied to Sheathing Elements

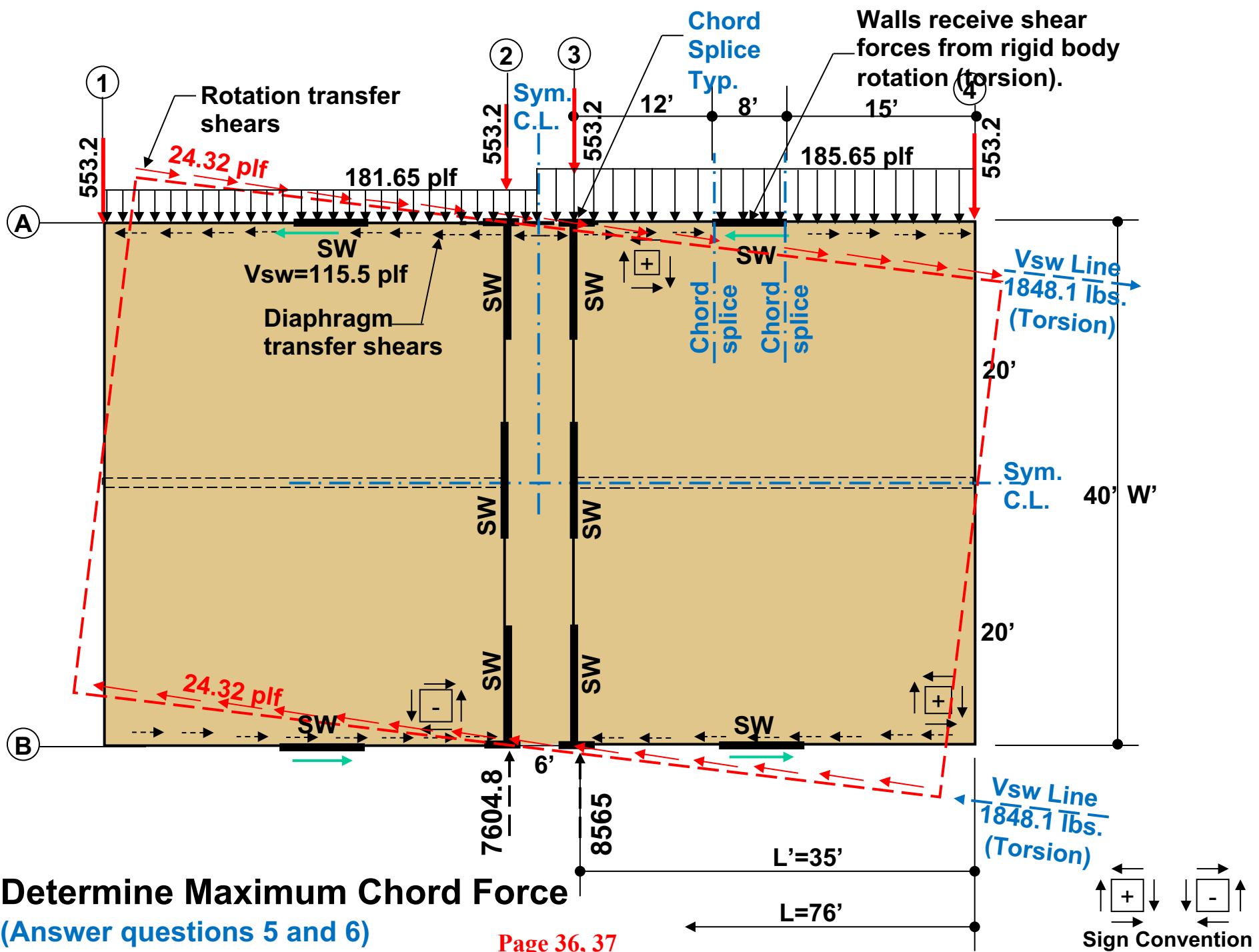


↑ Unit shear acting on sheathing element (plf)

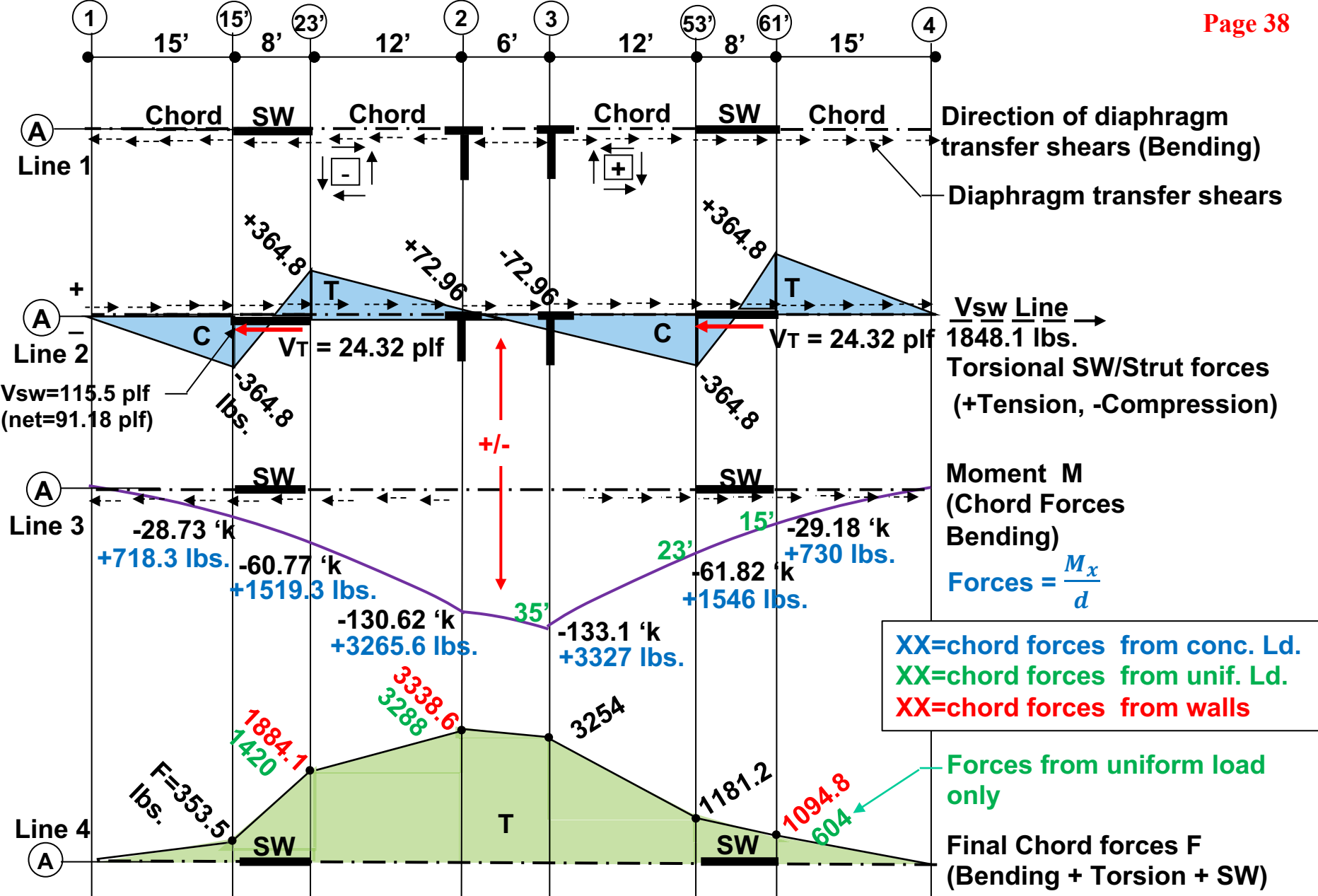
↑ Unit shear transferred from the sheathing element into the boundary element (plf)

## Shears Transferred Into Boundary Elements

The Visual Shear Transfer Method. How to visually show the distribution of shears through the diaphragm



**Determine Maximum Chord Force**  
 (Answer questions 5 and 6)



1. By inspection, the walls along the chord line affect the chord forces by a small amount, 364.8 lbs.
2. Calculations show that the conc. wall force at end of cantilever increase the chord force by +21% at the 15'splice diminishing to +9% increase at 23', and +1% at the support. Walls had a larger effect.

# Diaphragm Chords

## Diaphragm Deflection (ASD)

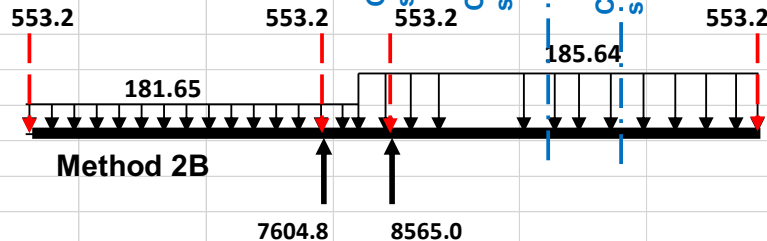
Splice Forces (Lbs.)			$\Sigma \delta_{slip}$ In.	v unif. plf	v conc. plf	Ga k/in.	L' Ft.	W' Ft.	$\delta_{Diaph Uni}$ In.	$\delta_{Diaph conc}$ In.	Total $\delta$ In.	
F 15	F 23	F 35										
1094.3	1180.9	3253.7	0.072	186.75	13.83	25.0	35.00	40.00	0.225	0.02	0.248	Rt. Cantilever

Nails Req'd=	4.84	5.23	14.40
Use Nails=	8	16	24
Slip=	0.023	0.013	0.023

EA= 28050000, (2)2x6

includes effects of sw's along chord line

Wall Load



W2	W1
183.65	183.65
2.0	-2.0
185.64	181.65

## Diaphragm Deflection (ASD)

353.6	1884.0	3338.5	0.070	183.26	13.83	25.0	35.00	40.00	0.219	0.02	0.243	Lft. Cantilever
1.56	8.34	14.77										
8	16	24										
0.008	0.020	0.024										

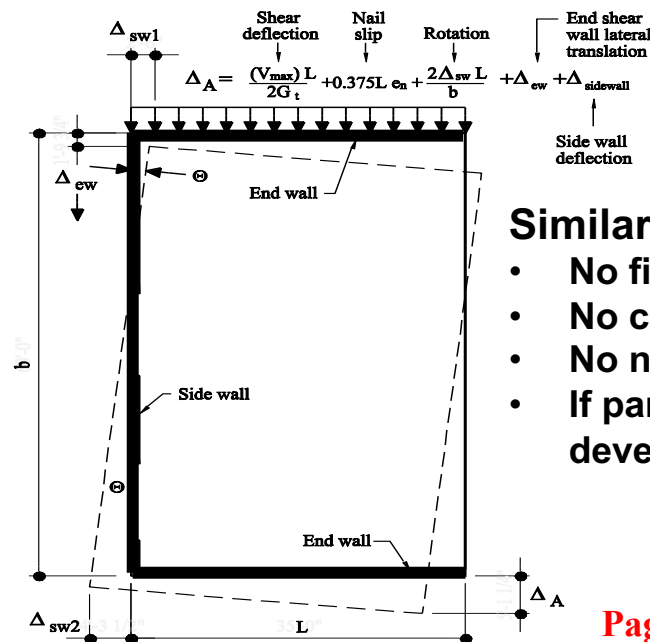
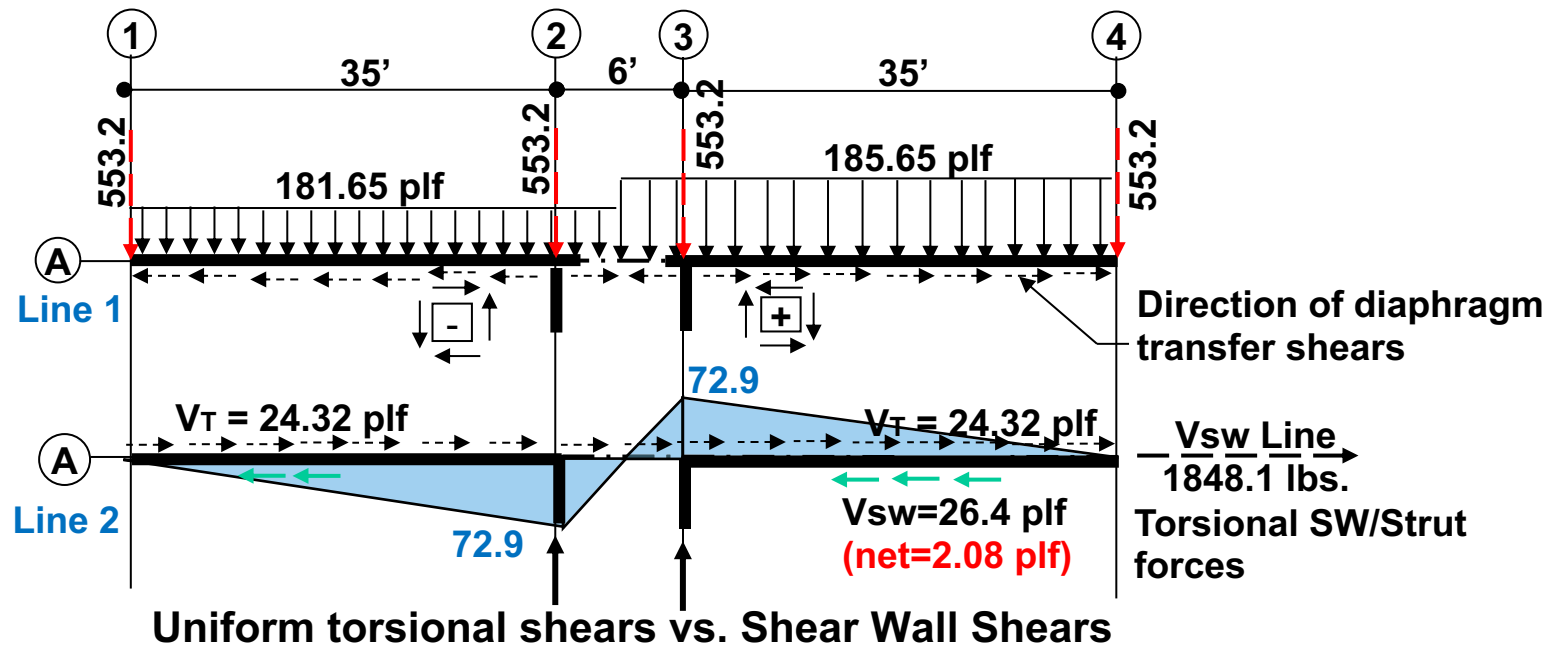
Maximum chord force = 3338.5 lbs.

Using (2)2x6 DF-Larch No.1 wall top plates as the diaphragm chords: 2015 NDS Supplement Table 4A  $F_t = 675$  psi,  $F_c // = 1500$  psi. Only one 2x6 plate resists the chord forces due to the nailed splice joint.

$$f_t = \frac{F_{chord}}{(1)2x6}, \text{ Number of nails} = \frac{F_{chord}}{226}, \text{ where 226 lbs. is adjusted lateral design value, } Z' \text{ (ASD), for 16d nails (face nailed).}$$

Compression stresses OK by inspection. Chords braced about both axes.

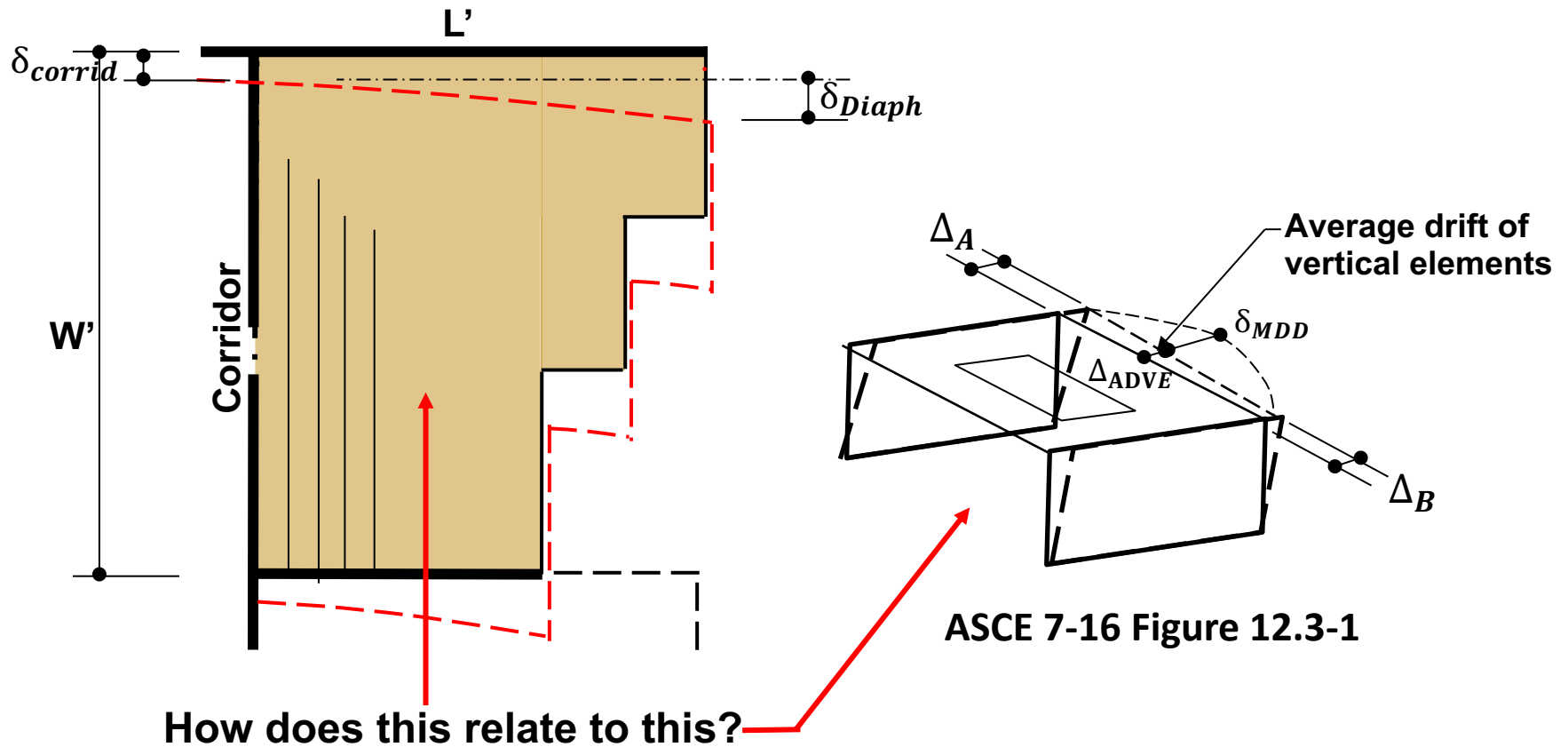
# Check for Effects of Full Length Shear Walls on Chord Forces



## Similar to APA Example

- No fixity at support
- No chord bending
- No net rotational shears
- If partial length end walls, will develop strut forces

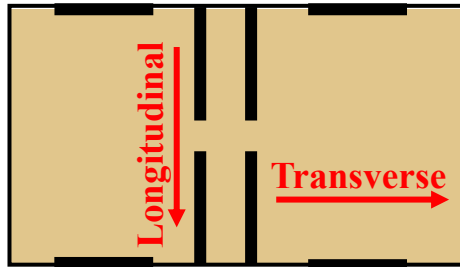
## Diaphragm Flexibility, $\rho=1.0, A_x=1.25$





# Analysis Flow

## Longitudinal Design



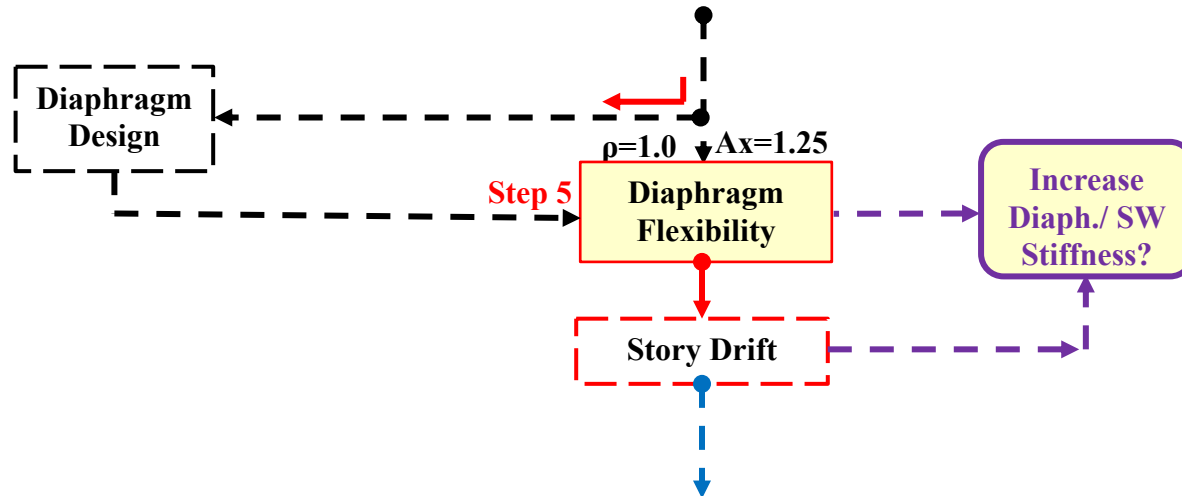
Example Plan

### Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



## Check Diaphragm Flexibility

Seismic-  $\rho=1.0$ ,  $A_x=1.25$

# A matter of Stiffness

## Seismic:

ASCE 7-16 Section 12.3.1- Diaphragm flexibility-The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force resisting system.

