WoodWorks

Cantilever Wood Diaphragm Webinar Series

A Design Example of a Wood Cantilever Diaphragm

Part 3- Cantilever Diaphragm Design, And Flexibility and Drift Checks

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

Course Description

Part 3 of this series will take an in-depth look at cantilever diaphragm design processes. Using calculation steps per ASCE 7 and the 2015 edition of the American Wood Council's Special Design Provisions for Wind & Seismic (SDPWS), diaphragm force distribution will be used to illustrate diaphragm design procedures in an open front structure. Diaphragm chord forces, chord slip, and chord design and detailing will all be examined. Diaphragm flexibility will also be analyzed, reviewing options proposed for the 2021 SDPWS and other rational, engineering judgement-based approaches. Finally, diaphragm deflection and story drift calculations will be explained, emphasizing factors that contribute to story drift in a cantilever diaphragm, and measurement criteria.

Learning Objectives

1. **Discuss evolutions in mid-rise building typology that have led to the need for open-front diaphragm analysis.**
2. **Review diaphragm flexibility provisions in ASCE 7 and the 2015 Special Design Provisions for Wind & Seismic (SDPWS).**
3. **Explore one option for open-front diaphragm analysis under seismic and wind loading in a wood-frame structure.**
4. **Highlight how to calculate story drift, diaphragm deflection and torsional irregularities, and discover their effects on load distribution through a cantilever diaphragm structure.**

Fasten Your Seatbelts

5 out of 5 Calculators

WoodWorks **Example and Method of Analysis:**

- **The solutions paper and this webinar were developed independently from the AWC task group for open-front diaphragms. 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.**
- **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 and engineers.**

Cantilever Wood Diaphragm Webinar Series-Content

Webinar Part 1- Code Requirements and Relative Stiffness issues:

- Introduction
- Questions needing resolution
- Horizontal distribution of shear and stiffness issues
- 2015 SDPWS open-front requirements
- Review preliminary design assumptions

Webinar Part 2- Shear Wall Design in Cantilever Diaphragm Structures:

- Introduction to open-front example
- Calculation of seismic forces and distribution
- Preliminary shear wall design
- Nominal shear wall stiffness
- Verification of shear wall design

Webinar Part 3- Cantilever Diaphragm Design, Flexibility and Drift Checks :

- Diaphragm design
- Maximum diaphragm chord force
- Diaphragm flexibility
- Story drift

Webinar Part 4-Torsional Irregularity, Other Design Checks, and Final Comments :

- Amplification of accidental torsion
- Redundancy
- Transverse direction design
- Multi-story shear wall effects

Contents and Learning Objectives

Diaphragm Design, Flexibility Check, and Drift:

- **Diaphragm design**

  Calculate the maximum diaphragm design force and learn various ways to distribute torsional forces into a diaphragm.

- **Maximum diaphragm chord force**

  Discover the maximum chord force if shear walls are located along the chord line or concentrated in-plane wall loads are located at the ends of a cantilever.

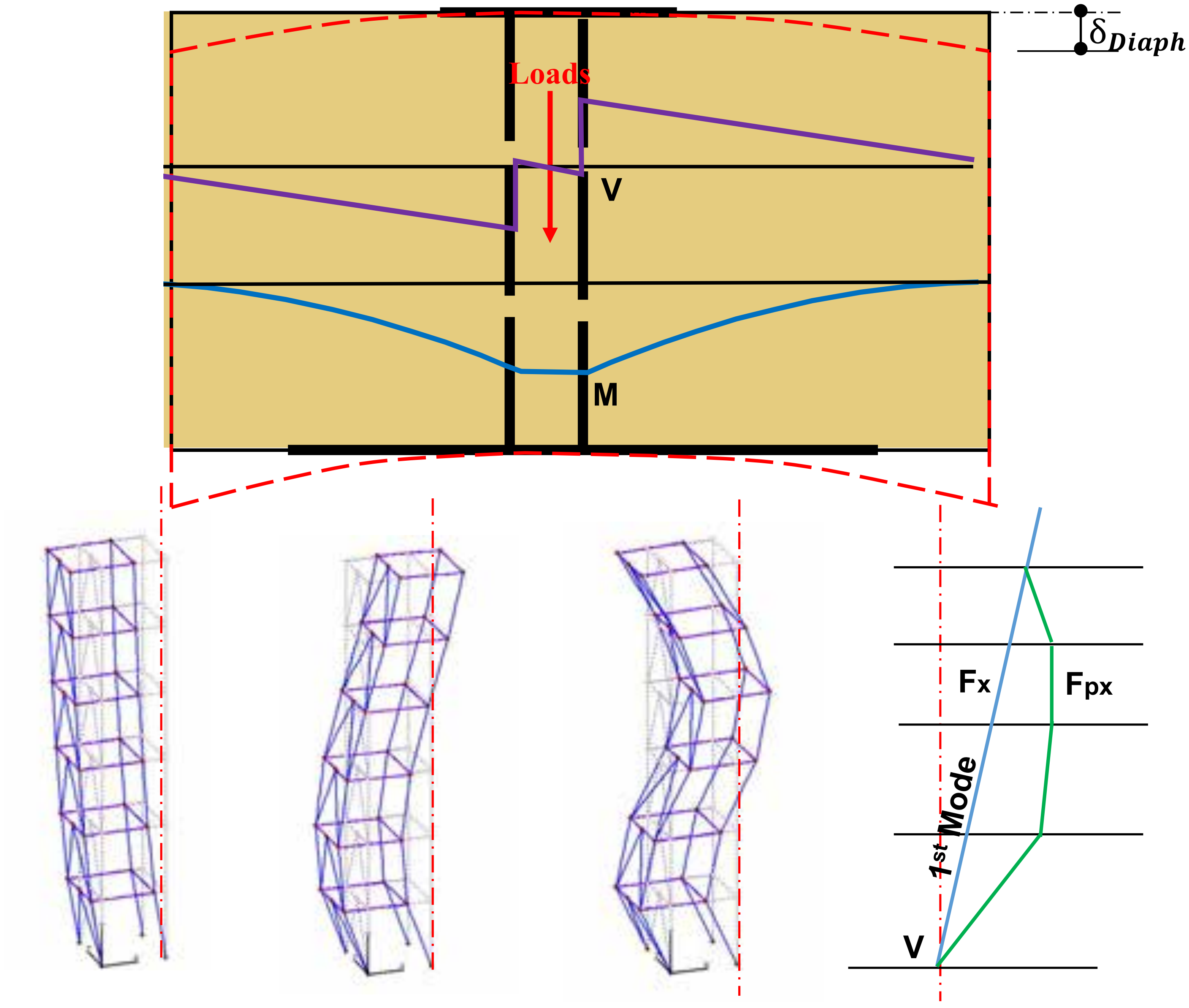
- **Diaphragm flexibility**

  Cantilever diaphragm deflection equations will be presented. Learn how to determine cantilever diaphragm flexibility.