## Wind:

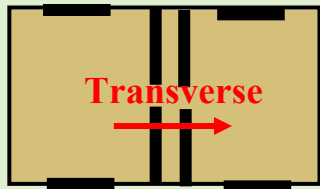
ASCE 7-16 Section 27.4.5- Diaphragm flexibility-The structural analysis shall consider the relative stiffness of diaphragms and vertical elements of the MWFRS.

**Flexible structures are susceptible to damage from wind or seismic forces**

**Can require engineering judgement**

# ASCE7-16 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 seismic lateral force resisting system.



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

Yes

Start  
Is diaphragm untopped steel decking or Wood Structural Panels

No

Is diaphragm concrete slab or concrete filled steel deck ?

Yes

No

Is span to depth ratio  $\leq 3$  and having no horizontal irregularities ?

Yes

No

Idealize as rigid

Idealize as flexible

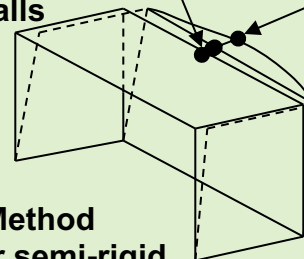
Yes

Structural analysis must explicitly include consideration of the stiffness of the diaphragm (i.e. semi-rigid modeling), or calculated as rigid in accordance with 2018 IBC Section 1604.4 or ASCE 7-16 Section 12.3.1.3.

No

Envelope Method Allowed for semi-rigid modelling

Average drift of walls



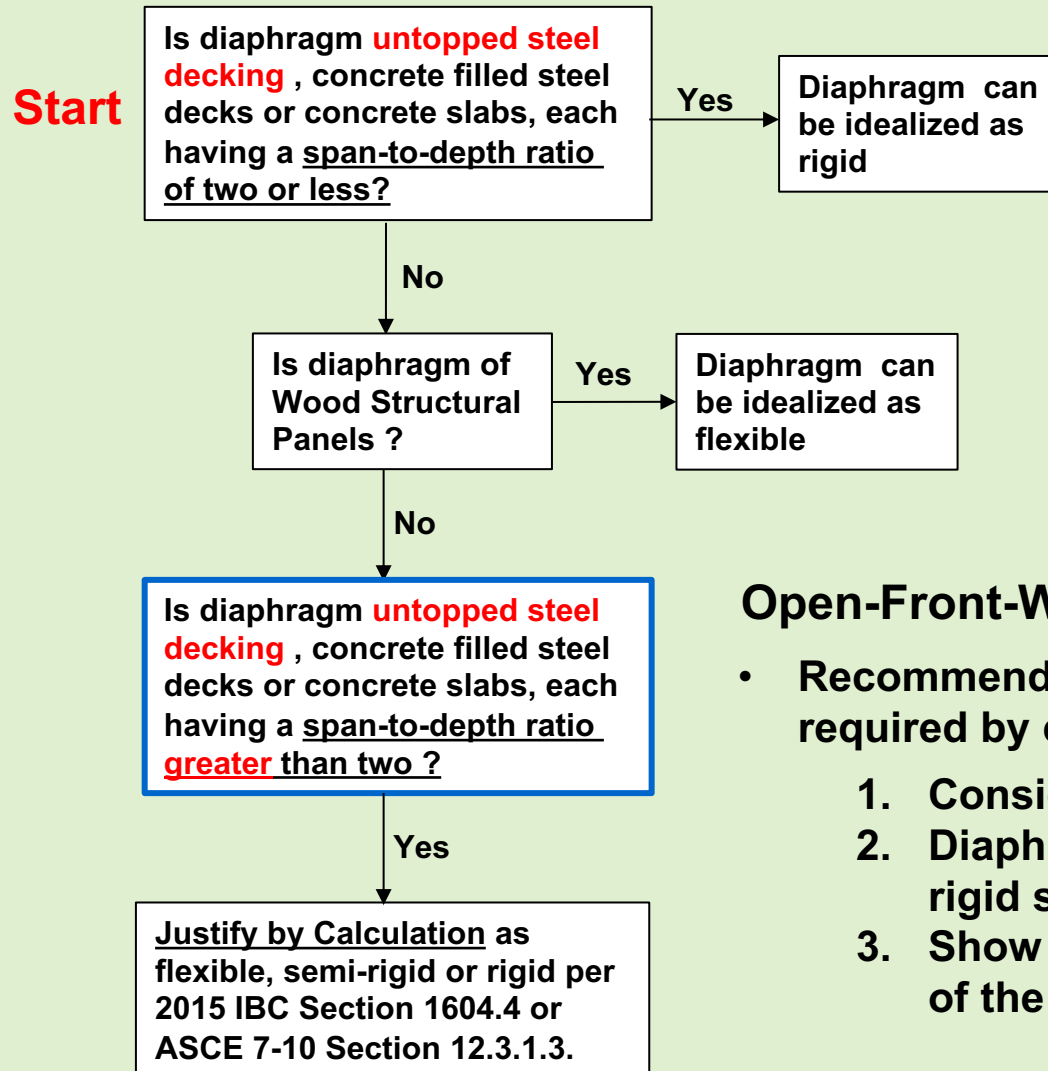
(Could apply to CLT or Heavy timber diaphragms)

Maximum diaphragm deflection

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

# ASCE7-16, Sections 26.2 and 27.4.5 Diaphragm Flexibility **Wind**

ASCE 7-16 Section 27.4.5-Diaphragm flexibility-The structural analysis shall consider the stiffness of diaphragms and vertical elements of the MWFRS



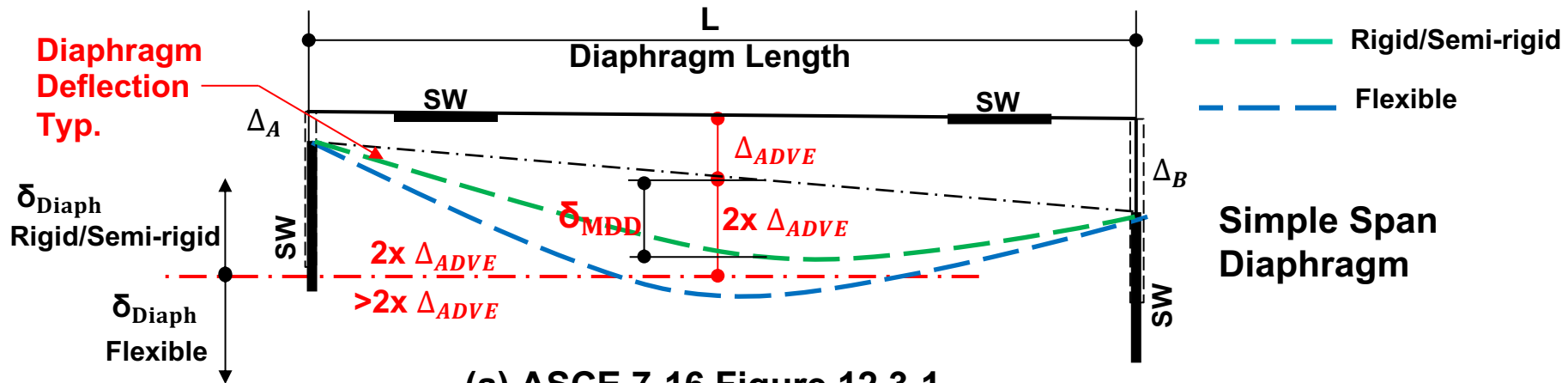
## Open-Front-Wind

- Recommend Following SDPWS 4.2.5.2 (not required by code):
  1. Considered good engineering practice
  2. Diaphragm should meet semi-rigid or rigid stiffness requirements
  3. Show that the resulting drift at the edges of the structure can be tolerated.

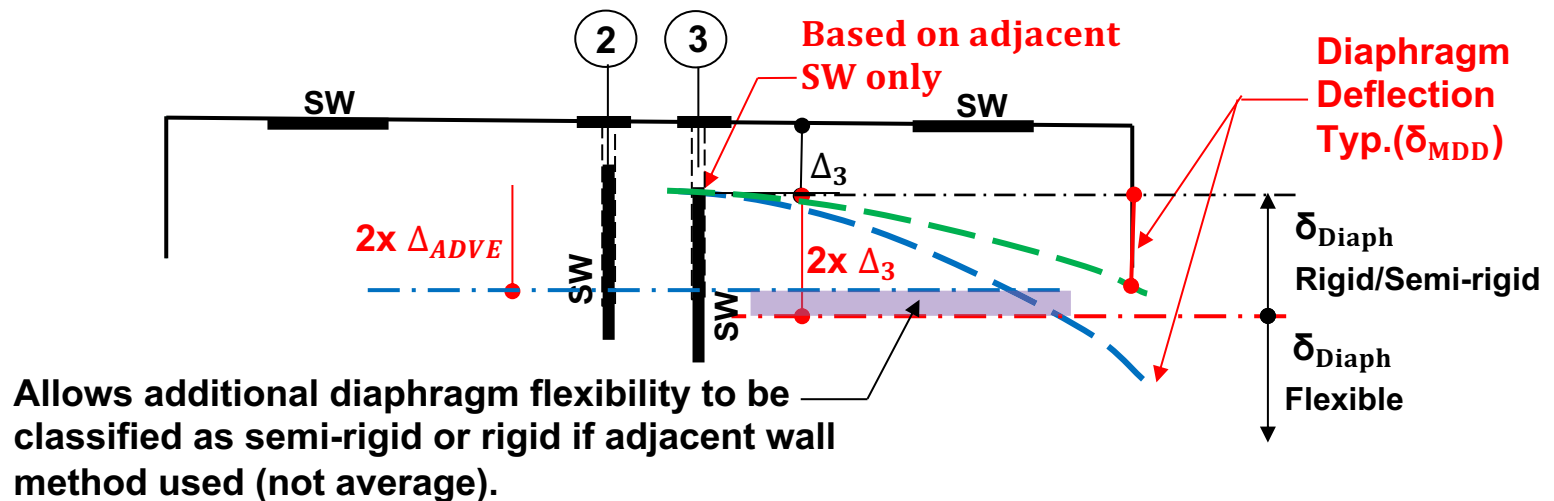
# Determination of Cantilever Diaphragm Flexibility (Question 3):

Page 42

Use the drift of adjacent wall line supporting the Cantilever

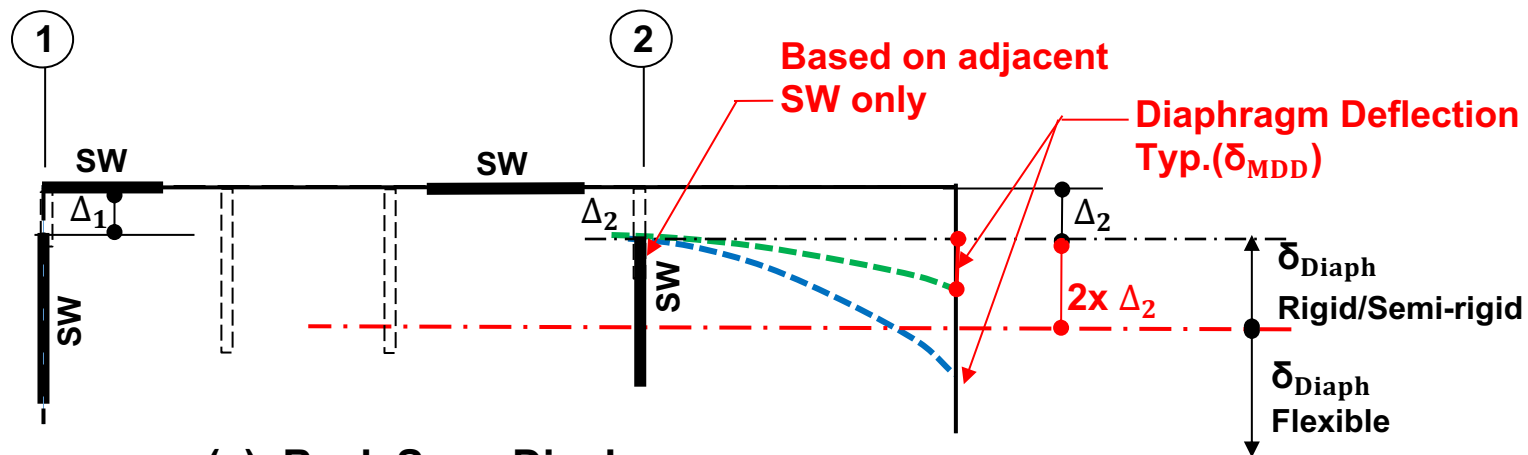


(a) ASCE 7-16 Figure 12.3-1

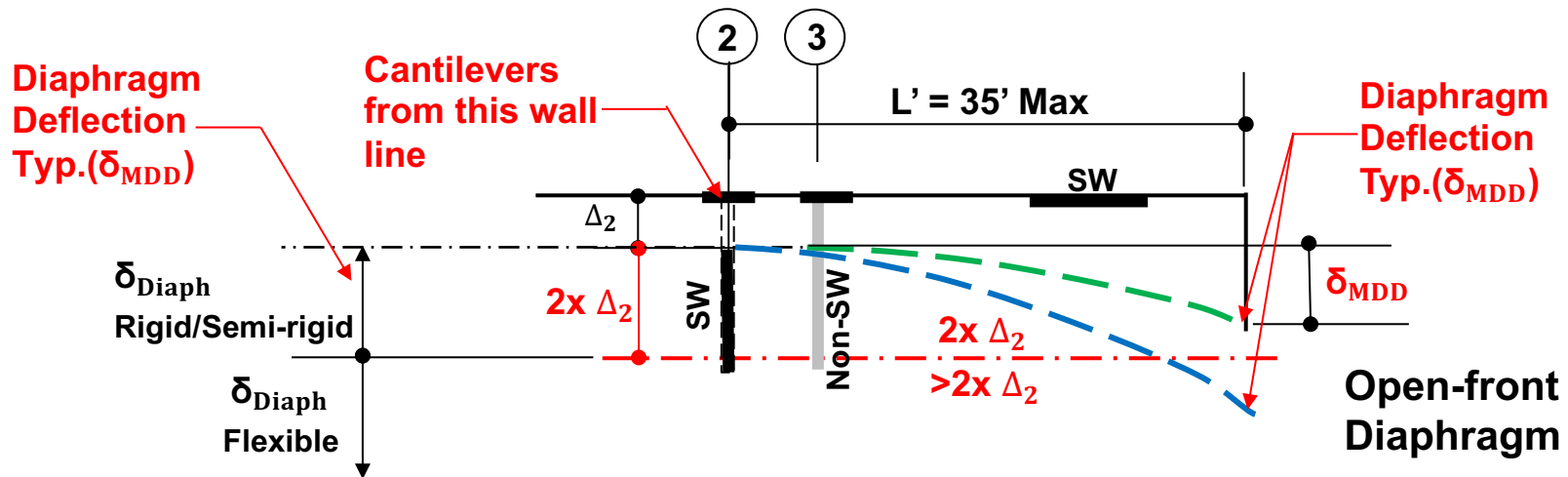


(b) Corridor Walls Only

Preferred Method – Simplifies Check



(c) Back Span Diaphragm  
SDPWS Figure 4A Case (b)



(d) Diaphragm flexibility Shear Wall One Side

# Cantilever Diaphragm Deflection Equations (Question 2):

## Three-term equation for uniform load:

$$\delta_{Diaph\ Unif} = \frac{3vL'^3}{EAW'} + \frac{0.5vL'}{1000G_a} + \frac{\Sigma x\Delta_c}{W'}$$

## Four-term equation for uniform load:

$$\delta_{Diaph\ Unif} = \frac{3vL'^3}{EAW'} + \frac{0.5vL'}{Gvtv} + 0.376 L' e_n + \frac{\Sigma x\Delta_c}{W'}$$

## Three-term equation for point load:

$$\delta_{Diaph\ Conc} = \frac{8vL'^3}{EAW'} + \frac{vL'}{1000G_a} + \frac{\Sigma x\Delta_c}{W'}$$

## Four-term equation for point load:

$$\delta_{Diaph\ Conc} = \frac{8vL'^3}{EAW'} + \frac{vL'}{Gvtv} + 0.75 L' e_n + \frac{\Sigma x\Delta_c}{W'}$$

For method 2B, the maximum diaphragm deflection is equal to the sum of the uniform load deflection plus the concentrated load deflection:

EA chords = 28,050,000 lbs., 2-2x6 wall top plate.

Where:

$L'$  = cantilever diaphragm length, ft

$W'$  = cantilever diaphragm width, ft

$E$  = modulus of elasticity of diaphragm chords, psi

$A$  = area of chord cross-section, in.<sup>2</sup>

$v_{\max}$  = induced unit shear at the support from a uniform applied load, lbs/ft

$G_a$  = apparent diaphragm shear stiffness from nail slip and panel shear deformation, kips/in

$Gvtv$  = Panel rigidity through the thickness

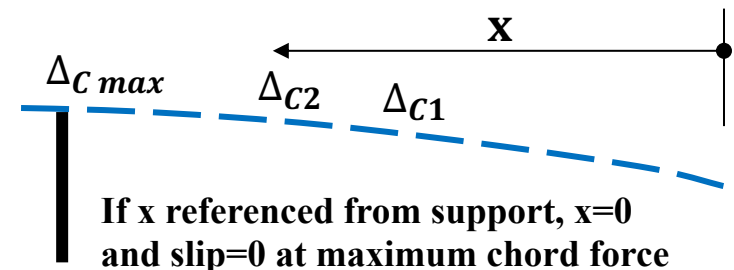
$x$  = distance from chord splice to the free edge of the diaphragm, ft

$\Delta_c$  = diaphragm chord splice slip, in.

$\delta_{Diaph\ Unif}$  = calculated deflection at the free edge of the diaphragm, in.

$e_n$  = Nail slip per SDPWS C4.2.2D for the load per fastener at  $v_{\max}$

$\delta_{Diaph\ Conc}$  = calculated deflection at the free edge of the diaphragm, in.



Longitudinal Loading $e=4.75'$ , $T = 84403$ ft. lbs., $p=1.0$ , $Ax=1.25$										
Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv		F <sub>T</sub>	Fv+F <sub>T</sub>
2	43.54		3		130.63	391.89	8884.5		-527.7	8356.8
3	43.54		3		130.63	391.89	8884.5		527.7	9412.2
A		25.14		20	502.74	10054.73			2030.9	2030.9
B		25.14		20	502.74	10054.73			-2030.9	-2030.9
Σ	87.09	50.27			J=	20893.23	17769			

Walls at Grid Corridor lines A & B Walls

Diaphragm Deflection (STR)											Rt. Cantilever	
Splice Forces (Lbs.)			Σδ <sub>slip</sub> In.	v unif. plf	v conc. plf	Ga k/in.	L' Ft.	W' Ft.	δDiaph Unif In.	δDiaph conc In.	Total δ In.	
F 15	F23	F35										
1064.6	1159.7	3533.5	0.075	233.22	0.00	25.0	35.00	40.00	0.265	0.00	0.265	
ails Req'd=	4.71	5.13	15.64									
Use Nails =	8	16	24									
Slip=	0.023	0.012	0.025									
EA= 28050000, (2)2x6												
includes effects of sw's along chord line												
<div> <div>Chord splice</div> <div>Chord splice</div> <div>Chord splice</div> <div>Chord splice</div> </div> <div> <div>231.61</div> <div>236.00</div> </div> <div> <div>Method 2A</div> <div>8356.8</div> <div>9412.2</div> </div>												
Diaphragm Deflection (STR)											Lft. Cantilever	
250.6	1932.4	3626.7	0.073	229.38	0.00	25.0	35.00	40.00	0.260	0.00	0.260	
1.11	8.55	16.05										
8	16	24										
0.005	0.021	0.026										

Flexibility and Drift



## Diaphragm Deflection-Method 2A, $\rho=1.0$ , $A_x=1.25$

$$\delta_{Diaph\ Unif} = \frac{3v_{max}L'^3}{EAW'} + \frac{0.5v_{max}L'}{1000G_a} + \frac{\Sigma A_C X_C}{W'} \quad \text{Three-term equation for uniform load}$$

Wall displacements from Spreadsheet:

$$\delta_{Diaph\ left} = 0.26'', \delta_{Diaph\ right} = 0.265''$$

Deflection at grid line 3 = 0.216''

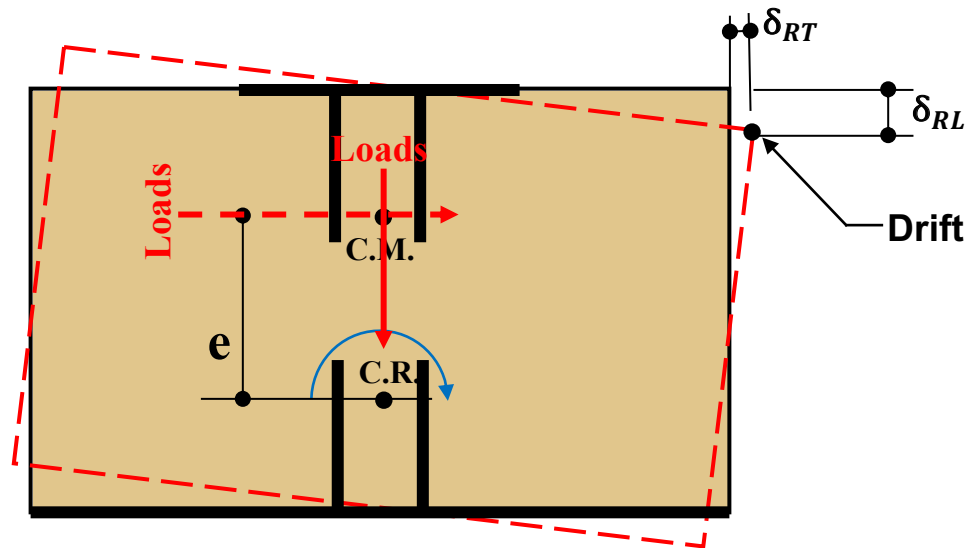
$$2 \times \Delta_3 = 0.432''$$

0.265'' < 0.432''  $\therefore$  Diaphragm can be idealized as Rigid

## Diaphragm Flexibility – Wind

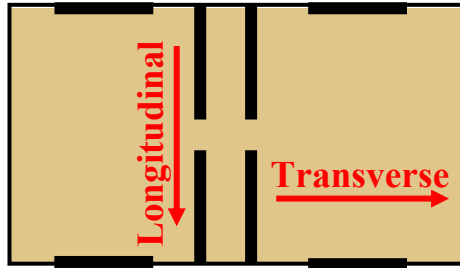
- ASCE 7-16, Chapter 27, Section 27.5.4-DIAPHRAGM FLEXIBILITY-requires that the structural analysis shall consider the stiffness of diaphragms and vertical elements of the main wind force resisting system (MWFRS).
- Section 26.2 - Definitions, DIAPHRAGM, diaphragms constructed of WSP are permitted to be idealized as flexible.
- There is no drift limit requirement in the code for wind design.