- **Story drift**

  The method of calculating story drift for a cantilever diaphragm under seismic or wind loading will be presented. Options for reducing story drift or torsional irregularities will be provided if allowable drift is exceeded.

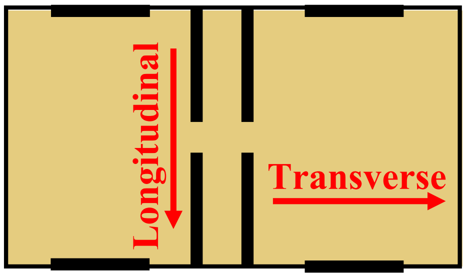
Diaphragm Design



Diaphragm Design Forces: MSFRS or F_{px}

Analysis Flow

Longitudinal Design

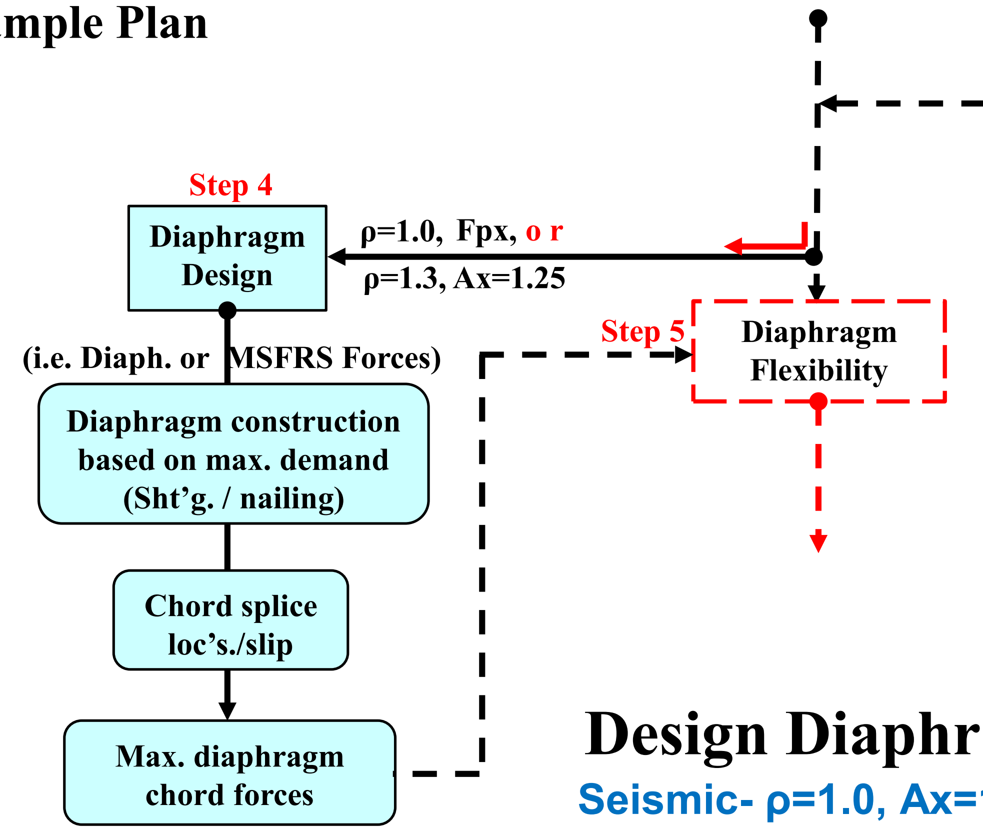


Example Plan

Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg., ρ, Ax

ASD Design | STR Design



Design Diaphragm

Seismic- ρ=1.0, Ax=1.25 or ρ=1.3, Ax=1.25

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?

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:**

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

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

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

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

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

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

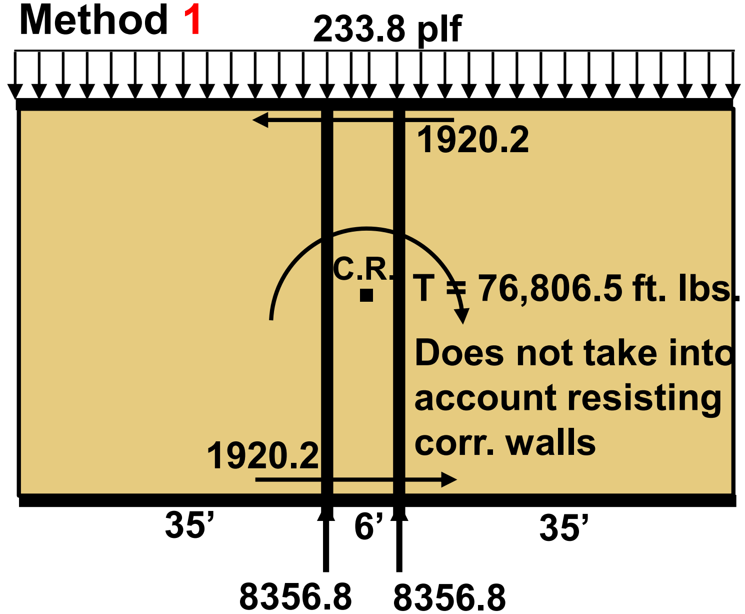
Torsional Distribution-Not mandatory

(Question 4) ρ=1.3, Ax=1.25

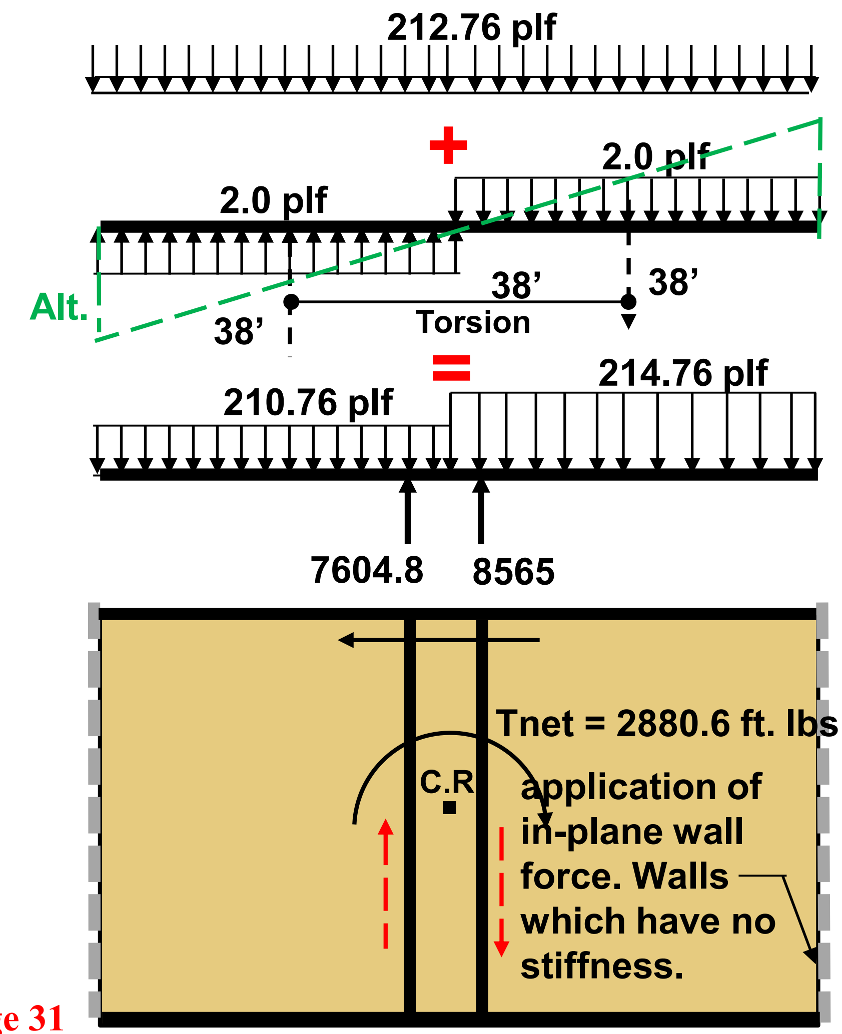
Method 2B will be used for diaphragm design
(To answer questions 5 and 6)

Method 2A will be used for all other checks

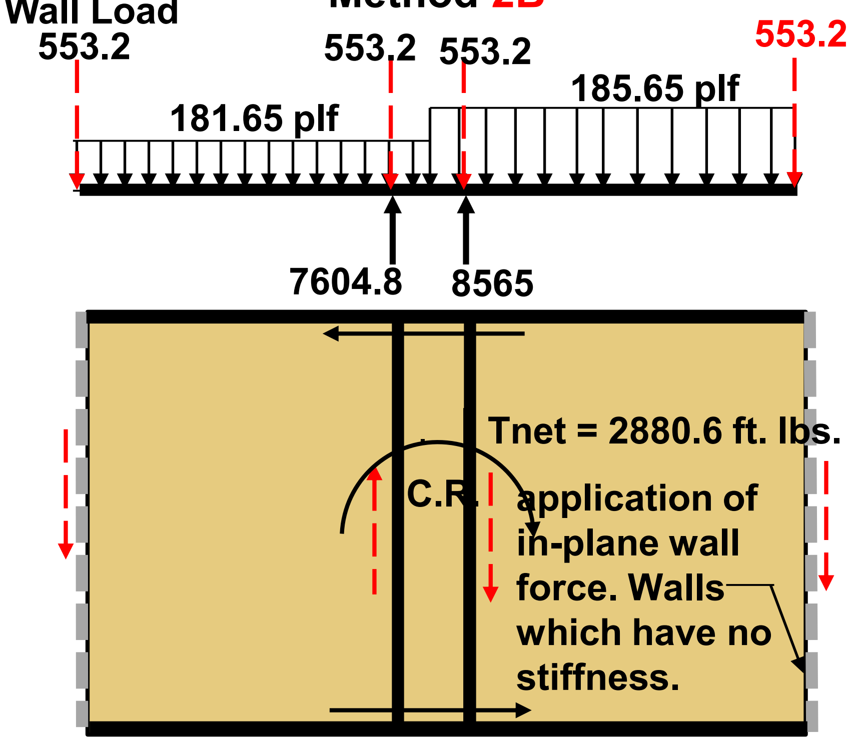
Method 1



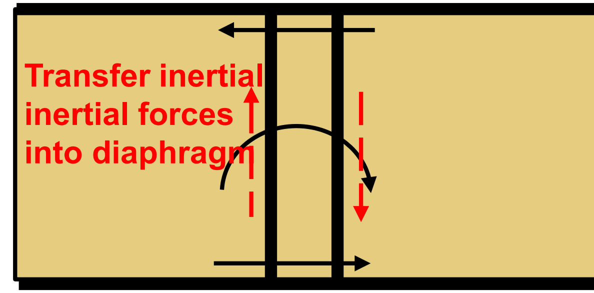
Method 2A



Method 2B

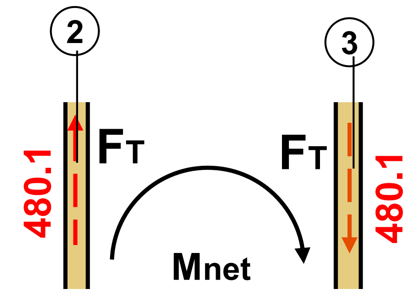


Using method 2B- ρ=1.3, Ax=1.25 :



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

M_{net} = 480.1(6 ft.) = 2880.6 ft. lbs. Net moment



Corridor walls

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 lbs.