# Story Drift, $\rho=1.0$ , $A_x=1.25$



# Analysis Flow

## Longitudinal Design



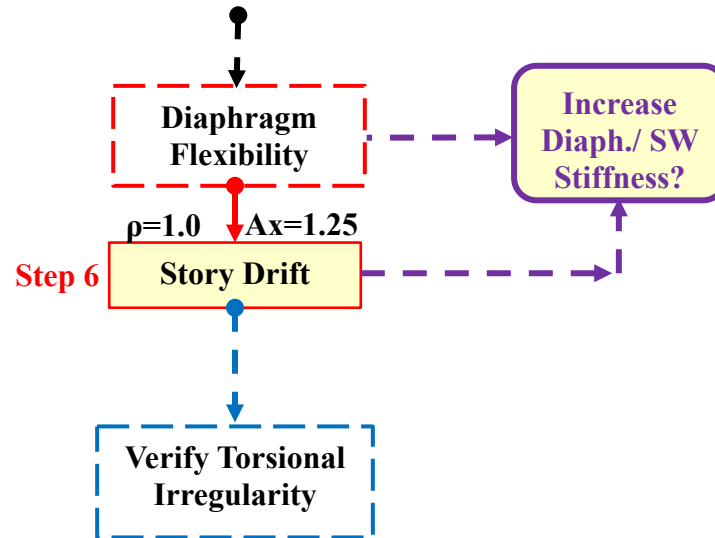
Example Plan

### Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



## Check Story Drift

Seismic-  $\rho=1.0$ ,  $A_x=1.25$

The diagram illustrates the relationship between the center of mass (C.M.) and the center of rigidity (C.R.) in a beam cross-section. A vertical line represents the beam's axis. A red dashed line indicates the direction of a load. The center of mass (C.M.) is marked with a black dot, and the center of rigidity (C.R.) is marked with a black dot. The distance between them is labeled  $\delta_T$ . The word "Translation" is written below the beam, indicating the movement of the beam's axis under the load.

The diagram illustrates a building frame under seismic loads. A central vertical column is labeled 'C.M.' (Center of Mass) and 'C.R.' (Center of Rotation). A red arrow labeled 'Loads' points downwards from the C.M. A blue curved arrow labeled 'Rotation' indicates the direction of rotation around the C.R. The frame is shown in a deformed state, with dashed red lines representing the original shape. On the right side, various drifts are indicated:  $\delta_{RH}$ ,  $\delta_{RV}$ ,  $\delta_{RT}$ ,  $\delta_B$ ,  $\delta_T$ , and  $\delta_{RL}$ . The word 'Drift' is written next to the vertical drifts. A '+' sign is at the top center, and a '-' sign is at the bottom center.

## Rotation

$$\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (12.8-15)$$



## Drift-Method 2A $\rho=1.0, A_x=1.25$

$$\text{Drift } \Delta = \delta_{Diaph} + \delta_{Rotation} + \delta_{Translation}$$

$$\delta_2 = 8.357 \text{ k} / 43.54 \text{ k/in} = 0.192 \text{ in,}$$

$$\delta_3 = 9.412 \text{ k} / 43.54 \text{ k/in} = 0.216 \text{ in}$$

$$\delta_A = 2.031 \text{ k} / 25.14 \text{ k/in} = 0.081 \text{ in,}$$

$$\delta_B = -2.031 \text{ k} / 25.14 \text{ k/in} = -0.081 \text{ in}$$

$$\Delta_{Diaph} = 0.265''$$

$$\Delta_{Average} = 0.204'' \text{ (Translation)}$$

$$\delta_{RL} = \frac{2\Delta_{SW A,B}(L' + 3')}{W'} = \frac{2(0.081)(35' + 3')}{40} = 0.154'' , \delta_{RT} = 0.081''$$

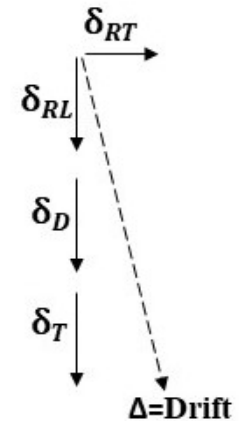
$$\text{Drift } \Delta = \sqrt{(\delta_T + \delta_D \pm \delta_{RL})^2 + (\delta_{RT})^2}$$

$$\text{Drift } \Delta_4 = \sqrt{(0.204 + 0.265 + 0.154)^2 + (0.081)^2} = 0.628''$$

$$\text{Drift } \Delta_1 = \sqrt{(0.204 + 0.26 - 0.154)^2 + (0.081)^2} = 0.320''$$

$$C_d = 4, I_e = 1$$

$$\delta_M = \frac{C_d \delta_{max}}{I_e} = \frac{4(0.628)}{1} = 2.51''$$



$\delta_{RT}$  = Transverse component of rotation

$\delta_{RL}$  = Longitudinal component of rotation

$\delta_D$  = Diaphragm displacement

$\delta_T$  = Translational displacement

**Table 12.12-1 Allowable Story Drift,  $\Delta_a$**

Structure	Risk Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, four stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and <b>exterior wall systems</b> that have been designed to accommodate the story drifts.	<b>0.025h<sub>sx</sub></b>	0.020h <sub>sx</sub>	0.015h <sub>sx</sub>
Masonry cantilever shear wall structures	0.010h <sub>sx</sub>	0.010h <sub>sx</sub>	0.010h <sub>sx</sub>
Other masonry shear wall structures	0.007h <sub>sx</sub>	0.007h <sub>sx</sub>	0.007h <sub>sx</sub>
<b>All other structures</b>	<b>0.020h<sub>sx</sub></b>	0.015h <sub>sx</sub>	0.010h <sub>sx</sub>

- Depends on the non-structural components and detailing.
- Most sheathed wood framed walls can undergo the 2.5% drift level while providing life safety performance at the seismic design level.
- 0.025h<sub>sx</sub> limit - interior walls, partitions, ceilings, and exterior walls can accommodate the higher story drift limit. The selection of the higher 2.5% drift limit should be taken only with consideration of the non-structural wall and window performance.
- Otherwise, the 2% drift limit requirements should be used.

$$0.025h_{sx} = 0.025(10)(12) = 3.0'' > 2.51'' \therefore \text{drift O.K.}$$

$$0.02h_{sx} = 0.02(10)(12) = 2.4'' < 2.51'' \therefore \text{drift not O.K. for 2\% drift}$$

## Page 48

**Additional stiffness must be provided in either the diaphragm or in the shear walls:**

**a. Diaphragms-**

- Increasing nail size, spacing and/or sheathing thickness can increase shear capacity but it will not, in most cases, increase the diaphragm stiffness, **if using the 3 term eq.**
- The largest deflection comes from shear deflection and nail slip.
- SDPWS Table 4.2A shows that the apparent shear stiffness diminishes as you decrease the boundary nail spacing from a 6/6/12 nailing pattern until you get to a 2/3/12 nailing pattern.
- **If using plywood, switch to OSB which has a higher  $G_a$**

**Table 4.2A Nominal Unit Shear Capacities for Wood-Framed Diaphragms Blocked**

Sheathing Grade	Common nail Size	Minimum Fastener Penetration In Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal width Of nailed face At adjoining Panel edges and boundaries (in.)	A Seismic				B Wind							
					Nail spacing (in.) at boundaries (all cases), at continuous panel edges parallel to load (cases 3 & 4), and at all panel edges (cases 5 & 6).								Panel Edge Fastener Spacing (in.)			
					6		4		2 ½		2		6	4	2 ½	2
					Nail spacing (in.) at <b>red panel edges</b> (cases 1, 2, 3 & 4)											
					6		6		4		3		6	6	4	3
Vs Ga (plf) (kips/in.)		Vs Ga (plf) (kips/in.)		Vs Ga (plf) (kips/in.)		Vs Ga (plf) (kips/in.)		Vw (plf)	Vw (plf)	Vw (plf)	Vw (plf)					
					OSB PLY		OSB PLY		OSB PLY		OSB PLY					

Sheathing and Single floor	8d	1-3/8	7/16	3	570 11 9	760 7 6	1140 10 8	1290 17 12	800	1065	1595	1805
			15/32	2	540 13 9.5	720 7.5 6.5	1060 11 8.5	1200 19 13	755	1010	1485	1680
	10d	1-1/2	15/32	2	580 25 15	770 15 11	1150 21 14	1310 33 18	810	1080	1610	1835
				3	600 10 8.5	800 6 5.5	1200 9 7.5	1350 15 11	840	1120	1680	1890
				3	650 21 14	860 12 9.5	1300 17 12	1470 28 16	910	1205	1820	2060
			19/32	2	640 21 14	850 13 9.5	1280 18 12	1460 28 17	895	1190	1790	2045
	3	720 17 12	960 10 8	1440 14 11	1640 24 15	1010	1345	2015	2295			



- b. **Shear walls**- Contrary to the diaphragm, decreasing the nail spacing on the shear walls would increase the wall stiffness, reference SDPWS Table 4.3A. The apparent shear stiffness,  $G_a$ , increases as the nail spacing decreases.
- c. **Other options** to increase stiffness:
- Increase the wall lengths.
  - Increase the number of shear walls in the lateral line of force-resistance.
  - Apply sheathing to both sides of the walls at grid lines A & B or decrease nail spacing.
  - Decrease nail spacing at corridor walls.
  - Increase the size of the hold downs(**with smaller  $\Delta a$** ) to lessen rod elongation and wall rotation.
  - Increase the number of boundary studs (decrease bearing perpendicular to grain stresses, crushing).
  - Add additional interior shear walls to decrease forces on other shear walls.
- d. Calculation Method: A final option which may increase the calculated system stiffness and reduce the deflections is to **use the four-term deflection equation** for the **shear wall and diaphragm** deflections to avoid introducing an artificial bias in the results by selectively combining three-term and four-term deflection calculations.

## Solution for 2% drift issue: Page 50

Following option (d), the 2% drift limit can potentially be achieved by using the four-term deflection equation, which reduces diaphragm deflection and drift, as noted below.

$$\delta_{\text{Diaph Unif}} = \frac{3vL'^3}{EAW'} + \frac{0.5vL'}{Gvtv} + 0.376 L' e_n + \frac{\Sigma x \Delta_c}{W'}$$

Where:

$$e_n = \left( \frac{V_n}{769} \right)^{3.276} = \left( \frac{116.6}{769} \right)^{3.276} = 0.002 \text{ in} \quad \text{SDPWS Table C4.2.2D}$$

where 116.6 is max. load per nail, 10d nails, dry lumber assumed.

$Gvtv = 35000 \text{ lb/in depth, 4-ply}$

SDPWS Table C4.2.2A

$v = 233.2 \text{ plf}$

$$\frac{2\Sigma x \Delta_c}{W'} = \frac{2[15(0.023) + 23(0.012) + 35(0.025)]}{40} = 0.075 \text{ in}$$

$$\delta_{\text{Diaph Unif}} = \frac{3(233.2)35^3}{28050000(40)} + \frac{0.5(233.2)35}{35000} + 0.376(35)0.002 + 0.075 = 0.245 \text{ in}$$

$$\text{Drift } \Delta_4 = \sqrt{(0.204 + 0.245 + 0.153)^2 + (0.081)^2} = 0.608 \text{ in}$$

$\delta_M = \frac{C_d \delta_{\max}}{I_e} = \frac{4(0.608)}{1} = 2.434 \text{ in.} \approx 2.4 \text{ in.}$  Close enough to comply with the 2% drift limitation. Drift can also be improved if  $\rho$  or  $A_x$  decreases (See Section 7.6.1).

# Check for Wind Drift

## Simplified Procedure Chapter 28, Part 1 Low-rise Buildings, Enclosed

ASCE 7-16 Section 2.4 ASD LC 0.6D+0.6W

Risk Category II,  $V_{ult}=115$  MPH

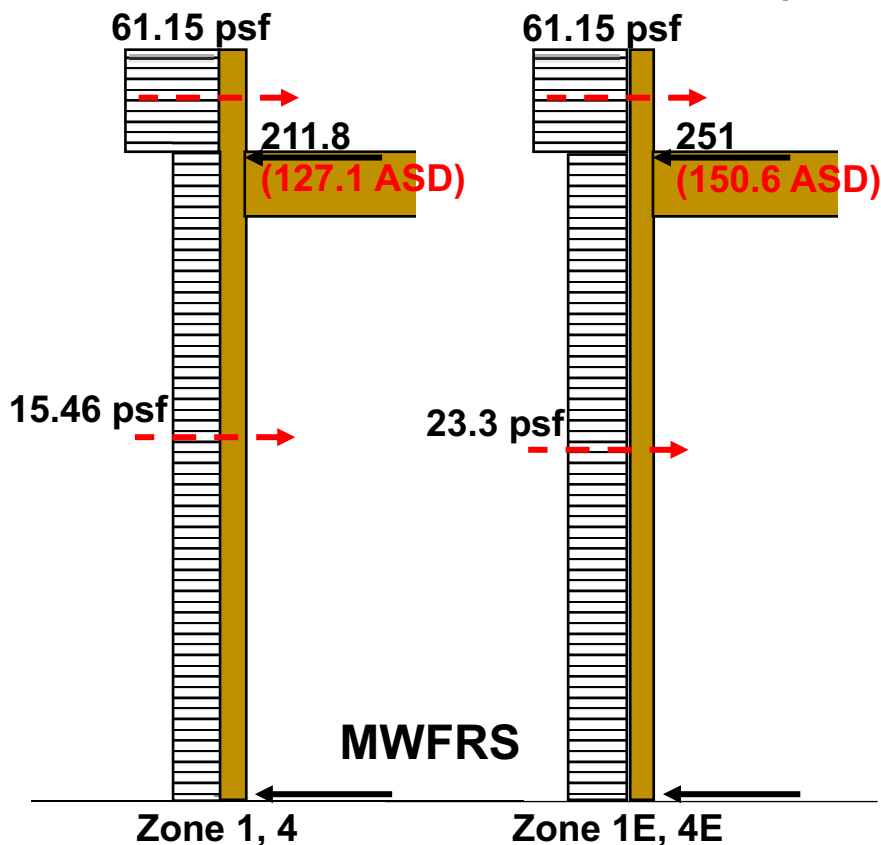
Exposure C

$P=Qh[(GC_{pf})-(GC_{pi})]$  MWFRS

Figure 26.5-1B

26.7, 26.7.2

28.3.1 Design  
wind  
pressure



$K_d=0.85$

$GC_{pi}=+/-0.18\left(\frac{15}{z_g}\right)^{\left(\frac{2}{\alpha}\right)}$

$K_z=2.01\left(\frac{15}{z_g}\right)^{\left(\frac{2}{\alpha}\right)}$

$K_z=0.78 @ h=10'$

$Q_h=0.00256K_zK_{zT}K_dV^2=22.4$  psf

Wind directionality factor 26.8

Internal pressure coeff. 26.13

Velocity pressure exp. coeff. 26.10-1

26.10-1

Figure 28.3-1

Surface	1	4	1E	4E
$GC_{pi}$	0.4	-0.29	0.61	-0.43
P (psf)	8.96	6.5	13.66	9.63

Parapet

15.46 psf

23.3 psf

$P_p=Q_p(GC_{pn})$

28.3-2

$K_z=0.85 @ 12'$  Top of parapet

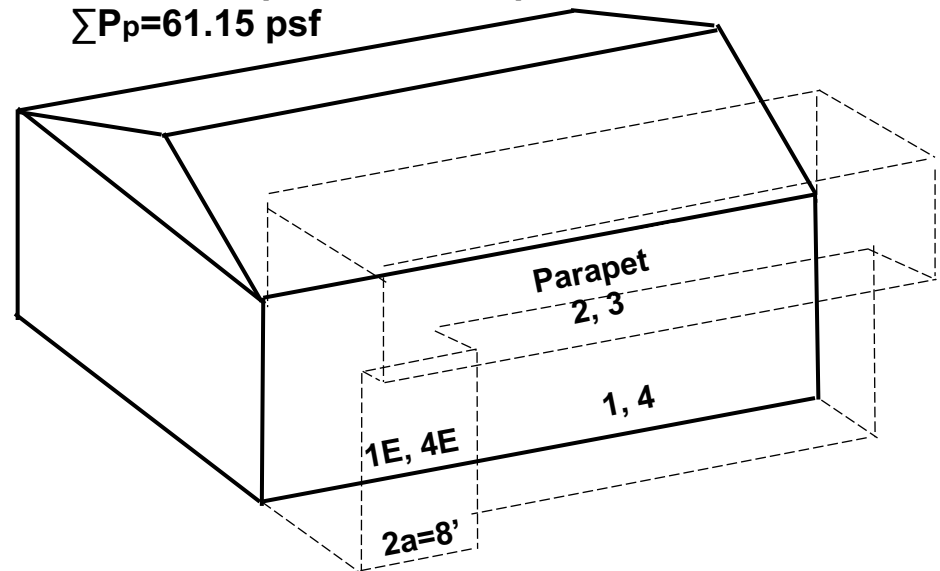
$Q_p=24.46$  psf

$GC_{pn} ww=1.5, GC_{pn} lw=-1.0$

28.3.2

$P_{pw}=36.69$  psf,  $P_{pl}=24.46$  psf

$\sum P_p=61.15$  psf



# Rigid Diaphragm Analysis (ASD)

Wind  $V_{ult}=115$  MPH

Requires Input

## Longitudinal Loading

Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv	FT	Fv+FT
2	43.54		3		130.63	391.89	4923.8	-40.0	4883.8
3	43.54		3		130.63	391.89	4923.8	40.0	4963.8
A		25.14		20	502.74	10054.72756		153.8	153.8
B		25.14		20	502.74	10054.72756		-153.8	-153.8
Σ	87.09	50.27			J=	20893.23102	9847.6		

Loads  
↓

$\delta_{SW}$	Rho=	1	2a=	8
0.112	Ax=	1	Net=	23.5
0.114				
0.0061	Fy=	9847.6	W1,4=	127.1
-0.006	e=	34	W1E,4E=	150.6
			T=	6392.0

## Transverse Loading

Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv	FT	Fv+FT
2	43.54		3		130.63	391.89		18.8	18.8
3	43.54		3		130.63	391.89		-18.8	-18.8
A		25.14		20	502.74	10054.72756	4923.8	72.4	4996.2
B		25.14		20	502.74	10054.72756	4923.8	-72.4	4851.4
Σ	87.09	50.27			J=	20893.23102	9847.6		

Loads  
→

0.000				
0.000				
0.199	Fx=	9847.6		
0.193	Emin=	16		
			T=	3008.0

Shear wall  $\rho=1.3$ ,  $Ax=1.25$   
Torsion,  $Ax \rho=1.0$ ,  $Ax=1.0$   
Flex/Drift  $\rho=1.0$ ,  $Ax=1.25$   
Redundancy  $\rho=1.0$ ,  $Ax=1.0$

Use this load combination for defining Nominal Stiffness values, Keff. Then use those Keff values for all other analyses.