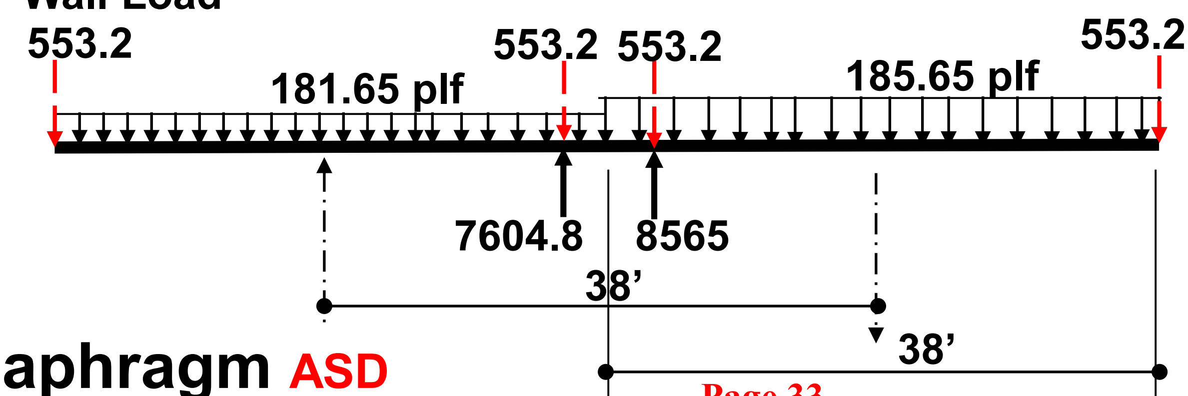
$W = \frac{13957}{76}$ = 183.65 plf uniform load

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

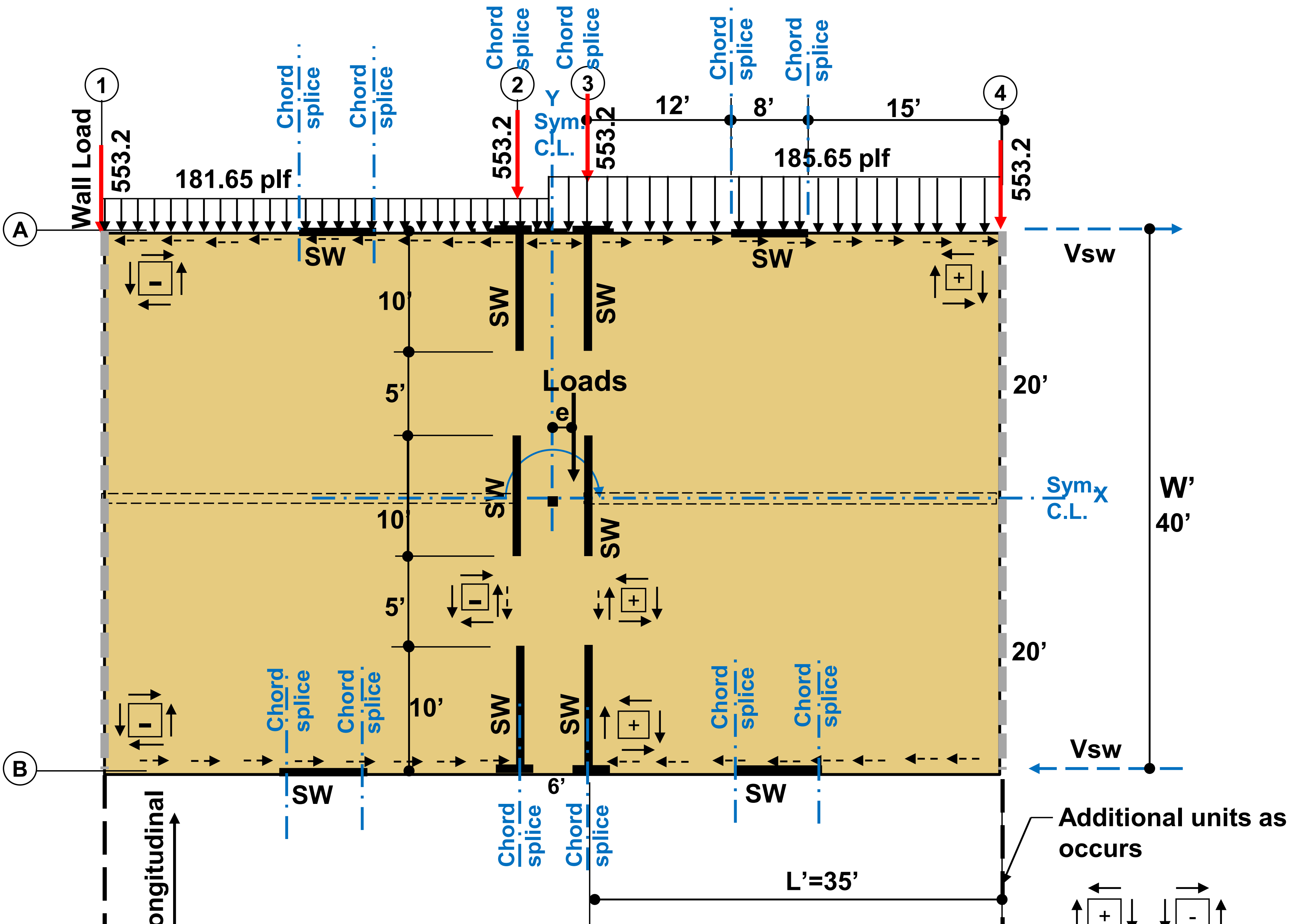
W1 = 183.65 – 2.0 = 181.65 plf: uniform load minus torsional load=net uniform load left cantilever

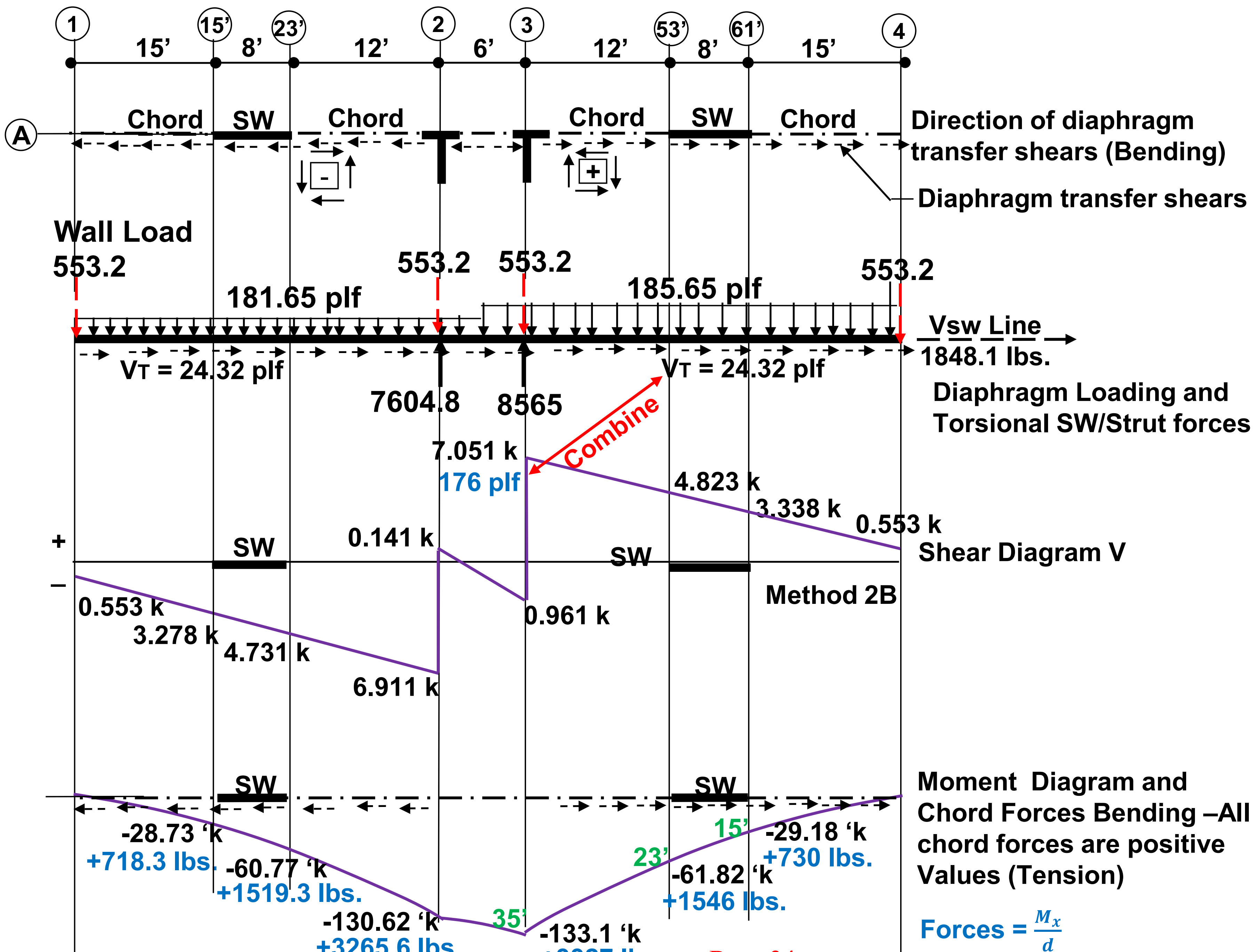
W2 = 183.65 + 2 = 185.65 plf
Right cantilever

Wall Load



Calculate Loads to Diaphragm ASD


Wall Load
553.2
181.65 plf
185.65 plf
12'
8'
15'
Chord splice
Y
Sym.
C.L.
SW
Vsw
10'
5'
Loads
e
20'
Sym.
X
C.L.
W'
40'
6'
Additional units as occurs
L'=35'
L=76'
Longitudinal
Transverse
Sign Convention
A
B
1
2
3
4
Method-2B


1
15'
15'
8'
23'
12'
2
6'
3
12'
53'
8'
61'
15'
4
A
Chord
SW
Chord
Chord
SW
Chord
Direction of diaphragm transfer shears (Bending)
Diaphragm transfer shears
Wall Load
553.2
553.2
553.2
553.2
181.65 plf
185.65 plf
Vsw Line
1848.1 lbs.
VT = 24.32 plf
VT = 24.32 plf
7604.8
8565
Diaphragm Loading and Torsional SW/Strut forces
Combine
7.051 k
176 plf
4.823 k
3.338 k
0.553 k
0.141 k
SW
SW
Shear Diagram V
Method 2B
0.553 k
3.278 k
4.731 k
6.911 k
0.961 k
SW
SW
Moment Diagram and Chord Forces Bending –All chord forces are positive Values (Tension)
-28.73 'k
+718.3 lbs.
-60.77 'k
+1519.3 lbs.
-130.62 'k
+3265.6 lbs.
35'
-133.1 'k
+3327 lbs.
23'
15'
-61.82 'k
+1546 lbs.
-29.18 'k
+730 lbs.
Forces = $\frac{M_x}{d}$

Diaphragm Capacity-Wood Structural Panels

Blocked

Table 4.2A Nominal Unit Shear Capacities for Wood-Framed Diaphragms[1,3,6,7]

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 other panel edges(cases 1, 2, 3 & 4															
					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.)		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
				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

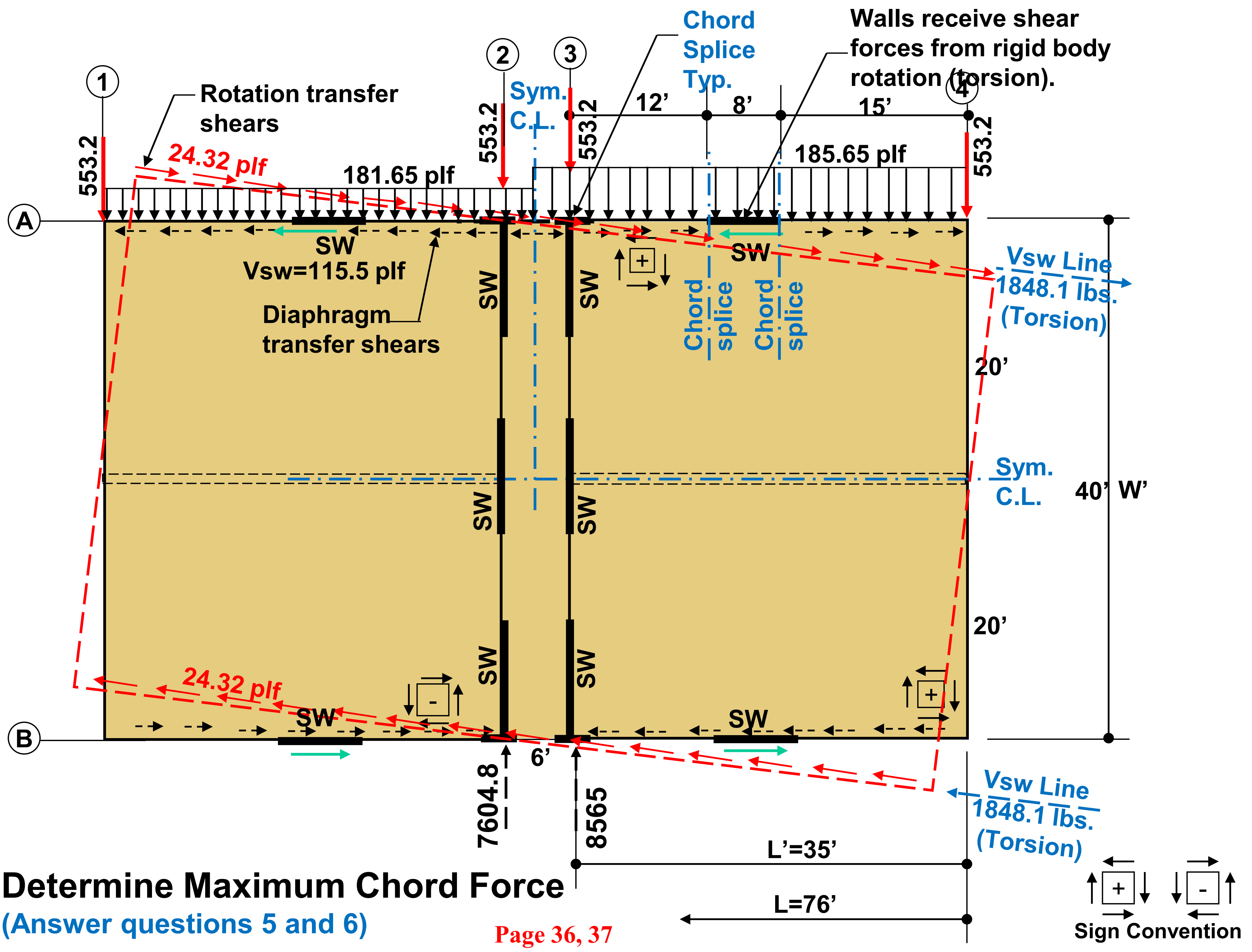
Roof framing-D.F. 1, E = 1,700,000 psi, roof joists @ 16" 0.c.