Expected Dead + Seismic **D+QE** (other terms if "expected" gravity loads as per AS  $\rho=1.0$ ,  $Ax=1.0$ )

$$\delta_{sw AB} = \frac{8vB^3}{EAB} + \frac{vB}{1000G_a} + \frac{\Delta A_a}{b}$$

Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\Delta_a$	$F_c$	Crush.	Shrink	$\delta_B$	$\delta_S$	$\delta_{Rot}$	$\delta_{SW}$	K (k/in)
Calculate Stiffness of Walls on A & B using <b>Transverse</b> loading																
A		37	1.0	7308.0	913.5	6390.8	13770	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581	A 25.14
B		37	1.0	7308.0	913.5	6390.8	13770	0.154	556.36	0.056	0.019	0.022	0.247	0.313	0.581	B 25.14
Calculate Stiffness of Walls on 2 & 3 using <b>Longitudinal</b> loading																
2		30	1.0	7022.0	702.2	6391.1	8340.7	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484	2 43.54
3		30	1.0	7022.0	702.2	6391.1	8340.7	0.154	505.50	0.045	0.019	0.020	0.234	0.230	0.484	3 43.54
V equal to revised wall force based on HD STR (design) capacity										625 Max.	Add stud					43.54

## Diaphragm Deflection (STR)

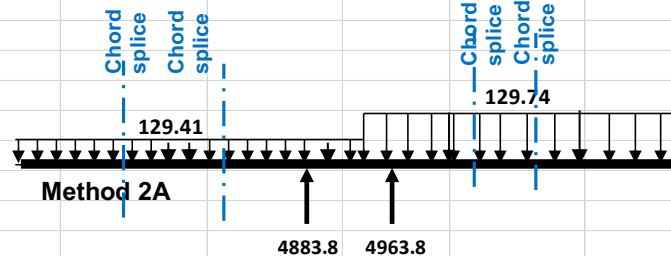
$$\delta_{Diaphragm} = \frac{3vL^3}{EAW} + \frac{0.5v_{max}L^3}{1000G_a} + \frac{\Sigma A_c X_c}{W}$$

Rt. Cantilever

Splice Forces (Lbs.)			$\Sigma \delta_{slip}$	v unif.	v conc.	Ga	L'	W'	$\delta_{Diaph}$ Unif	$\delta_{Diaph}$ conc	Total $\delta$
F15	F23	F35	In.	plf	plf	k/in.	Ft.	Ft.	In.	In.	In.
395.2	827.5	1980.6	0.041	115.55	0.00	25.0	35.00	40.00	0.135	0.00	0.135
1.75	3.66	8.76									
8	16	24									
0.008	0.009	0.014									

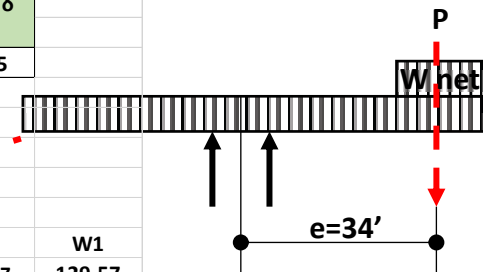
EA= 28050000, (2)2x6

lincludes effects of sw's along chord line



equivalent

W2	W1
129.57	129.57
0.17	-0.17
129.74	129.41

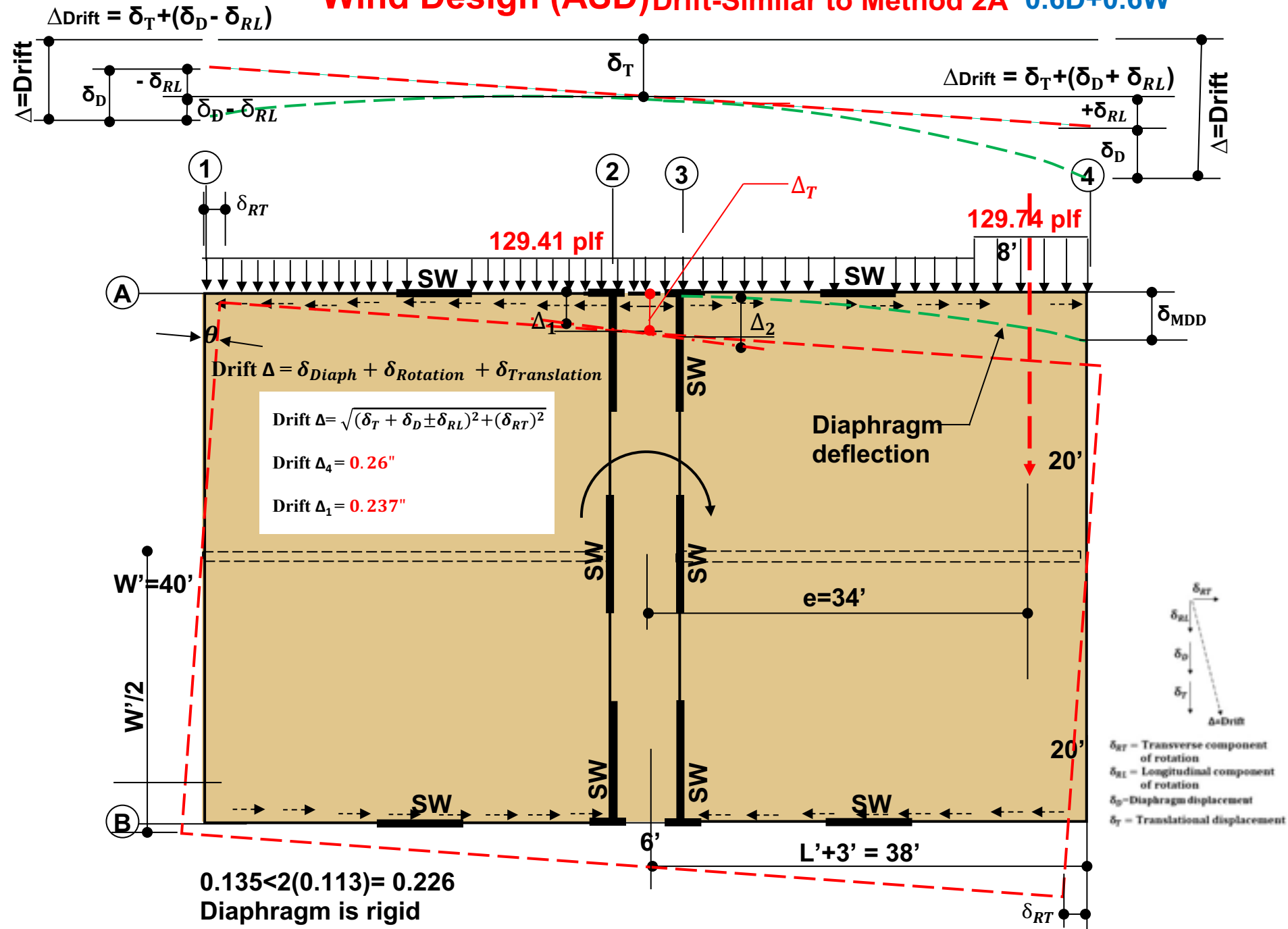


## Diaphragm Deflection (STR)

333.6	886.1	1987.6	0.041	115.26	0.00	25.0	35.00	40.00	0.135	0.00	0.135
1.48	3.92	8.79									
8	16	24									
0.007	0.009	0.014									

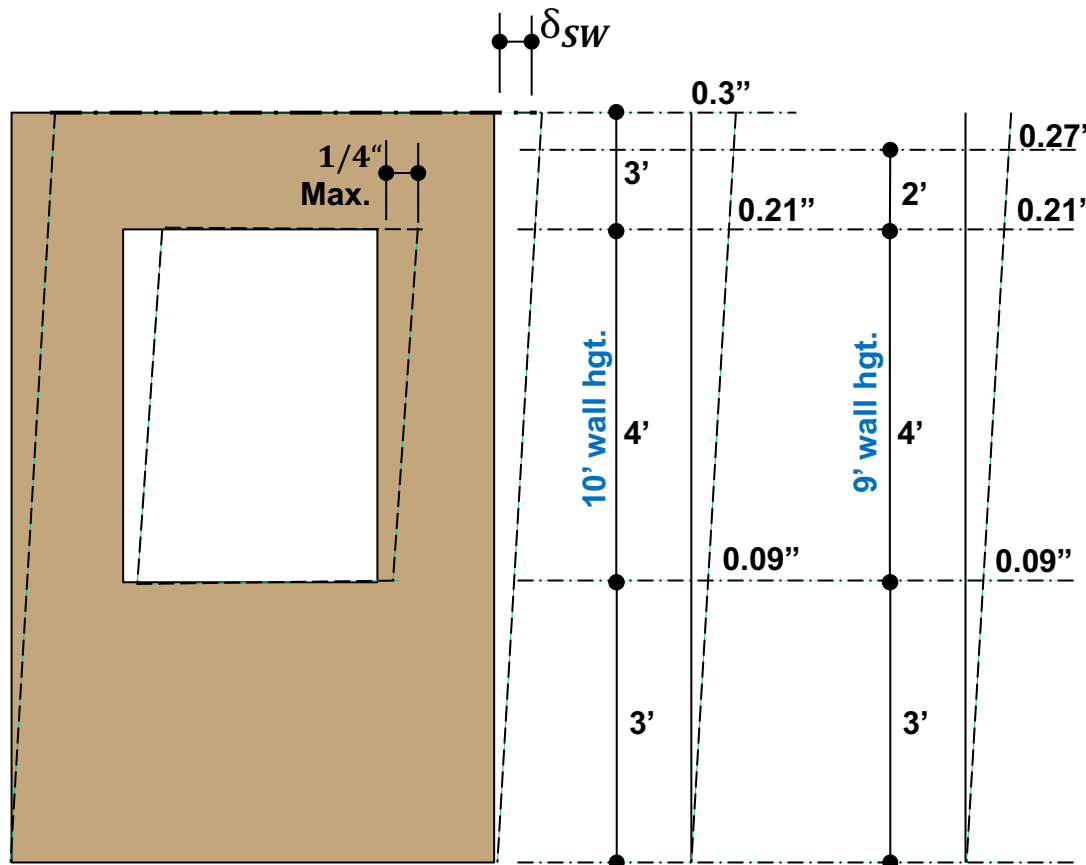
Lft. Cantilever

## Wind Design (ASD) Drift-Similar to Method 2A $0.6D+0.6W$



# Allowable Drift Wind? H/600, **H/400**, H/240, H/200 ???

(Nothing defined in code)



Assuming window manufacturers  
allowable tolerance (movement) = 0.25''  
(Check with window manufacturer)

10' wall hgt.

H/600 = 0.2'' < 0.26'' NG by inspection

**H/400** = 0.3'' at top of wall

**0.26'' < 0.3'' ∴ drift OK**

Maximum displacement at top of  
window = 0.21'' < 0.25'' ∴ OK

H/240 = 0.5'', at Top of wd. = 0.35'' > 0.25  
N.G.

9' wall hgt.

**H/400** = 0.27'' at top of wall

**0.26'' < 0.27'' ∴ drift OK**

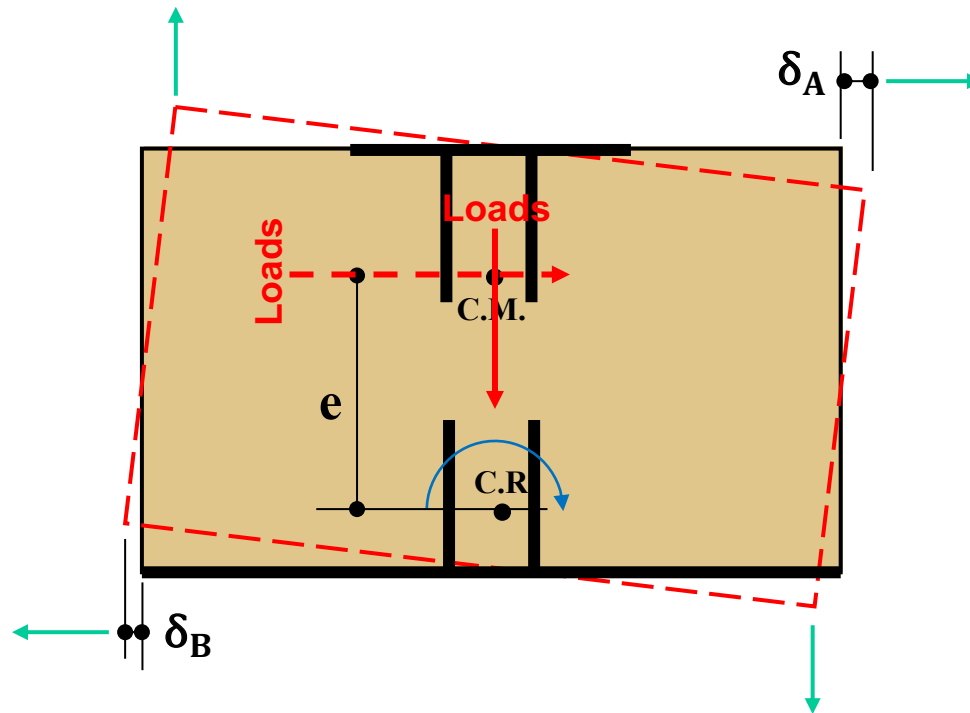
Maximum displacement at top of  
window = 0.21'' < 0.25'' ∴ OK

For resistance to **Wind** loads:

1. ASCE 7-16 Section 27.4.5-Diaphragm flexibility-The structural analysis shall consider the stiffness of diaphragms and vertical elements of the MWFRS

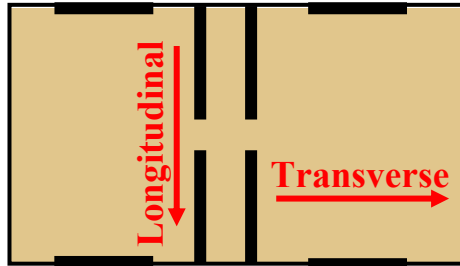
**2. Show that the resulting drift at the edges of the structure can be tolerated.**

# Torsional Irregularities



# Analysis Flow

## Longitudinal Design



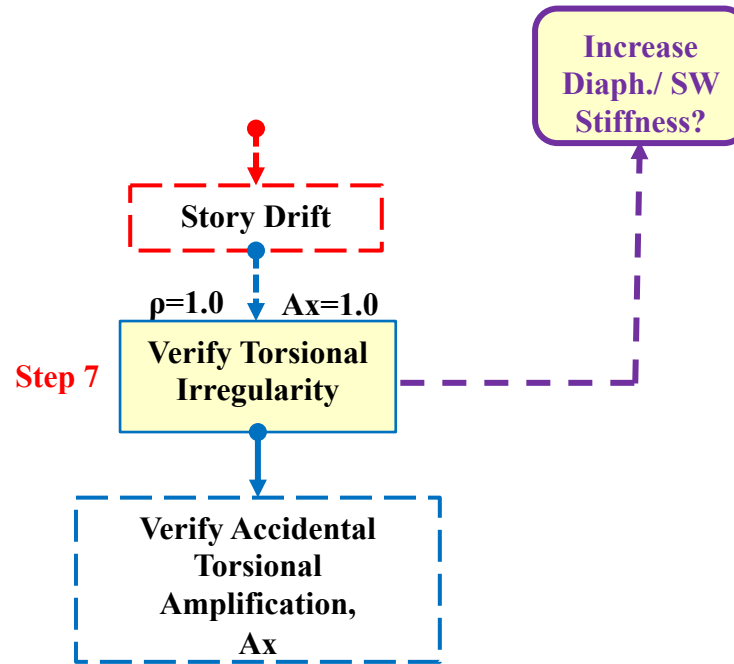
Example Plan

### Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

STR Design



## Verify Torsional Irregularity

Seismic-  $\rho=1.0$ ,  $A_x=1.0$



# Torsional Irregularities $\rho = 1.0$ and $A_x = 1.0$

ASCE 7-16 Table 12.3-1, **Type 1a and 1b** irregularities note that  $A_x=1.0$  when checking for torsional irregularities.

In many cases, open-front structures will result in torsional irregularities because of rotational effects.

SDPWS Section 4.2.5.1 addresses ASCE 7-16 torsional irregularity requirements.

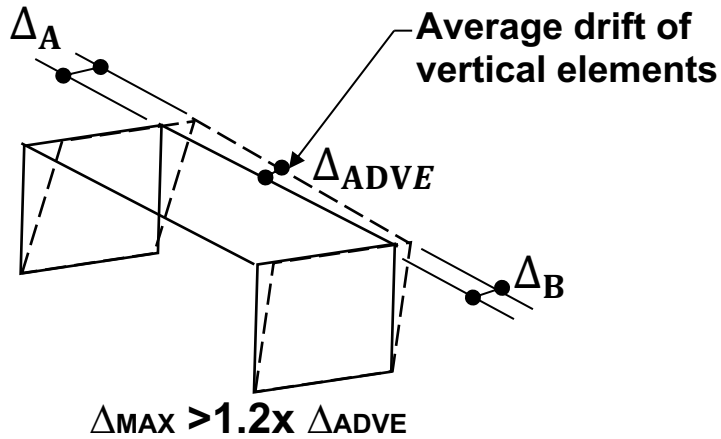
**Torsional Irregularity Type 1a** – seismic - Maximum story drift,  $\Delta_{MAX}$ , (including accidental torsion with  $A_x=1.0$ ),  $> 1.2 \times \Delta_{ADVE}$

- Model as semi-rigid or idealized as rigid
- Torsional irregularity, **Type 1a**, is allowed in structures assigned to SDC B, C, D, E, or F.

**Torsional Irregularity Type 1b** - seismic: Extreme torsionally irregular, Maximum story drift,  $\Delta_{MAX} > 1.4 \times \Delta_{ADVE}$

- An extreme torsional irregularity **Type 1b** is allowed in structures assigned to Seismic Design Categories B, C, and D, but not in SDC E, or F.

# ASCE 7 Triggers



## ASCE 7-16 Requirements Type 1a Horizontal Irregularity

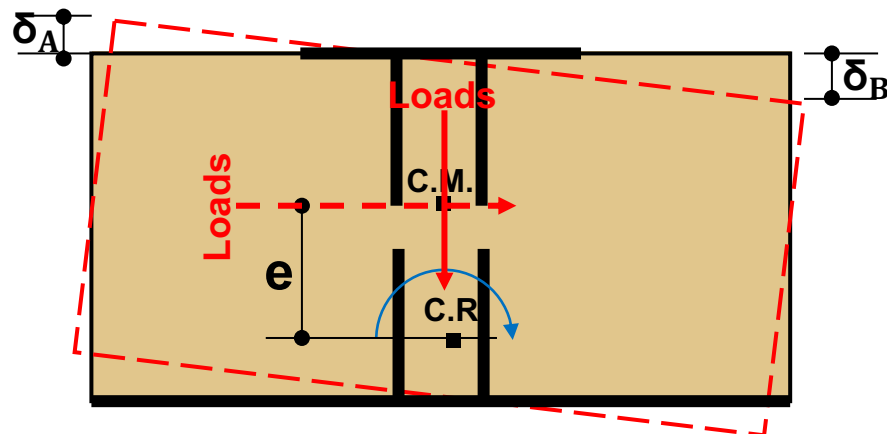
### ASCE 7-16: Table 12.3-1 Horizontal Structural Irregularity Requirement References

#### 1a. Torsional Irregularity $\Delta_{MAX} > 1.2x \Delta_{ADVE}$

- 12.3.3.4: 25% increase in forces - D, E, and F
- 12.7.3: Structural modeling - B, C, D, E, and F
- 12.8.4.3: Amplification of accidental torsion - C, D, E, and F
- 12.12.1: Drift - C, D, E, and F

#### 1b. Extreme Torsional Irregularity $\Delta_{MAX} > 1.4x \Delta_{ADVE}$

- 12.3.3.1 Type 1b is not permitted in E and F
- 12.3.3.4: 25% increase in forces – D
- 12.3.4.2: Redundancy factor – D
- 12.7.3: Structural modeling - B, C, and D
- 12.8.4.3: Amplification of accidental torsion - C and D
- 12.12.1: Drift - C and D



Longitudinal Loading $e=3.8'$ , $T = 67522.2$ ft. lbs. $\rho=1.0$ , $A_x=1.0$										
Grid Line	kx	Ky	dx	dy	kd	kd <sup>2</sup>	Fv		FT	Fv+FT
2	43.54		3		130.63	391.89	8884.5		-422.2	8462.3
3	43.54		3		130.63	391.89	8884.5		422.2	9306.7
A		25.14		20	502.74	10054.73			1624.7	1624.7
B		25.14		20	502.74	10054.73			-1624.7	-1624.7
$\Sigma$	87.09	50.27			J=	20893.23	17769			

Walls at Grid Corridor  
lines A & B Walls

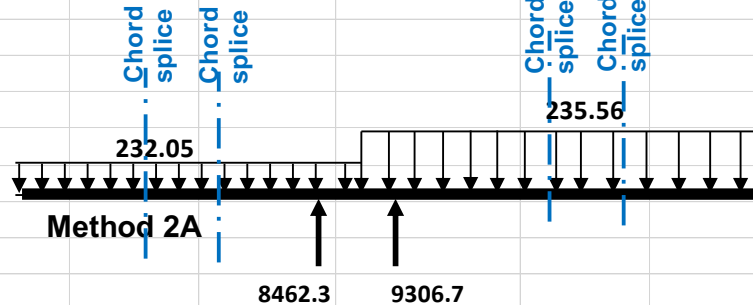
### Diaphragm Deflection (STR) $\rho=1.0$ , $A_x=1.0$

Rt. Cantilever

Splice Forces (Lbs.)			$\Sigma \delta_{\text{slip}}$ In.	v unif. plf	v conc. plf	Ga k/in.	L' Ft.	W' Ft.	$\delta_{\text{Diaph Unif}}$ In.	$\delta_{\text{Diaph conc}}$ In.	Total $\delta$ In.
F 15	F23	F35									
983.2	1236.9	3542.8	0.075	227.49	0.00	25.0	35.00	40.00	0.260	0.00	0.260
ails Req'd=	4.35	5.47	15.68								
Use Nails =	8	16	24								
Slip=	0.021	0.013	0.025								

EA= 28050000, (2)2x6

Includes effects of sw's along chord line



### Diaphragm Deflection (STR)

Lft. Cantilever

332.0	1855.1	3617.4	0.073	224.42	0.00	25.0	35.00	40.00	0.256	0.00	0.256
1.47	8.21	16.01									
8	16	24									
0.007	0.020	0.026									

[illegible]

## Torsion (Question 7):

# Check for Torsional Irregularity Type 1a - $\rho=1.0$ , $A_x=1.0$

SDPWS 4.2.5.2 (2):

A.R.  $\leq 1:1$  if torsional irregularity - one-story structure

A.R. = 0.67:1 - multi-story structure

A.R. = 0.875 < 1,  $\therefore$  O.K. Had this been a multi-story structure, the A.R. would have been exceeded and adjustments made accordingly.