$\text{Unit torsional shear} = 24.32 \text{ plf}$

$v_{Max\ diaph}$ = 176.3 + 24.3 = 200.6 plf.

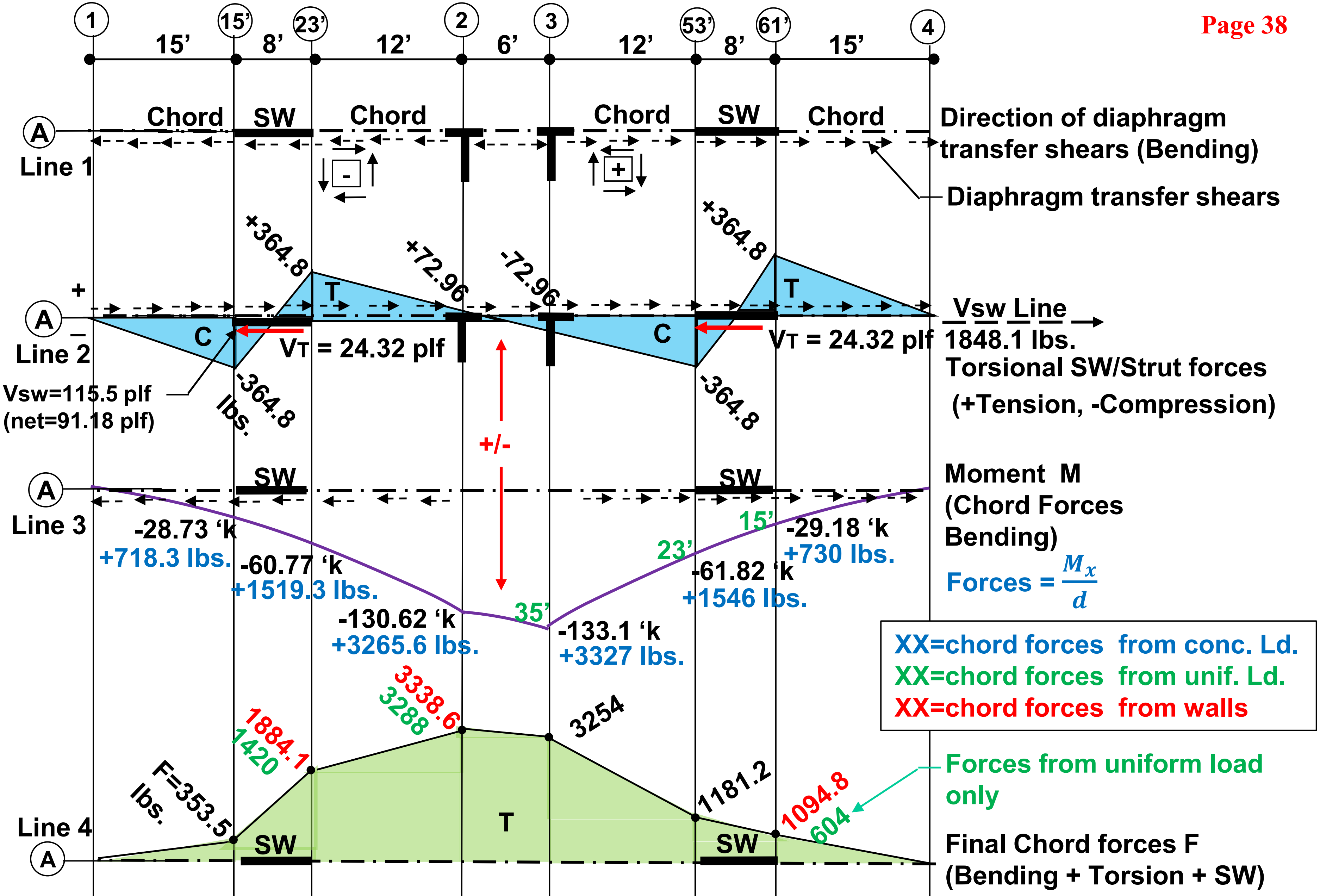
200.6 plf < 0.5(580) = 290 plf. o.k.

G_a = 25, blocked



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.