$$\Delta_2 = 0.194", \quad \Delta_3 = 0.214"$$

$$\Delta_{Aver} = \frac{0.194 + 0.214}{2} = 0.204"$$

$\delta_{SWA,B} = 0.065'' = \delta_{RT}$  Transverse displacement at Lines A and B from rigid diaphragm rotation

$$\delta_{RL} = \frac{2\delta_{SWA,B}(L' + 3')}{W_l} = 0.124'' \quad \text{Vertical component of rotation}$$

Diaphragm deflections:

$$\delta_{D,1} = 0.256''$$

$$\delta_{D,4} = 0.260''$$

$$\text{Drift } \Delta = \sqrt{(\delta_T + \delta_D \pm \delta_{RL})^2 + (\delta_{RT})^2}$$

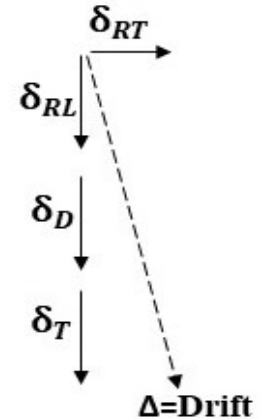
$$\text{Drift } \Delta_4 = \sqrt{(0.204 + 0.260 + 0.124)^2 + (0.065)^2} = 0.592"$$

$$\text{Drift } \Delta_1 = \sqrt{(0.204 + 0.256 - 0.124)^2 + (0.065)^2} = 0.342"$$

$$\Delta_{Aver} = \frac{0.592 + 0.342}{2} = 0.467"$$

**0.592 > 1.2(0.467) = 0.56", ∴ Horizontal torsional irregularity Type 1a does exist in this direction.**

**0.592 < 1.4(0.467) = 0.654", ∴ Horizontal torsional irregularity Type 1b does not exist in this direction.**



$\delta_{RT}$  = Transverse component of rotation

$\delta_{RL}$  = Longitudinal component of rotation

$\delta_D$  = Diaphragm displacement

$\delta_T$  = Translational displacement

# Lunch



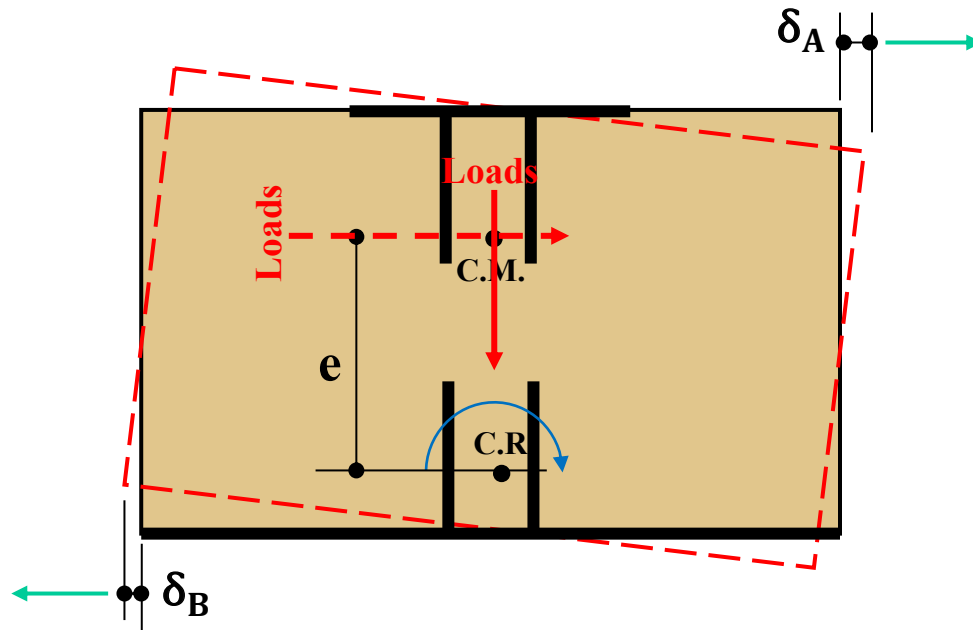
### **Part 4-Design Example (cont.):**

- **Amplification of accidental torsion**
- **Redundancy**
- **Transverse direction design**
- **Miscellaneous plan layouts and multi-story effects**



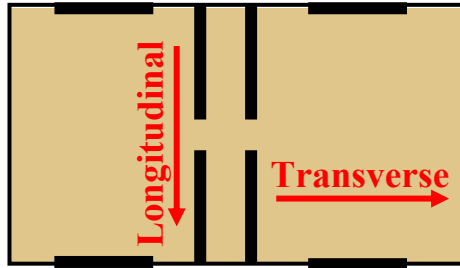
# Amplification of Accidental Torsion

Seismic-  $\rho=1.0$ ,  $A_x=1.0$

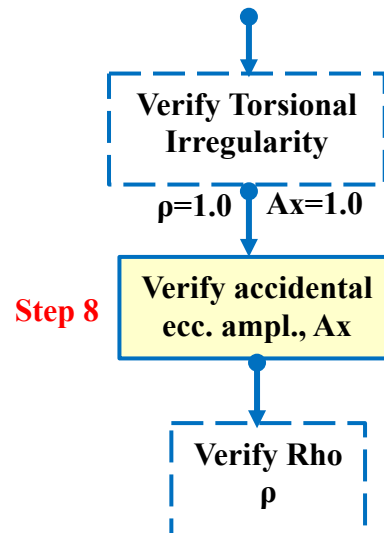
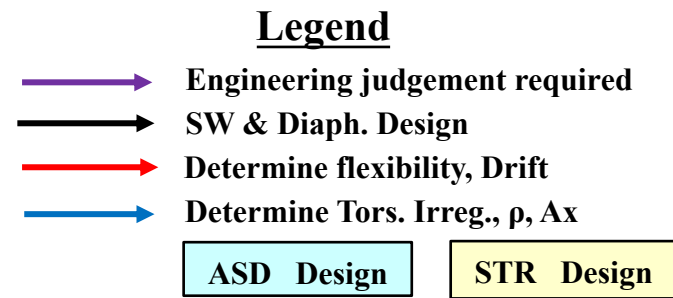


# Analysis Flow

## Longitudinal Design



Example Plan



## Verify Amplification of Accidental Torsion, $A_x$

Seismic-  $\rho=1.0$ ,  $A_x=1.0$

### ASCE 7-16 12.8.4.3 Amplification of Accidental Torsional Moment.

Structures assigned to Seismic Design Category C, D, E, or F, where **Type 1a or 1b** torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying  $M_{ta}$  at each level by a torsional amplification factor ( $A_x$ ) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left( \frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 \quad 12.8-14$$

Where

$\delta_{max}$  = maximum displacement at level  $x$  computed assuming  $A_x = 1$

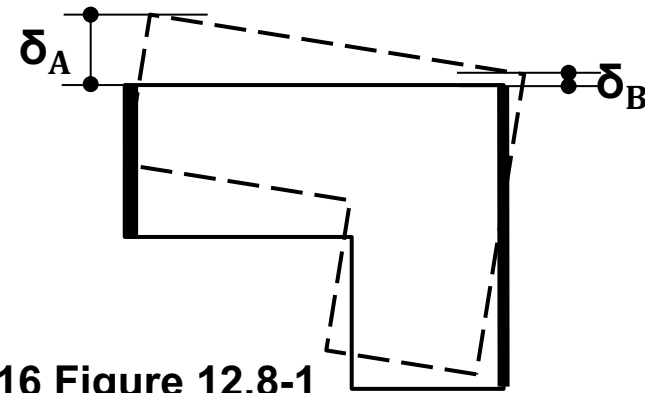
$\delta_{avg}$  = average of the displacements at the extreme points of the structure at level  $x$  computed assuming  $A_x = 1$ .

$M_{ta}$  = accidental torsional moment

From torsion section:

$$A_x = \left( \frac{\delta_{max}}{1.2\delta_{avg}} \right)^2 = \left( \frac{0.592}{1.2(.467)} \right)^2 = 1.116 < 1.25 \text{ assumed.}$$

∴ Can recalculate if desired.

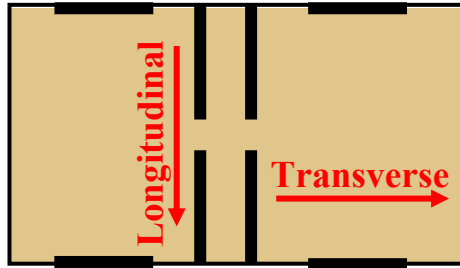


ASCE 7-16 Figure 12.8-1  
Amplification of accidental torsion

**ASCE 7-10 (1<sup>st</sup> printing) 12.8.4.1 Inherent Torsion Exception below is not in 3<sup>rd</sup> printing of ASCE 7-10 or ASCE 7-16**  
Most diaphragms of light-framed construction are somewhere between rigid and flexible for analysis purposes, that is, semi-rigid. Such diaphragm behavior is difficult to analyze when considering torsion of the structure. As a result, it is believed that consideration of the **amplification of the torsional moment is a refinement that is not warranted for light-framed construction.**

# Analysis Flow

## Longitudinal Design



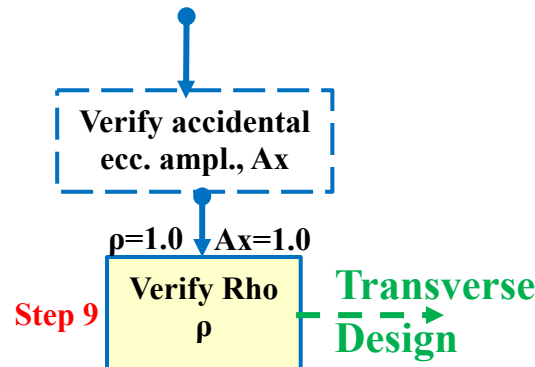
Example Plan

## Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

ASD Design

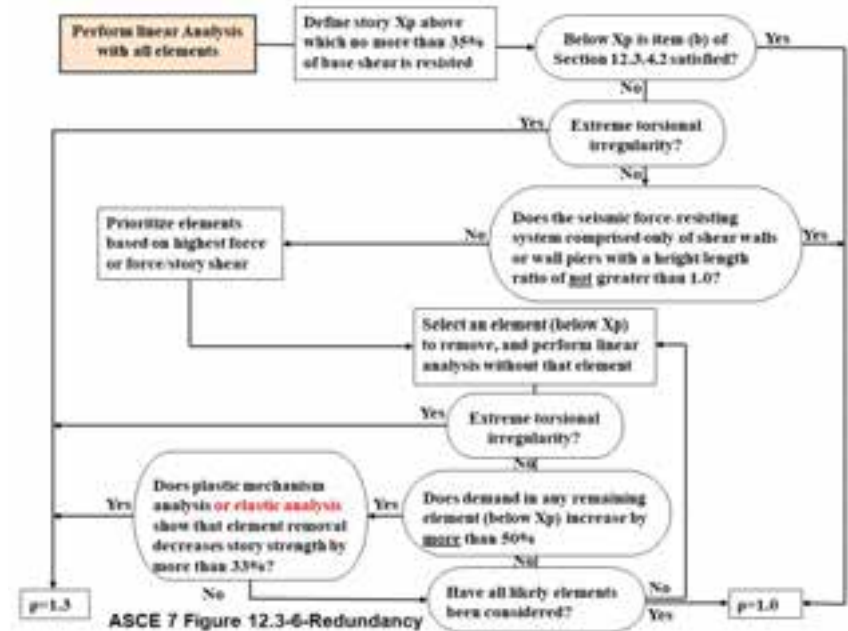
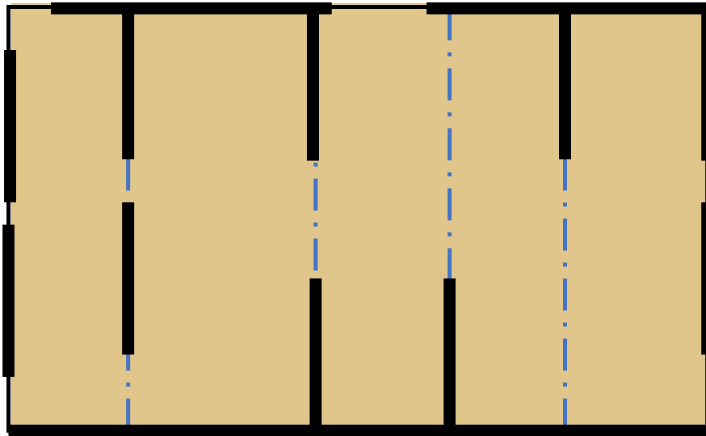
STR Design



**Verify Redundancy,  $\rho$**   
**Seismic-  $\rho=1.0$ ,  $A_x=1.0$**

# Redundancy

Seismic-  $\rho=1.0$ ,  $A_x=1.0$



ASCE 7-16 Redundancy Flow Chart  
Figure C12.3-6

- The application of rho relates directly to increasing the capacity of the walls only, or adding more walls.
- The rho factor has an effect of reducing R, for less redundant structures which increases the seismic demand
- Shear wall systems have been included in Table 12.3-3 so that either an adequate number of walls are included, or a proper redundancy factor has been applied.

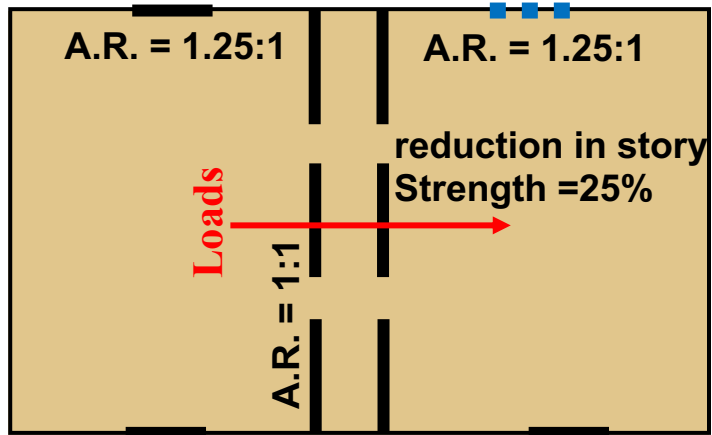
**12.3.4.1 Conditions Where Value of  $\rho$  is 1.0. The value of  $\rho$  is permitted to equal 1.0 for the following:**

- 2. Drift calculation and P-delta effects.**
- 5. Design of collector elements, splices, and their connections for which the seismic load effects including over-strength factor of section 12.4.3 are used.**
- 6. Design of members or connections where seismic load effects including over-strength factor of section 12.4.3 are required for design.**
- 7. Diaphragm loads,  $F_{px}$ , determined using Eq. 12.10-1, including min. & max. values.**

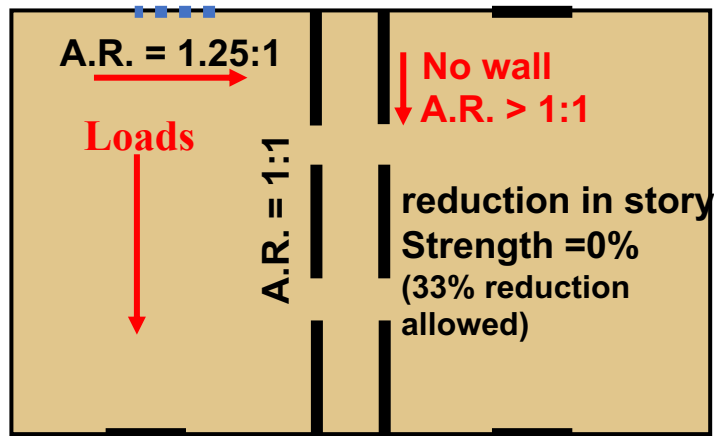
**12.3.4.2 Redundancy Factor,  $\rho$ , for Seismic Design Categories D through F.**

- For structures assigned to Seismic Design Category D and having extreme torsional irregularity as defined in Table 12.3-1, Type 1b,  $\rho$  shall equal 1.3.**
- For other structures assigned to Seismic Design Category D and for structures assigned to Seismic Design Categories E or F,  $\rho$  shall equal 1.3 unless one of the following two conditions (a. or b.) is met, whereby  $\rho$  is permitted to be taken as 1.0.**
  - a. Each story resisting more than 35% of the base shear in the direction of interest shall comply with Table 12.3-3.**

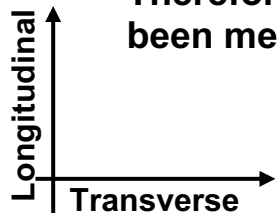
**Let's check condition b. first**



No. bays =  $2(8)(2)/10 = 3.2$  bays  
(But not all 4 sides)



Therefore condition “a” has been met and  $p=1.0$ .



**b.** Structures that are regular in plan at all levels  $p=1.0$  provided:

- SFRS consist of at least two bays of **perimeter SFRS framing** on **each side of the structure in each orthogonal direction** at each story resisting more than 35% of the base shear.
- The number of bays for a shear wall =  $L_{sw} / h_{sx}$ , or  $2L_{sw} / h_{sx}$ , for light-frame construction.

Although the plan is regular, in the longitudinal direction, there are no SFRS walls at all exterior wall lines.

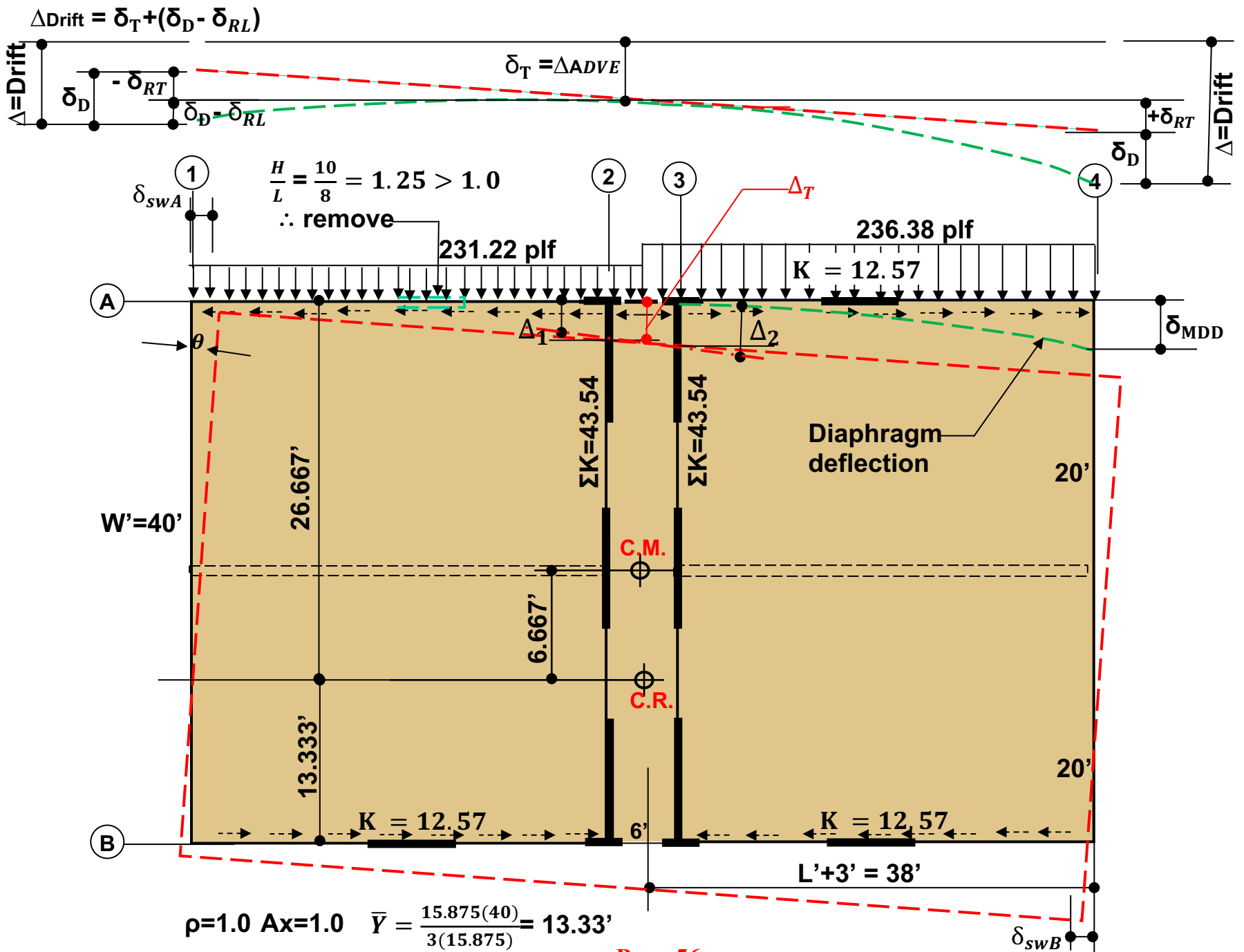
For this example, it is apparent that in the longitudinal direction the structure does not comply with condition “b”.

Therefore condition “a” must be met.

**Condition a.** Table 12.3-3.

Removing one wall segment with A.R. > 1:1

- No wall with A.R. > 1:1
- No reduction in story strength > **33% limit.**
- Removing 1 wall at line A will not result in extreme torsional irregularity, Type 1b.