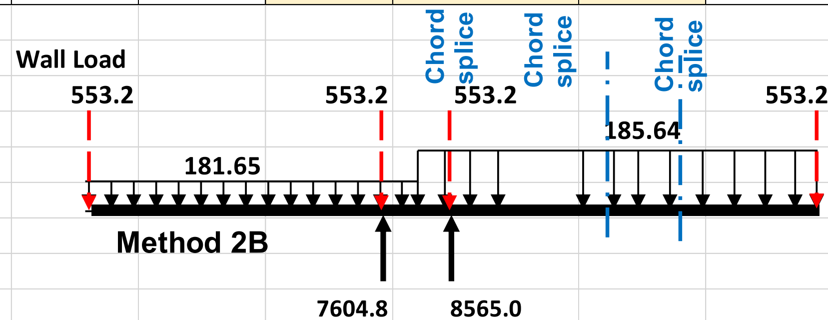
Diaphragm Chords

Diaphragm Deflection (ASD)

	Splice Forces (Lbs.)			Σδ_slip	v unif.	v conc.	Ga	L'	W'	δDiaph Uni	δDiaph conc	Total δ	
	F 15	F23	F35	In.	plf	plf	k/in.	Ft.	Ft.	In.	In.	In.	
	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

Iincludes effects of sw's along chord line



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										

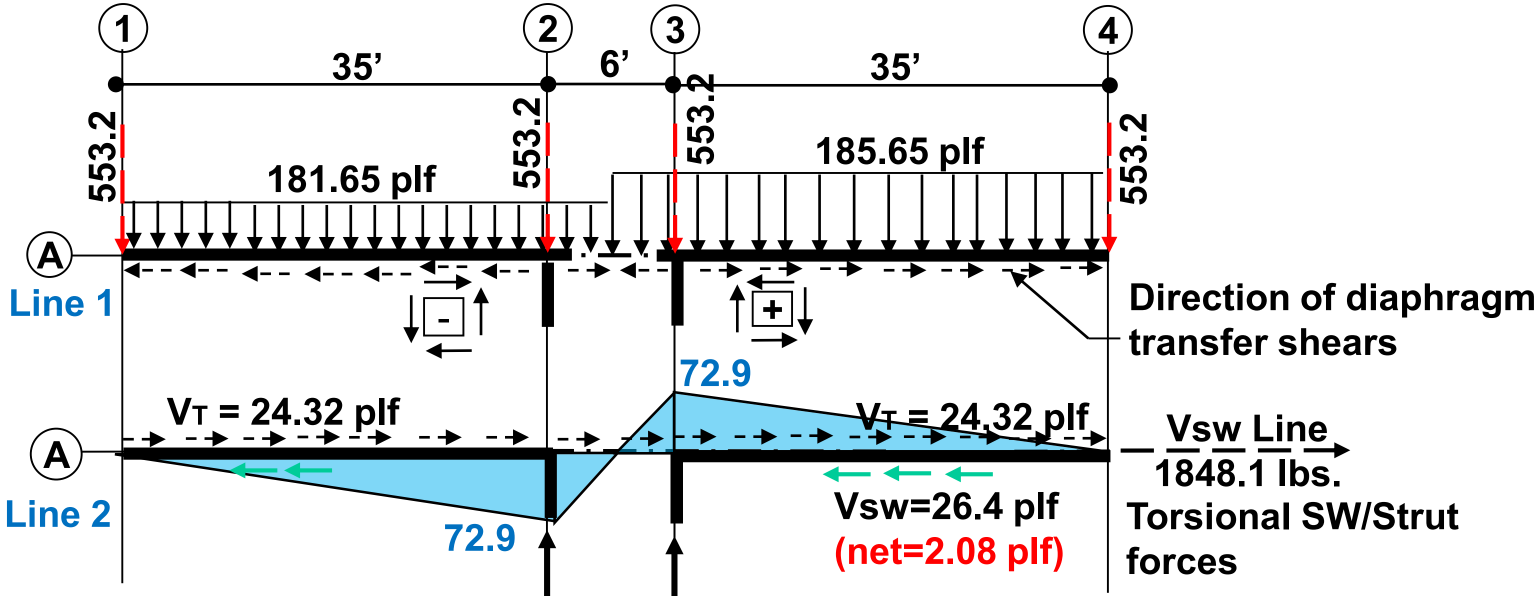
Maximum chord force = 3338.5 lbs.

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

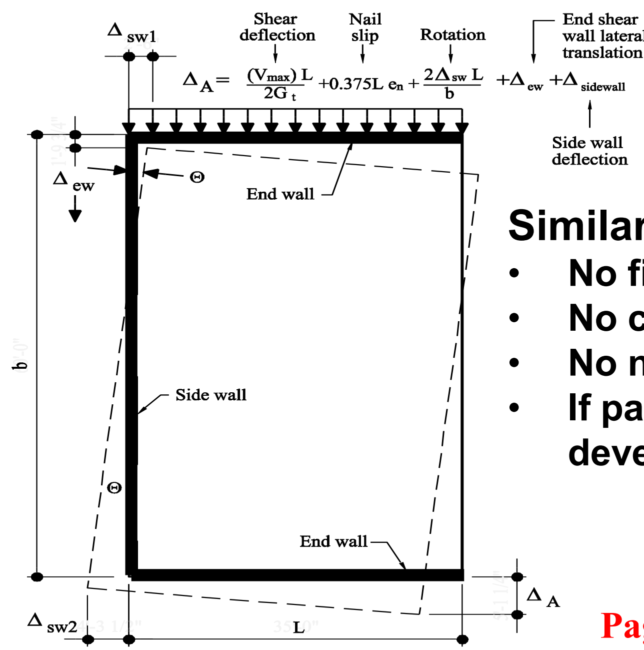
$f_t = \frac{F_{chord}}{(1)2x6}$, **Number of nails =** $\frac{F_{chord}}{226}$ **, where 226 lbs. is adjusted lateral design value, Z' (ASD), for 16d nails (face nailed).**