# Redundancy Study

## Spreadsheet results

- $\delta_A = 0.127''$
- $\delta_B = 0.063''$
- $\delta_2 = 0.190''$
- $\delta_3 = 0.218''$
- $\Delta_{Diaph} L = 0.256''$
- $\Delta_{Diaph} R = 0.260''$

	Total
F <sub>A</sub>	1595
F <sub>B</sub>	1595
F <sub>2</sub>	8263
F <sub>3</sub>	9506

## Check

$$\Delta_{Rot} = \frac{0.127(38)}{26.667} = 0.181''$$

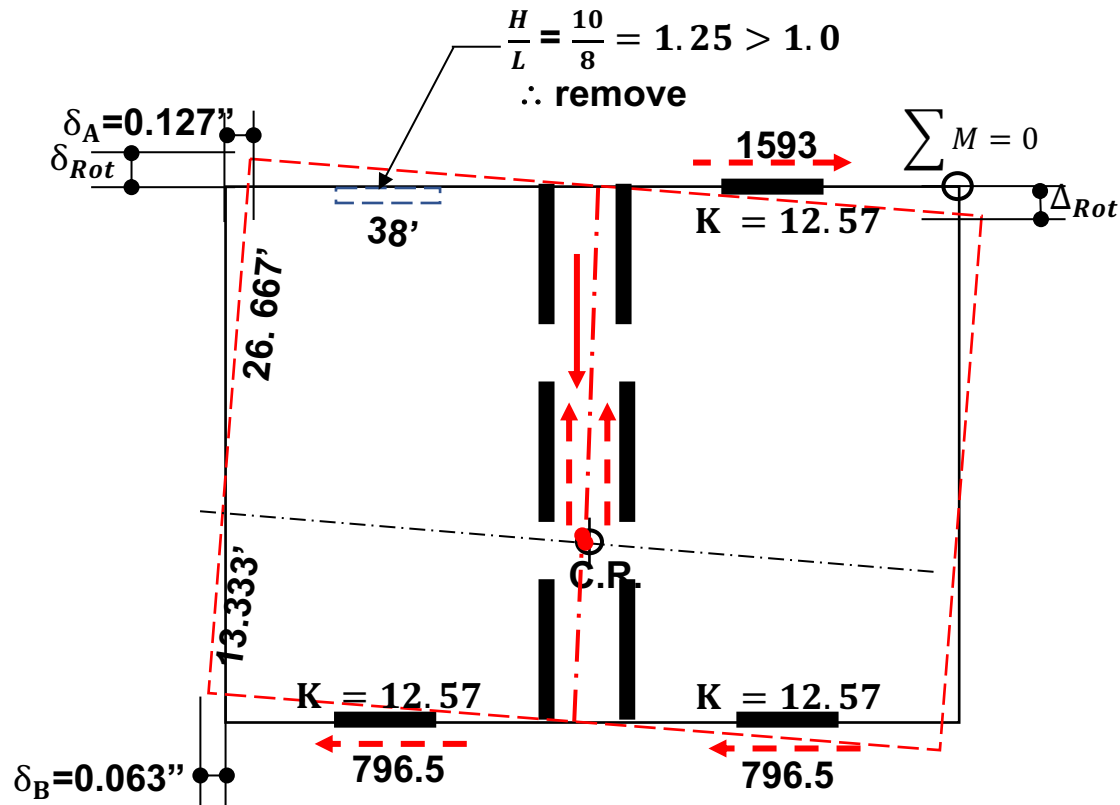
$$\Delta_T = \frac{0.190 + 0.218}{2} = 0.204''$$

$$Drift_{\Delta_4} = \sqrt{(0.204 + 0.260 + 0.181)^2 + (0.127)^2} = 0.657''$$

$$Drift_1 = \sqrt{(0.204 + 0.256 - 0.181)^2 + (0.127)^2} = 0.307''$$

$$\Delta_{Aver} = \frac{0.657 + 0.307}{2} = 0.482''$$

$0.657 < 1.4(0.482) = 0.674''$ ,  $\therefore$  Horizontal torsional irregularity Type 1b **does not** exist in this direction and  $\rho = 1.0$



# 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 w/o over-strength but not if they support discontinuous walls or frames.

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

Shall be the maximum of:

**Same** {  $\Omega_o F_x$  - 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

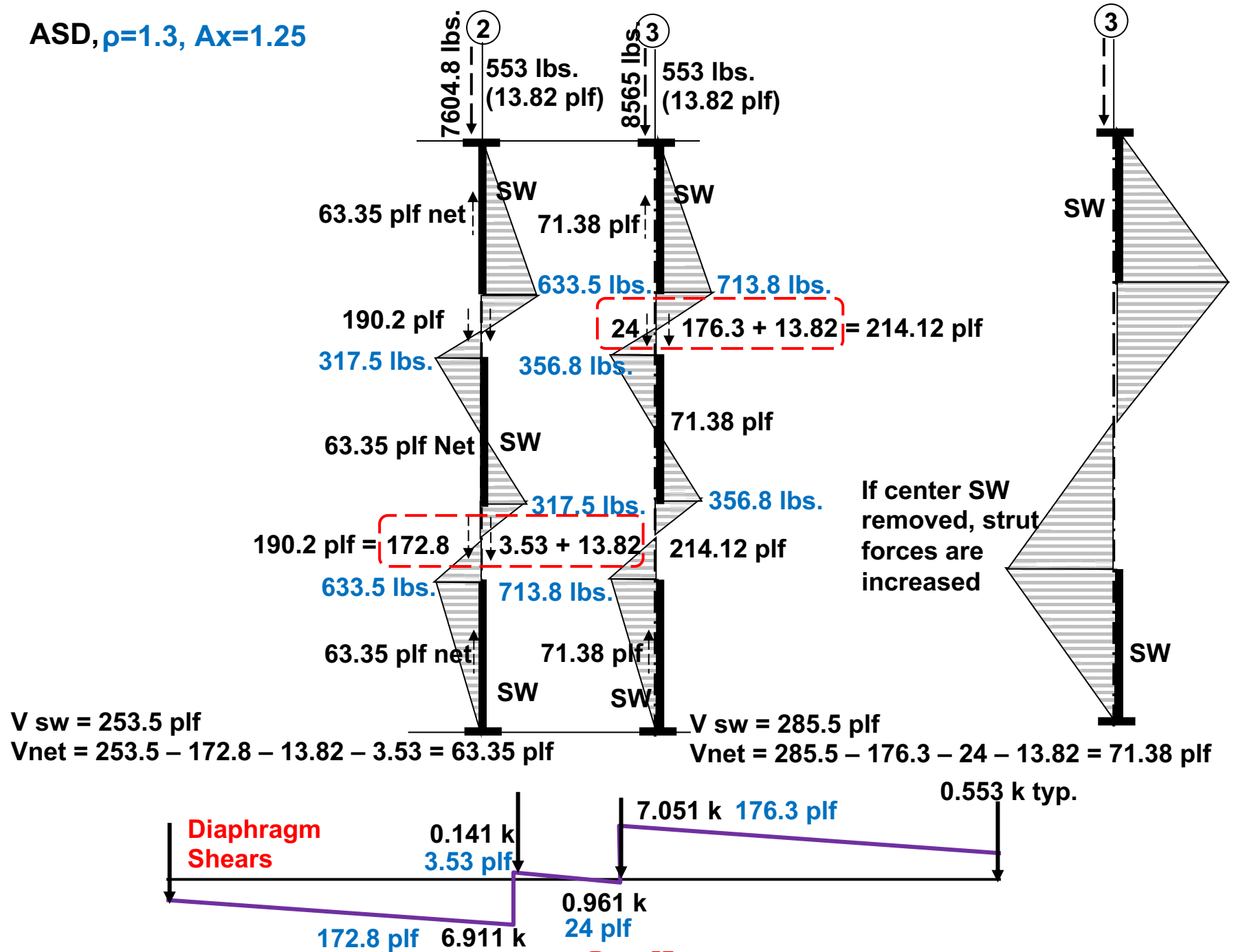
**and**  $F_{px\ min} = 0.2S_{DS}I_e w_{px}$  - Lower bound seismic diaphragm design forces determined by Eq. 12.10-2 (**Fpx<sub>min</sub>**) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength)-**do not require the over-strength factor.**

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

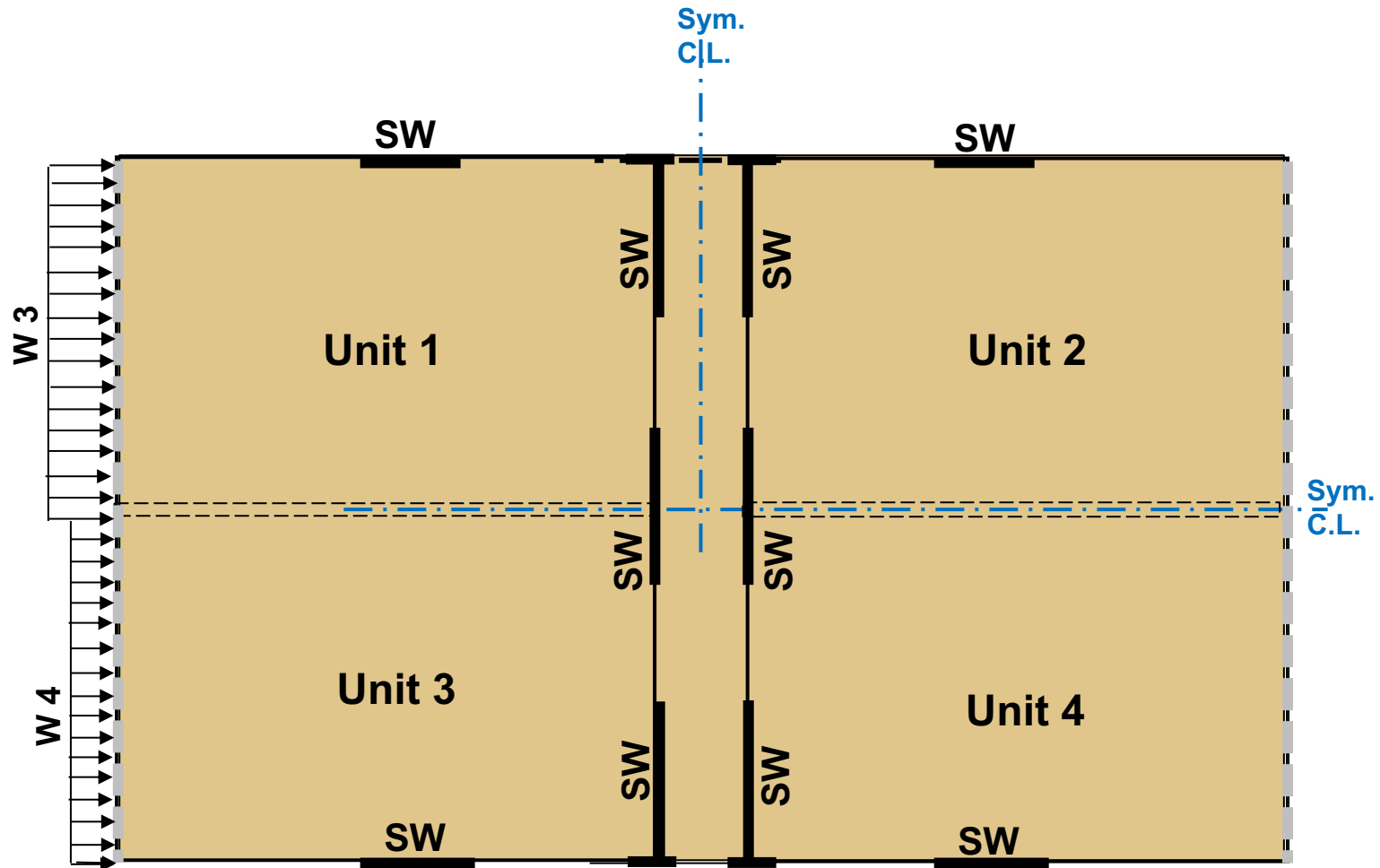
Exception:

1. In structures (or portions of structures) braced entirely by light framed shear walls, 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,  $F_{px}$ ).

**ASD,  $\rho=1.3$ ,  $A_x=1.25$**

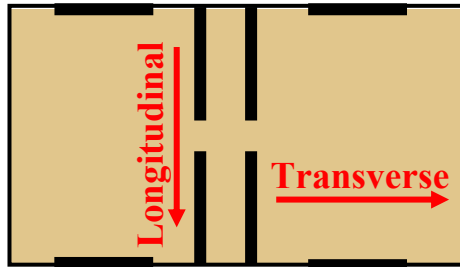


# Design Example- Transverse Direction



# Analysis Flow

## Longitudinal Design



Example Plan

## Transverse Design

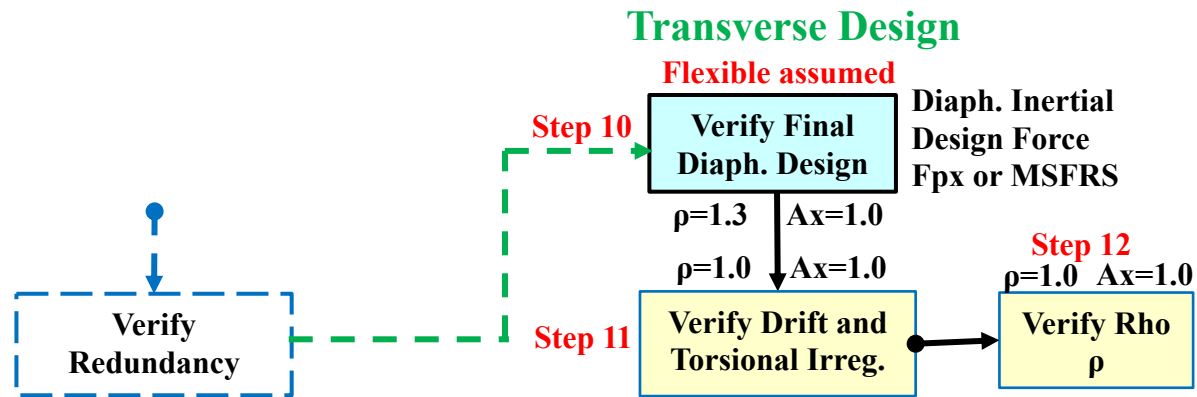
Seismic-  $\rho=1.3$ ,  $A_x=1.0$

### Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg.,  $\rho$ ,  $A_x$

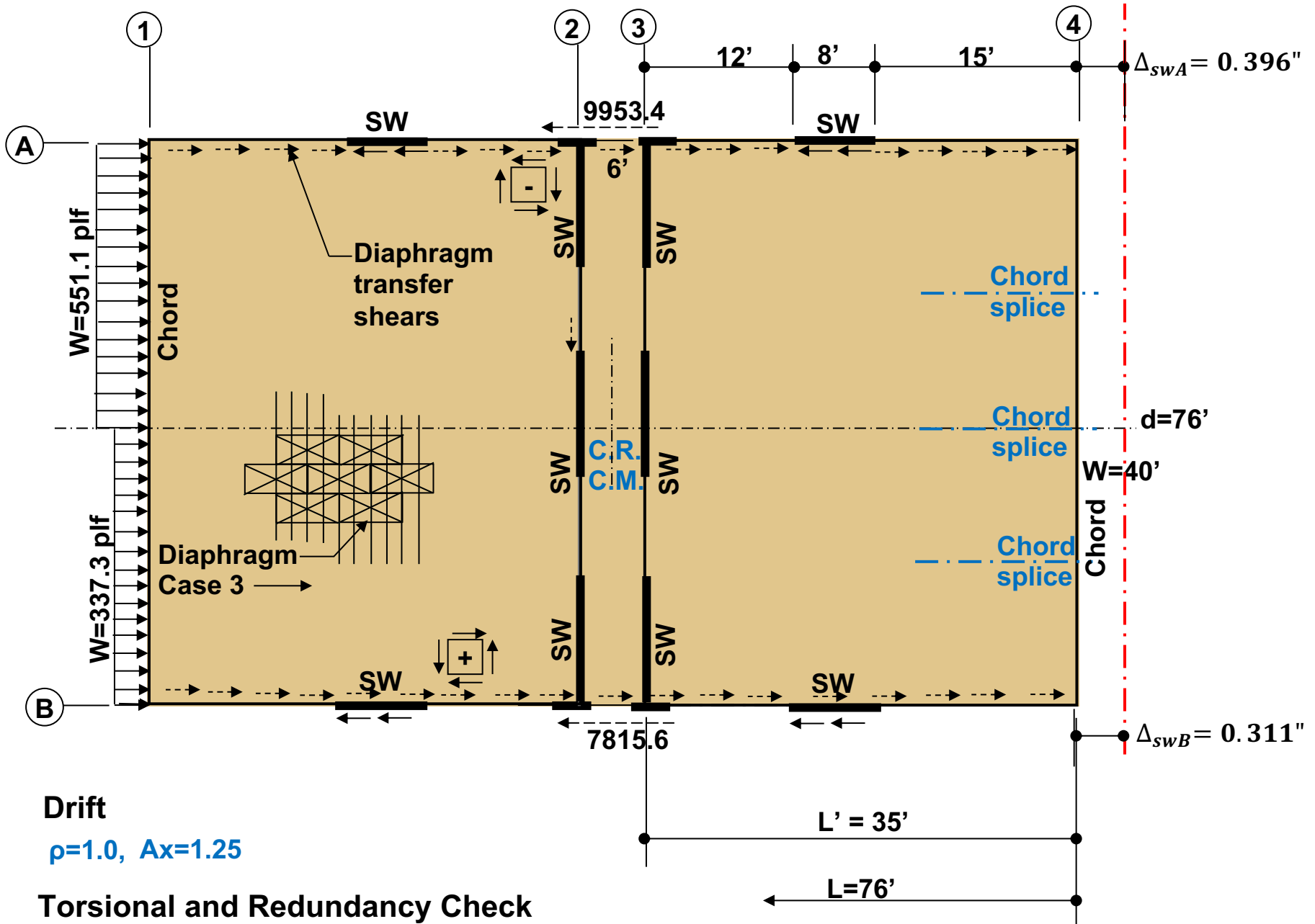
ASD Design

STR Design



12.3.1.1- (c), **Light framed construction**, diaphragms meeting all the following conditions are allowed to be idealized as flexible:

1. All Light framed construction
2. Non-structural concrete topping  $\leq 1 \frac{1}{2}$ " over wood structural panels (WSP).
3. Each elements of the seismic line of vertical force-resisting system complies with the allowable story drift of Table 12.12-1



## Diaphragm Flexibility, Resulting numbers: $\rho=1.0$ , $A_x=1.25$

$$W = 17769/76 = 444.1 \text{ plf (ASD)}$$

$$V_A = 9057.6 \text{ lbs.}$$

$$V_{\max \text{ Diaph}} = \frac{9057.6}{76} = 119.2 \text{ plf} < 464 \text{ plf} \therefore \text{O.K}$$

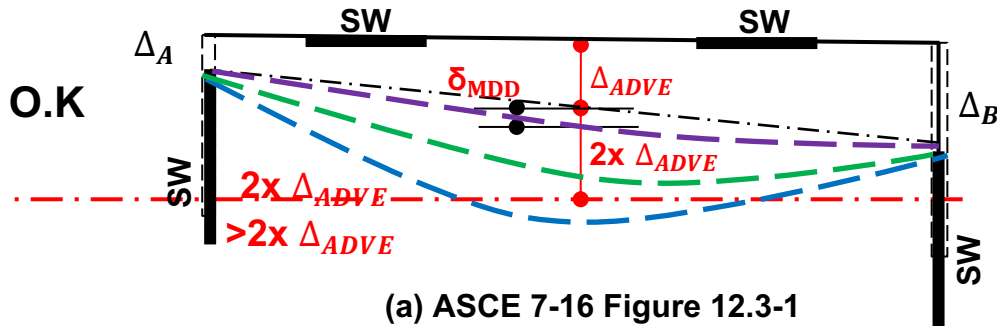
From spreadsheet (STR)

$$\delta_{\text{Diaph}} = 0.066''$$

$$\Delta_{SWA} = 0.396'', \Delta_{SWB} = 0.311'', 2x\Delta_{\text{Average}} = 0.707''$$

$0.066'' < 0.707'' \therefore$  **Rigid diaphragm**, as initially assumed.

--- Rigid  
 --- Semi-rigid  
 --- Flexible



## Check Story Drift

$$\rho = 1.0 \text{ and } A_x = 1.25$$

$$C_d = 4, I_e = 1$$

$$\delta_{SWA} = 0.396 \text{ in from spreadsheet}$$

$$\delta_M = \frac{C_d \delta_{\max}}{I_e} = \frac{4(0.396)}{1} = 1.58 \text{ in}$$

$$0.20 h_{sx} = 0.020(10)(12) = 2.4 \text{ in} > 1.58 \text{ in}, \therefore \text{Drift OK}$$

## Check for Torsional Irregularity $\rho=1.0$ , $A_x=1.0$

Rigid diaphragm,  $\rho = 1.0$  and  $A_x = 1.0$  as required by ASCE 7 Table 12.3-1

From spreadsheet

$$\delta_{SWA} = 0.387''$$

$$\delta_{SWB} = 0.319''$$

$$\Delta_{Average} = \frac{0.387 + 0.319}{2} = 0.353'' \text{ From spreadsheet}$$

$0.387 < 1.2(0.353) = 0.424''$ ,  $\therefore$  No torsional irregularity exists in this direction, as assumed.



# Redundancy Check $\rho=1.0, A_x=1.0$

## Table 12.3-3 Requirements

- Removal of SW with  $H/L > 1.0$ 
  - Will not result in  $> 33\%$  reduction in strength
  - Will not result in extreme torsional irregularity

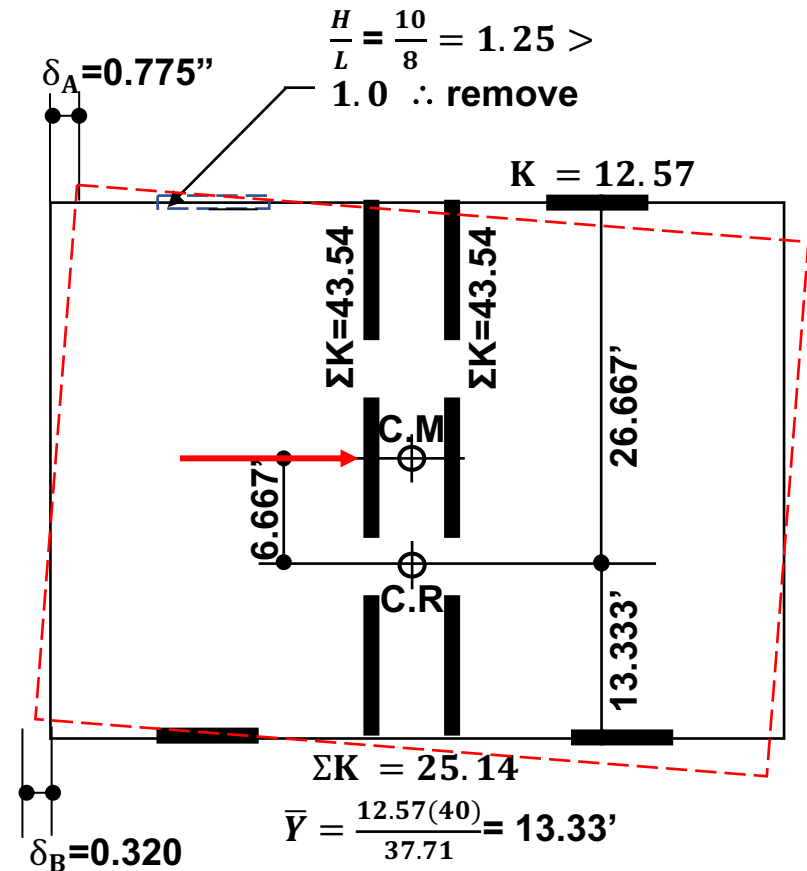
- $\delta_A = 0.775''$

- $\delta_B = 0.320''$

$$\Delta_{Aver} = \frac{0.775 + 0.320}{2} = 0.547''$$

Only 25% decrease in story strength.

$$0.775'' > 1.4(0.547) = 0.765'' \therefore \text{Type 1b} \therefore \rho=1.3$$



# Example Summary

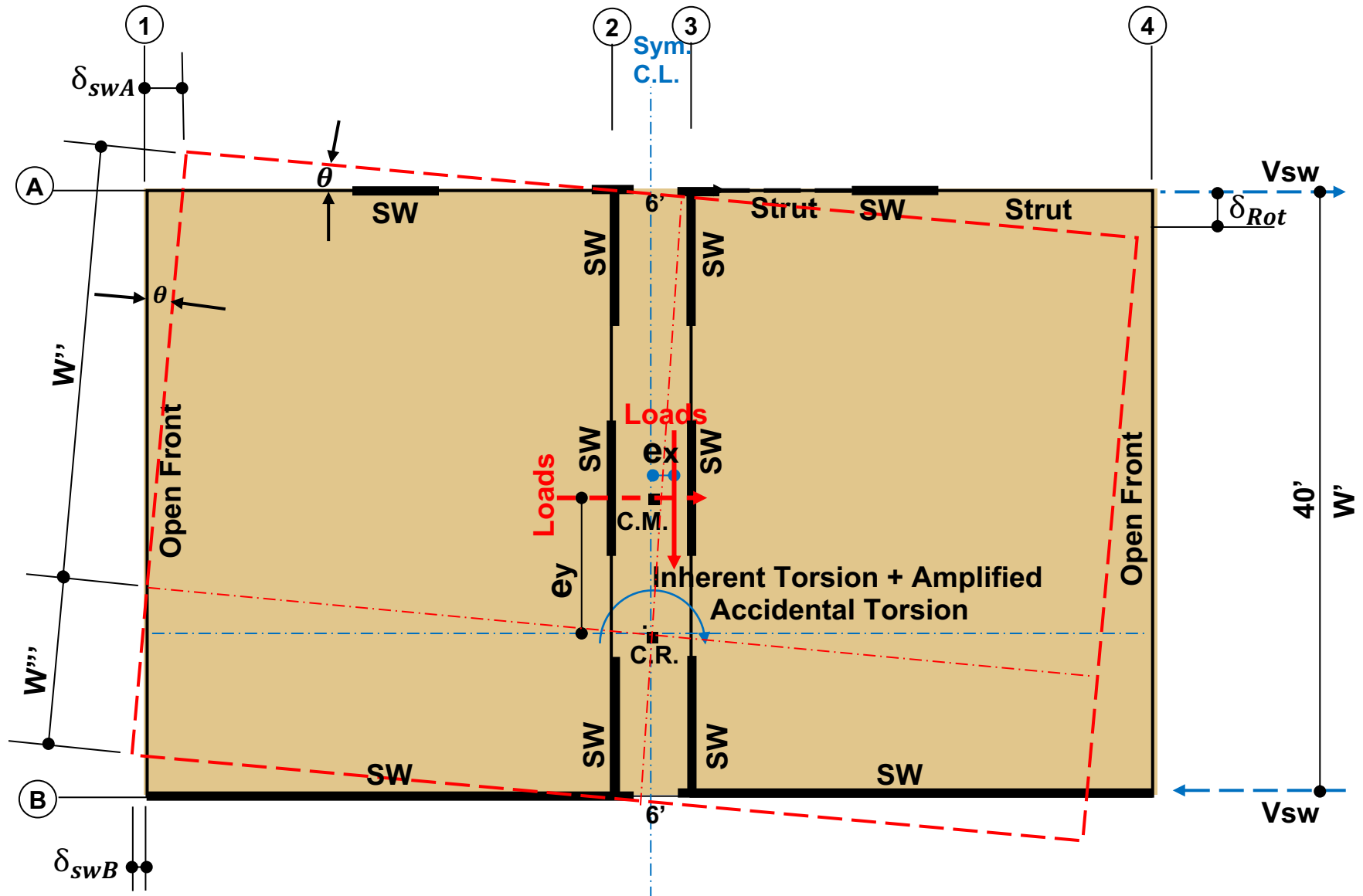
## Preliminary Assumptions Made:

- Diaphragm is rigid or semi-rigid in both directions. **Correct**
- Torsional irregularity Type 1a occurs in longitudinal direction, but not transverse, **Correct**
- $A_x=1.25$  assumed. **Incorrect,  $A_x=1.121$**
- Horizontal irregularity Type 1b does not occur in either direction. **Correct, however, when checking redundancy, it occurs in the transverse direction by the removal of 1 wall.**
- No redundancy in both directions,  $\rho=1.3$  **Incorrect:**
  - $\rho = 1.0$  Longitudinal
  - $\rho = 1.3$  Transverse

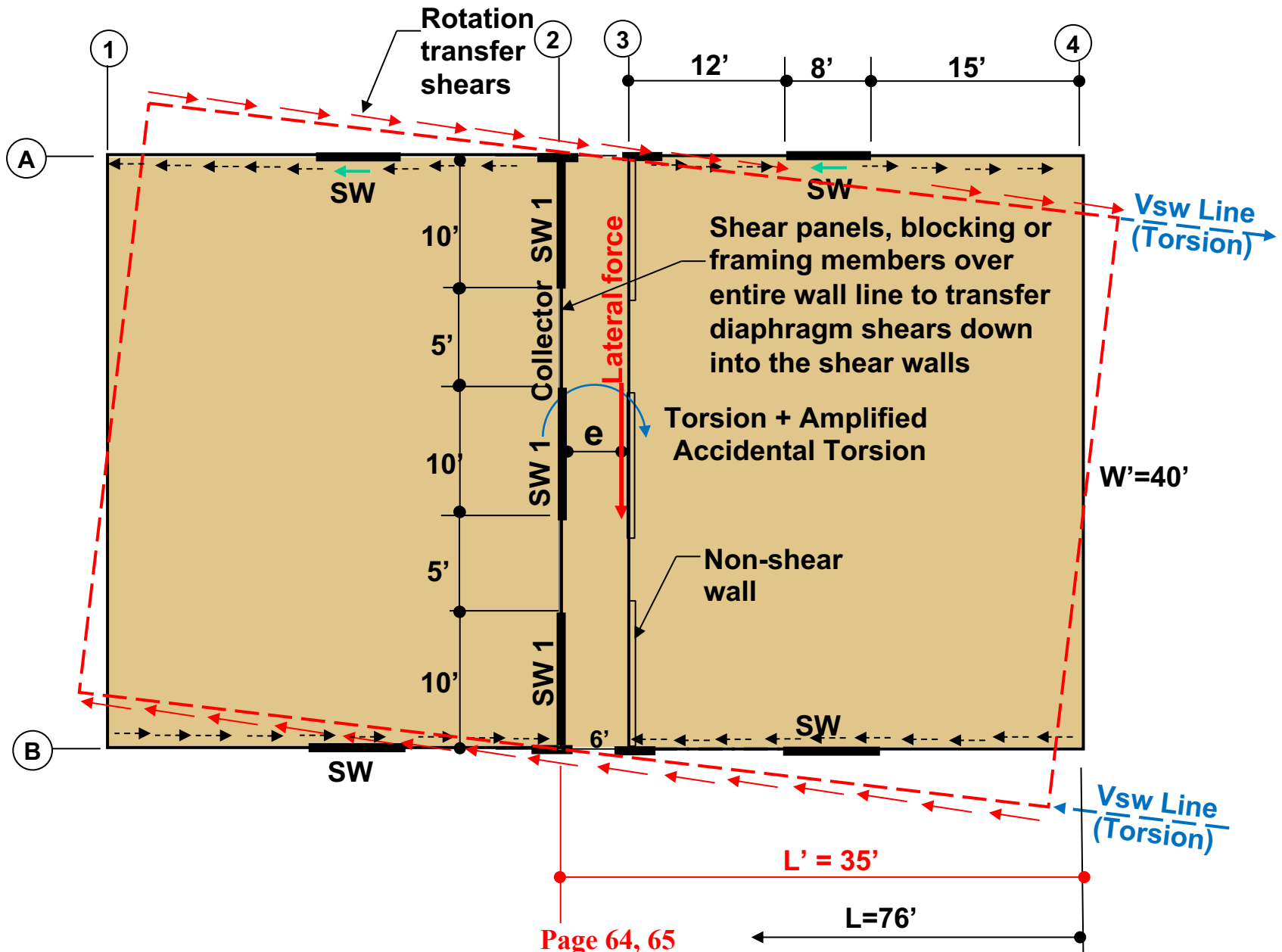
## Other Design Requirements:

- Drift **< allowable**

# Unsymmetrical Plan Layouts



# Corridor Walls One Side Only

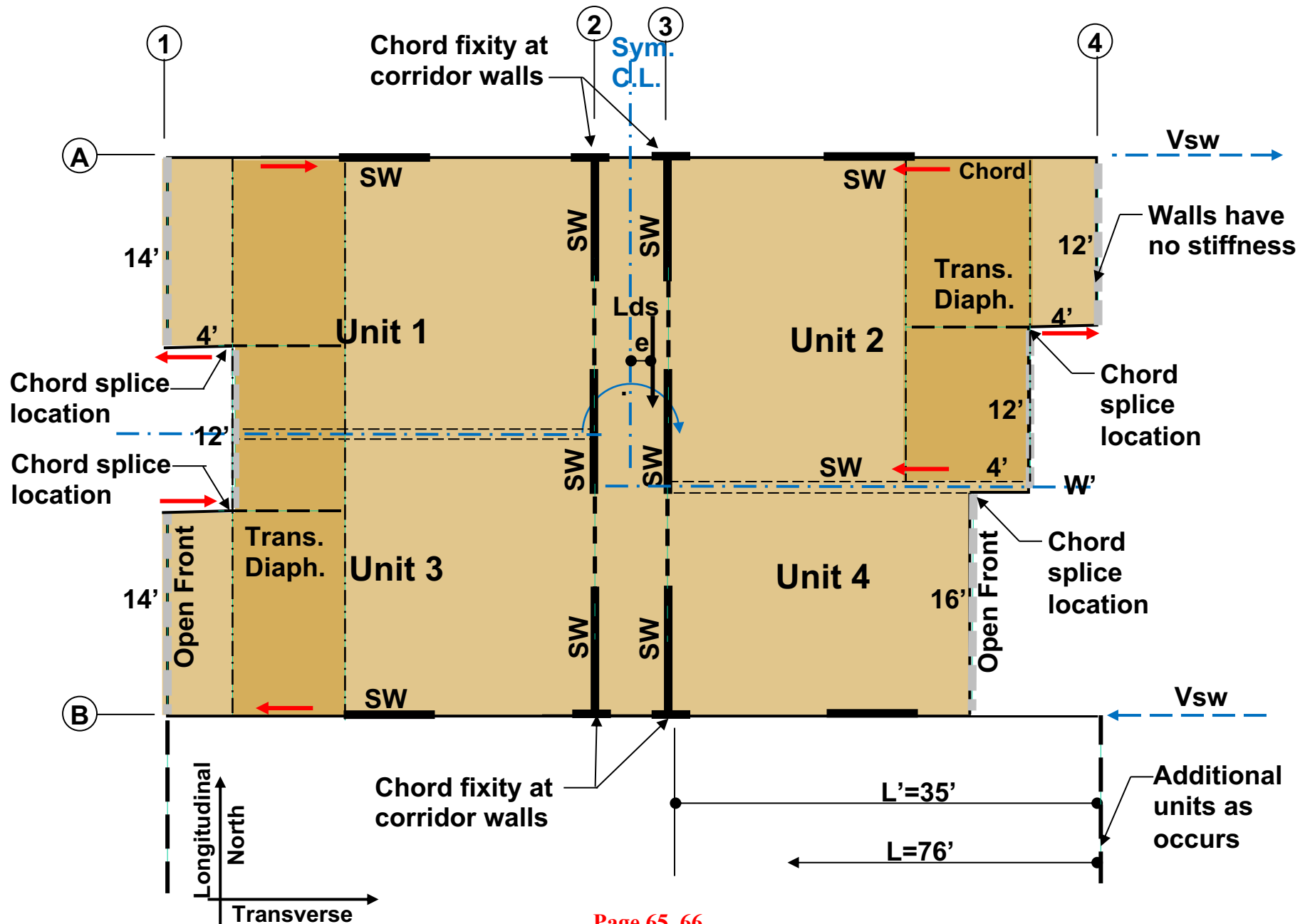


# Multi-Story, Stiffness Issues

---

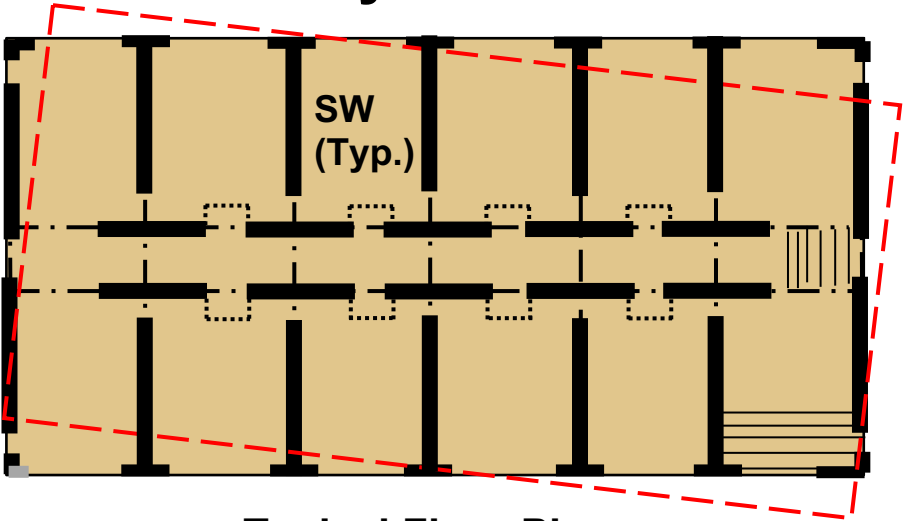


# Complex Plans



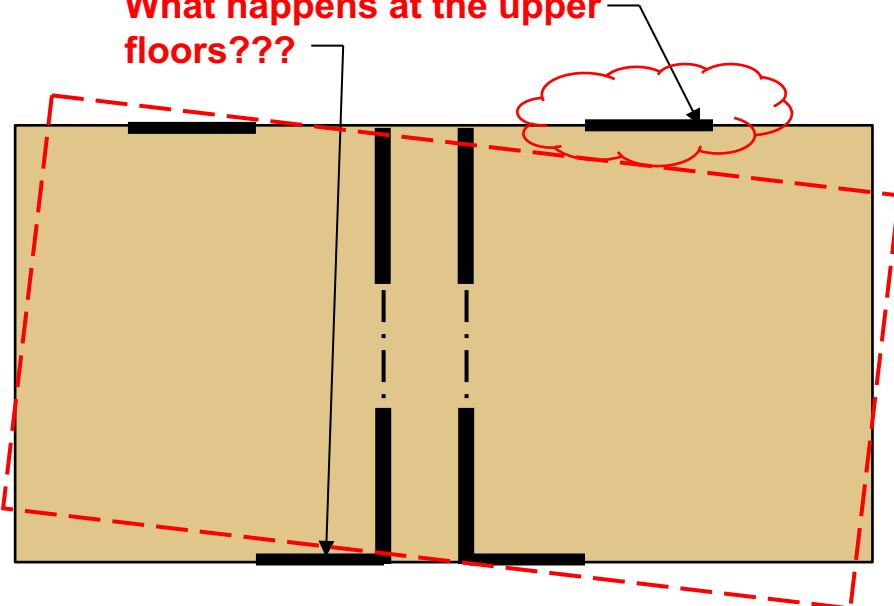
# Consideration of Shear Wall Multi-story Effects- Not in paper

Torsional Irregularity?

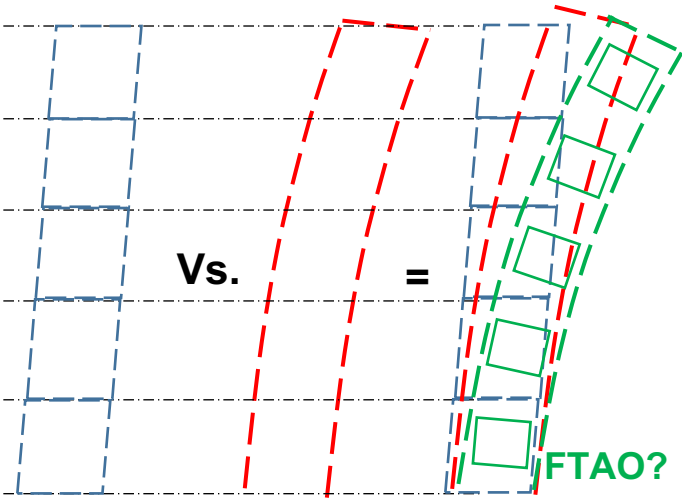


Typical Floor Plan

Multi-story SW Effects ???  
What happens at the upper floors???



Unsymmetrical Floor Plan



Traditional SW    MBA SW

Not in example

# Current Examples of Shear Wall Multi-story Effects and Mid-rise Analysis

## Current Examples of Mid-rise Analysis-Traditional Method

- **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>

- **SEAOC/IBC Structural Seismic Design Manual, Volume 2. 2015. Structural Engineers Association of California. Sacramento, CA**

## Current Examples of Mid-rise Analysis-Mechanics Based Approach **Not currently addressed or required by code**

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

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

→ • **FPInnovations-Website **NEW****

**"Seismic Analysis of Wood-Frame Buildings on Concrete Podium", Newfield**

→ • **2016 WCTE: A Comparative Analysis of Three Methods Used For Calculating Deflections For Multi-storey Wood Shear Walls: Grant Newfield, Jasmine B. Wang**

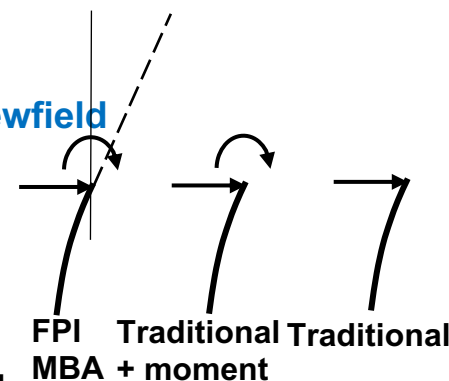
→ • **FPInnovations-Website**

**"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**





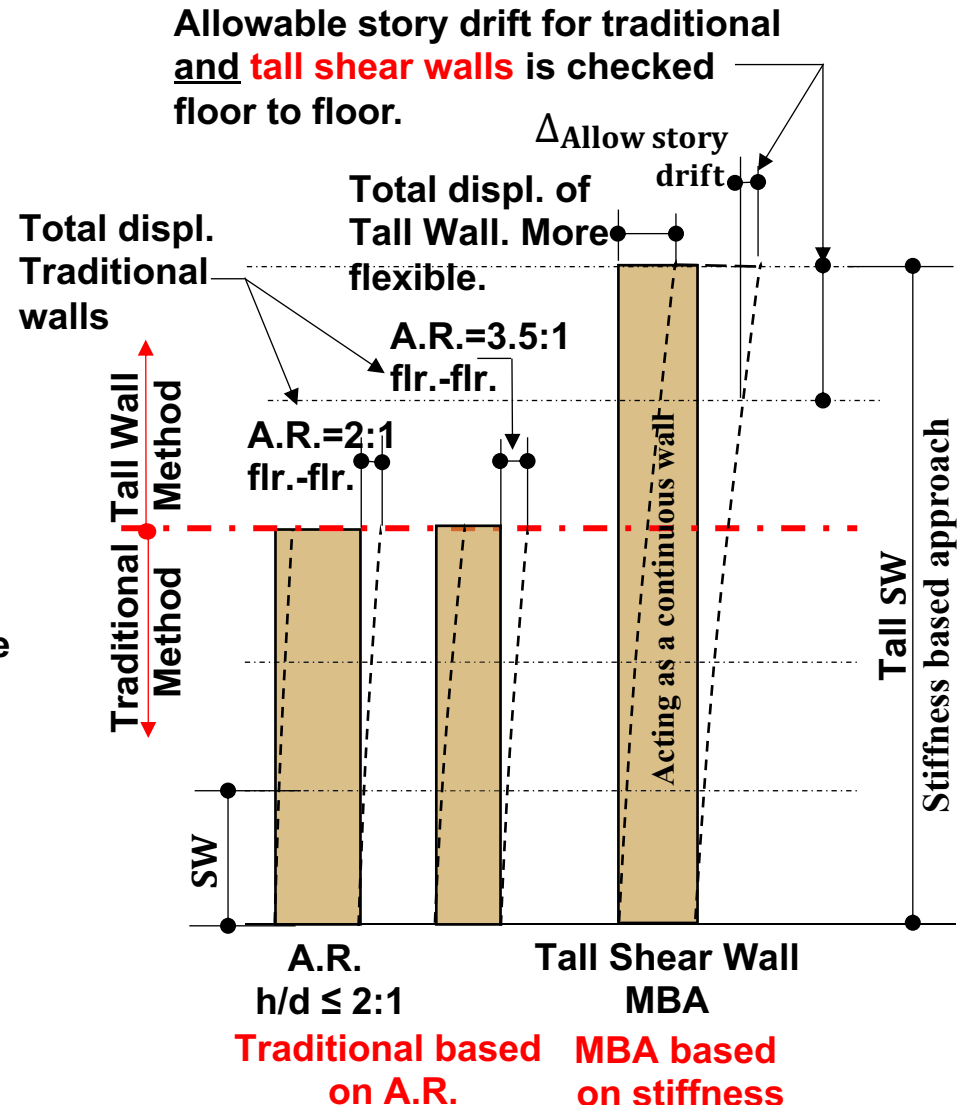
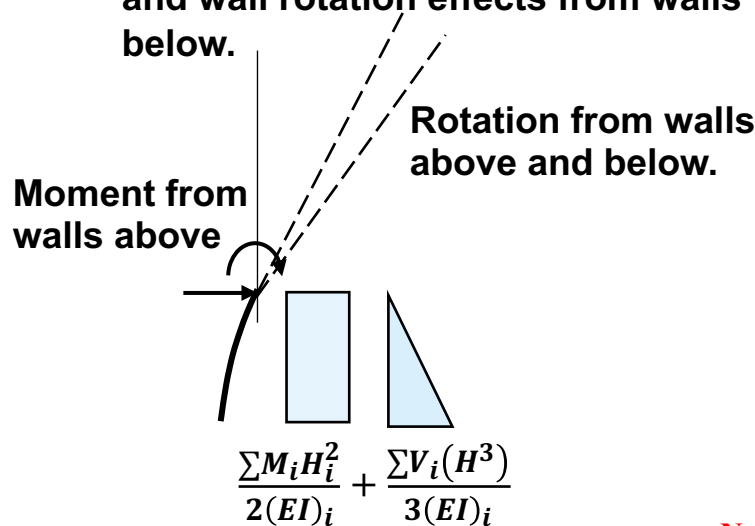
# New Research and Analytical methods-Tall Shear Walls

Currently not addressed or required by code:  
Engineering preference and/or judgement

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

(Dolan)

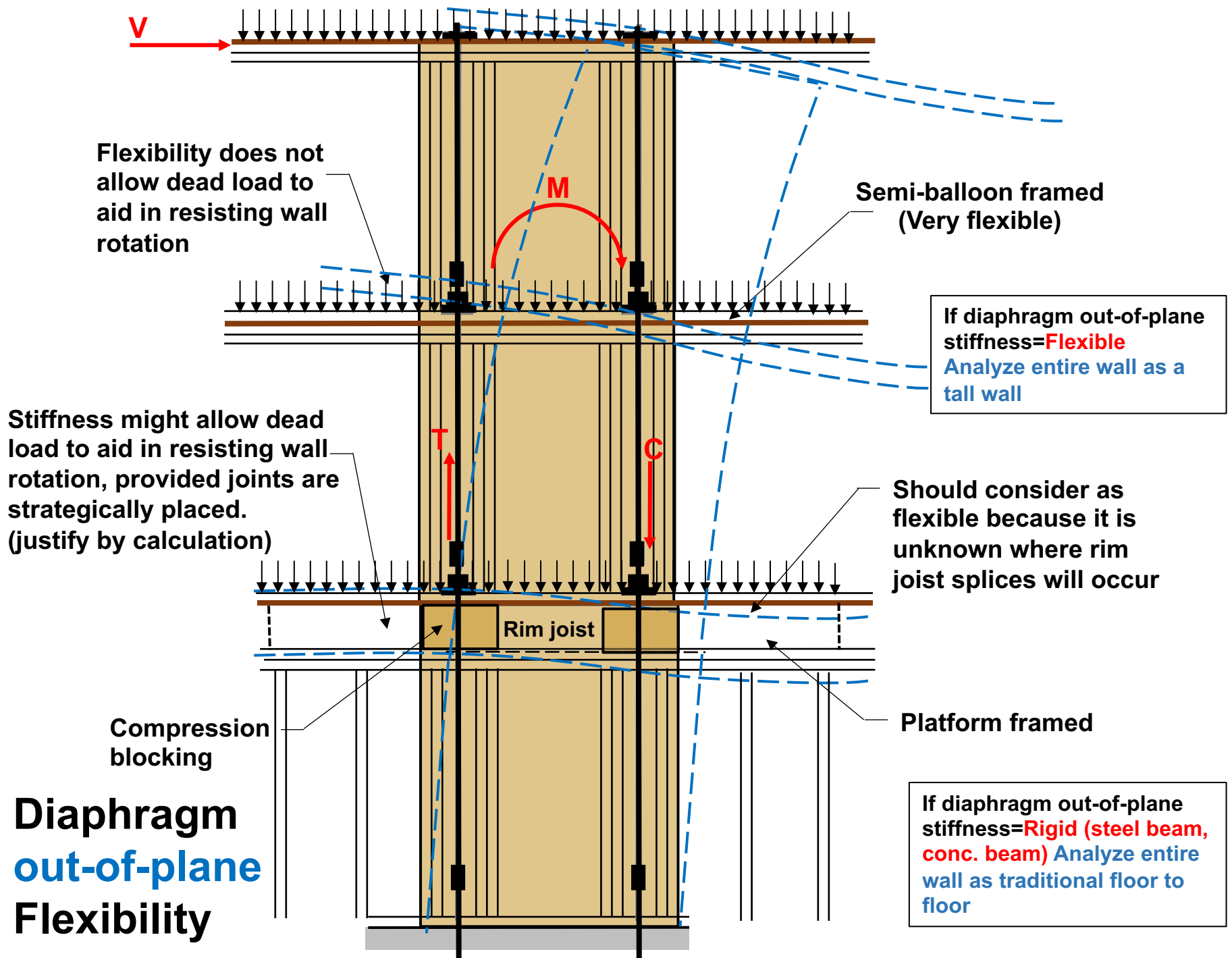
- Current research suggests that The traditional method of shear wall analysis might be more appropriate for low-rise structures.
- Multi-story walls greater than 3 stories should:
  - Consider flexure and wall rotation.
  - Rotation and moment from walls above and wall rotation effects from walls below.



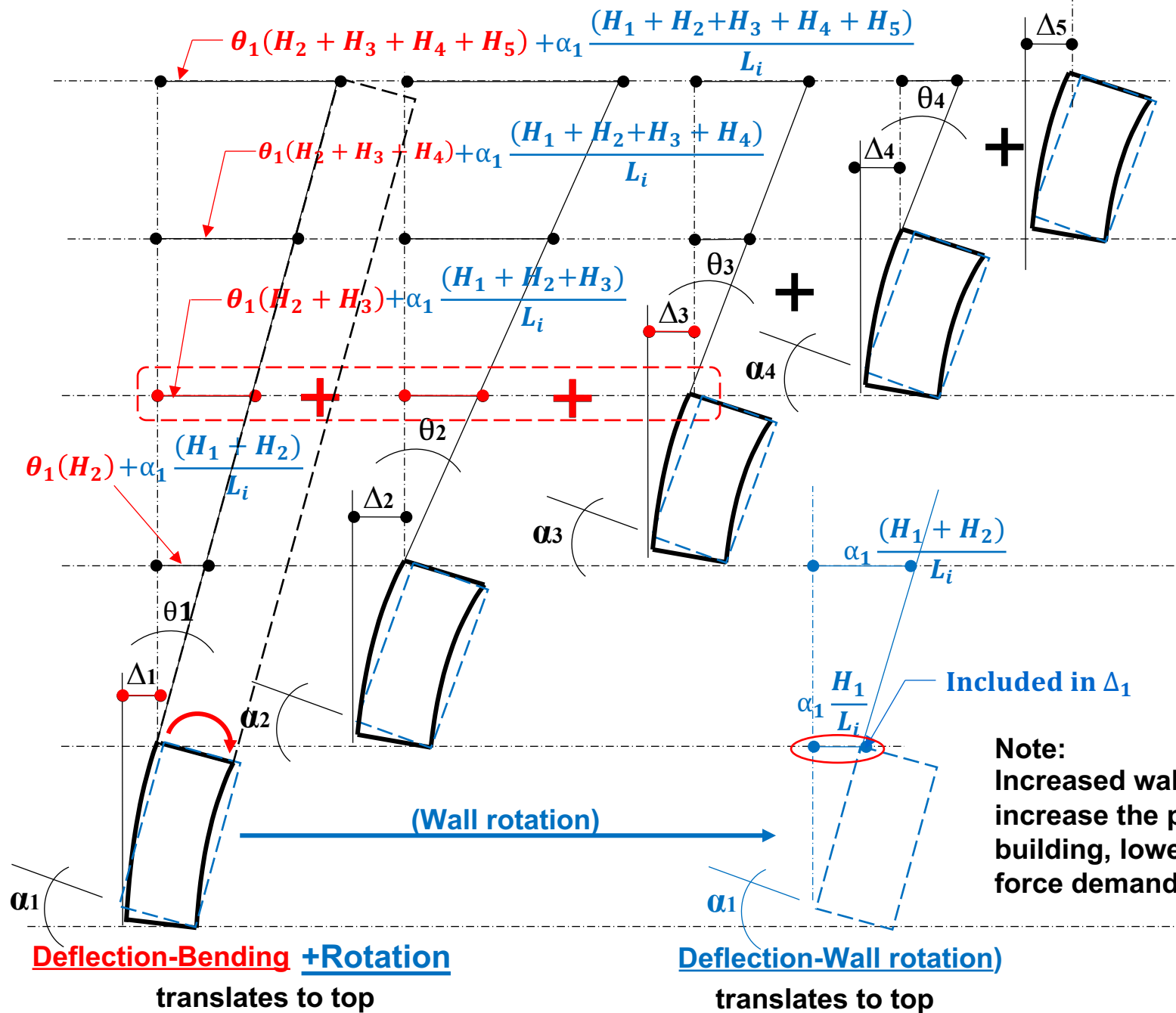
Floor to floor A.R.'s and Stiffness of Shear Walls

Not in example

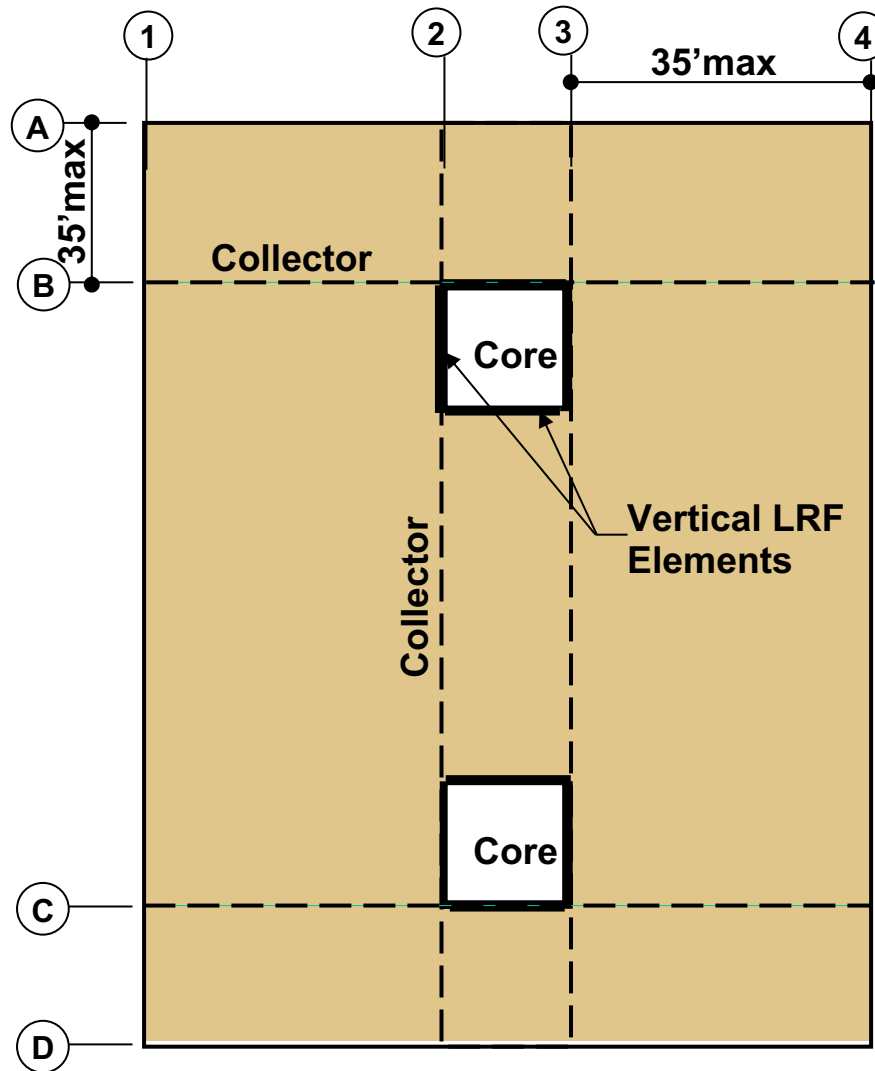
# Diaphragm out-of-plane Flexibility



**Tall Wall Deflection**  $\Delta_i = \frac{\sum M_i H_i^2}{2(EI)_i} + \frac{\sum V_i (H^3)}{3(EI)_i} + \frac{V_i H_i}{G_{v,i} t_{v,i}} + 0.75 H_i e_{n,i} + \frac{H_i}{L_i} d_{a,i} + H_i \sum_{j=1}^{i-1} \left( \frac{M_j H_j}{(EI)_j} + \frac{V_j H_j^2}{2(EI)_j} \right) + H_i \sum_{j=1}^{i-1} \frac{d_{a,j}}{L_j}$



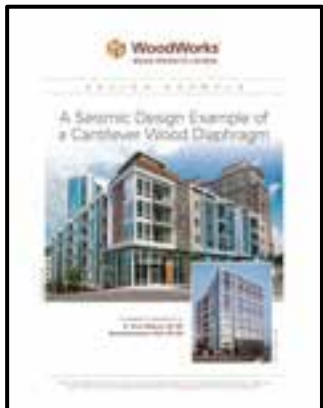
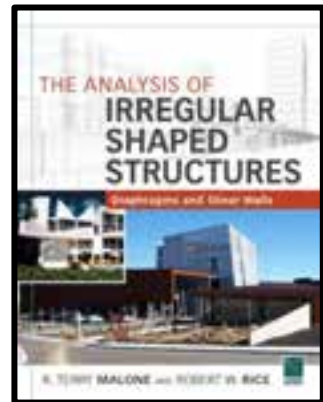
# Core Structures



- Light framed
- CLT

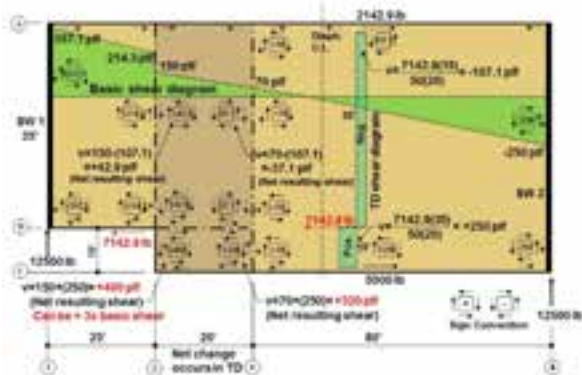
# Reference Materials

- **The Analysis of Irregular Shaped Structures: Diaphragms and Shear Walls-Malone, Rice-Book published by 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**
- **Woodworks-The Analysis of Irregular Shaped Diaphragms (paper). Complete Example with narrative and calculations.**  
[http://www.woodworks.org/wp-content/uploads/Irregular-Diaphragms\\_Paper1.pdf](http://www.woodworks.org/wp-content/uploads/Irregular-Diaphragms_Paper1.pdf)
- **Woodworks-Guidelines for the Seismic Design of an Open-Front Wood Diaphragm (paper). Complete Example**



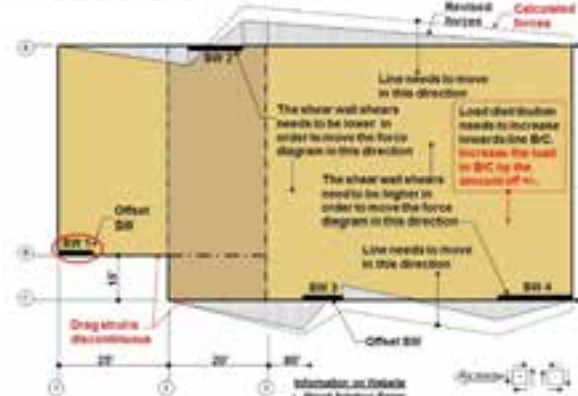
# Method of Analysis and Webinar References

## Offset Diaphragms



<https://vimeo.com/woodproductscouncil/review/114574994/b64da97f09>

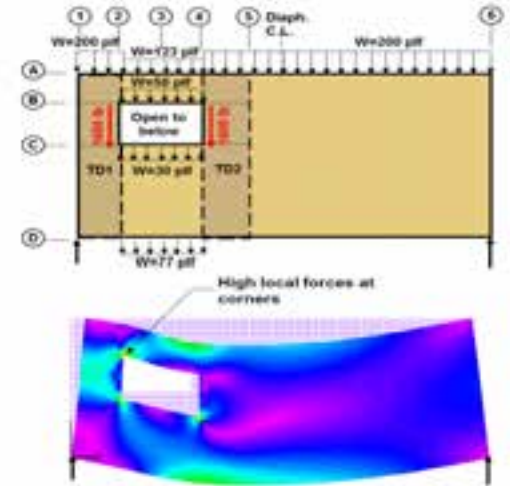
## Offset Shear Walls



### Example Results

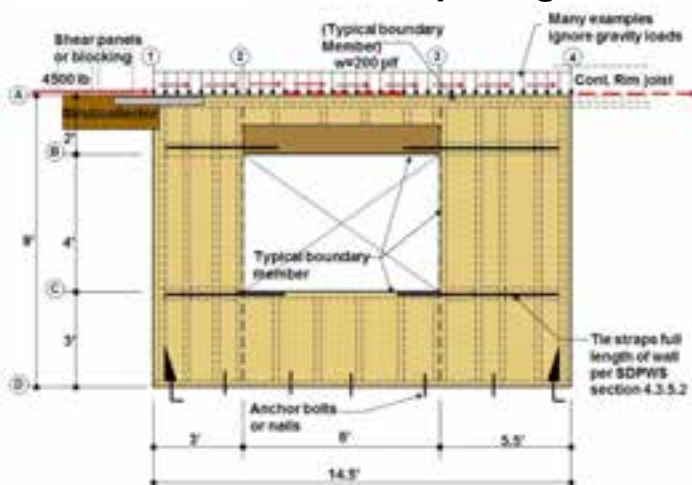
<https://vimeo.com/woodproductscouncil/review/149198464/c1183f2cf8>

## Diaphragm Openings



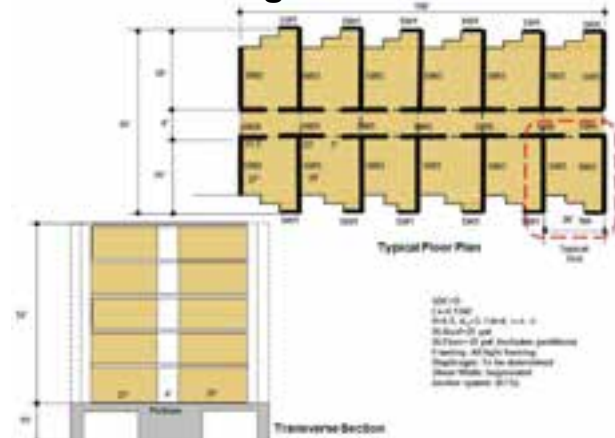
<https://vimeo.com/woodproductscouncil/review/212986898/17ca94ef6f>

## Shear Walls with Openings



<https://vimeo.com/woodproductscouncil/review/217888849/e3018a496a>

## Mid-rise Design Considerations



<https://vimeo.com/woodproductscouncil/review/220727334/516f37ce1e>

# Questions?

This concludes Woodworks Presentation on:

## Guidelines for the Seismic Design of an Open-Front Wood Diaphragm

**Your comments and  
suggestions are valued.  
They will make a difference.**

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928-775-9119**



**Thank You**

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