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

Check for Effects of Full Length Shear Walls on Chord Forces



Uniform torsional shears vs. Shear Wall Shears

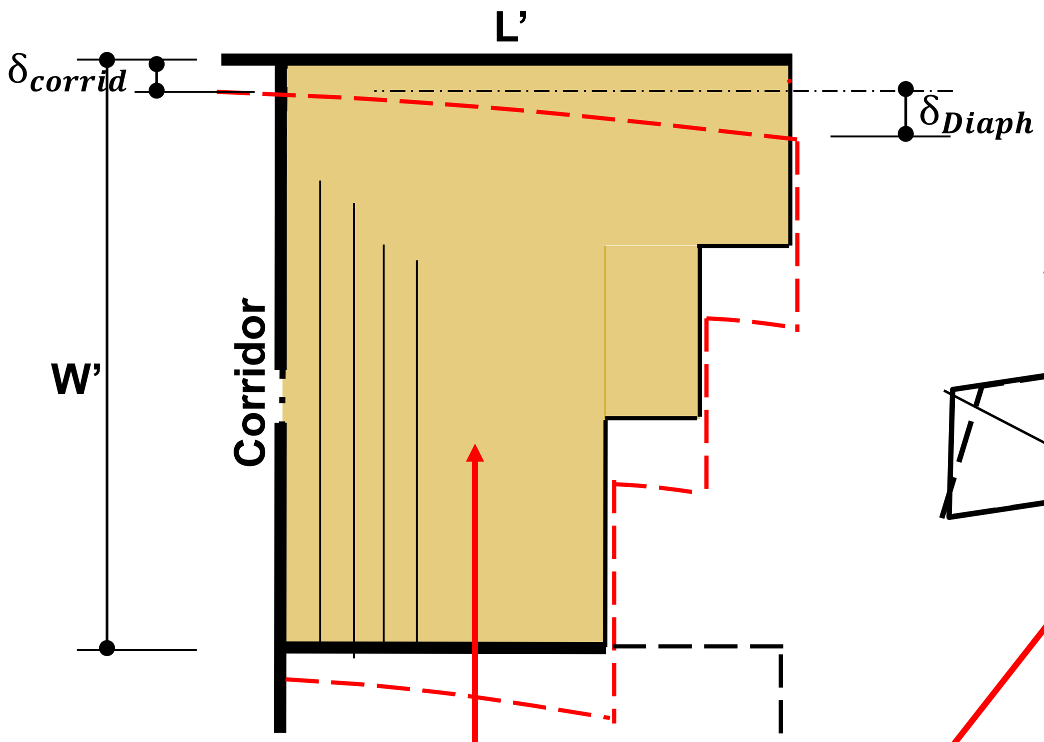


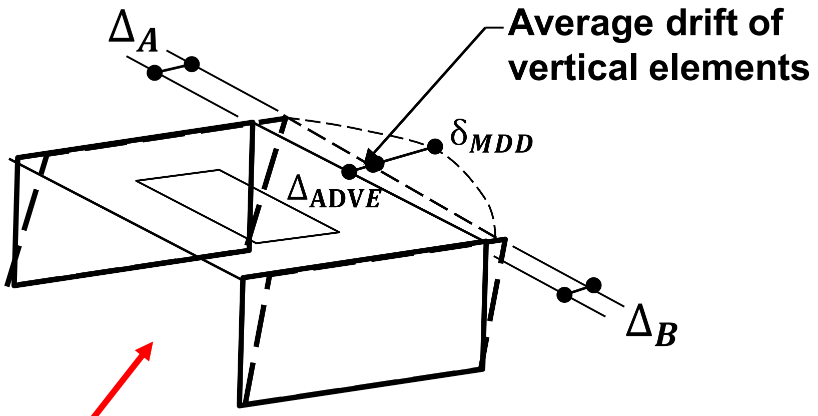
Similar to APA Example

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

Page 64

Diaphragm Flexibility, ρ=1.0, Ax=1.25



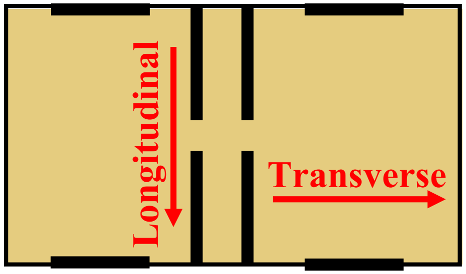


ASCE 7-16 Figure 12.3-1

How does this relate to this?

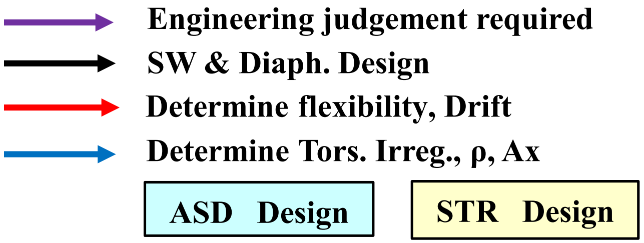
Analysis Flow

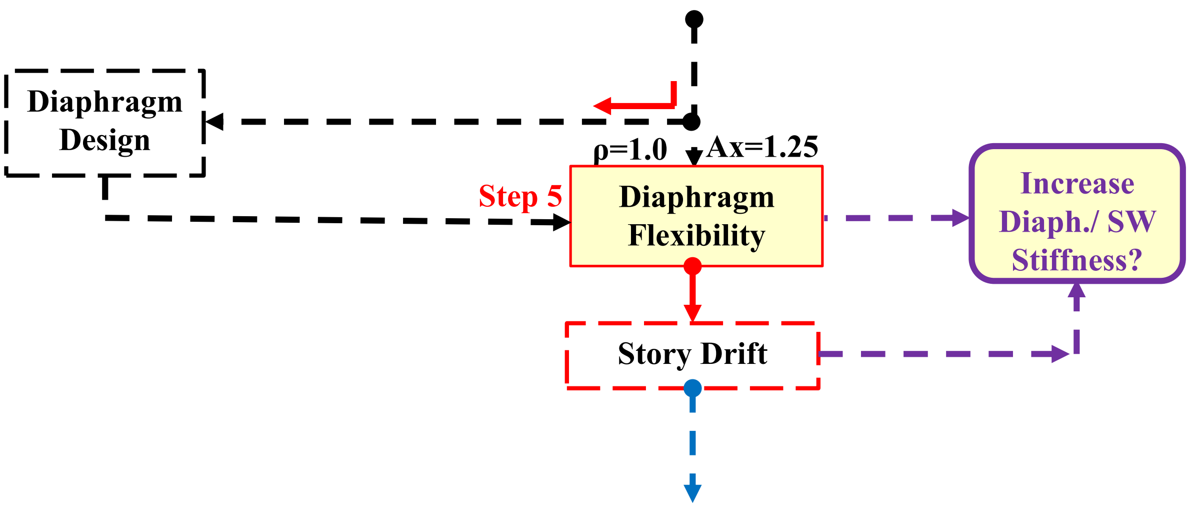
Longitudinal Design



Example Plan

Legend





Check Diaphragm Flexibility

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

- ASCE 7-16

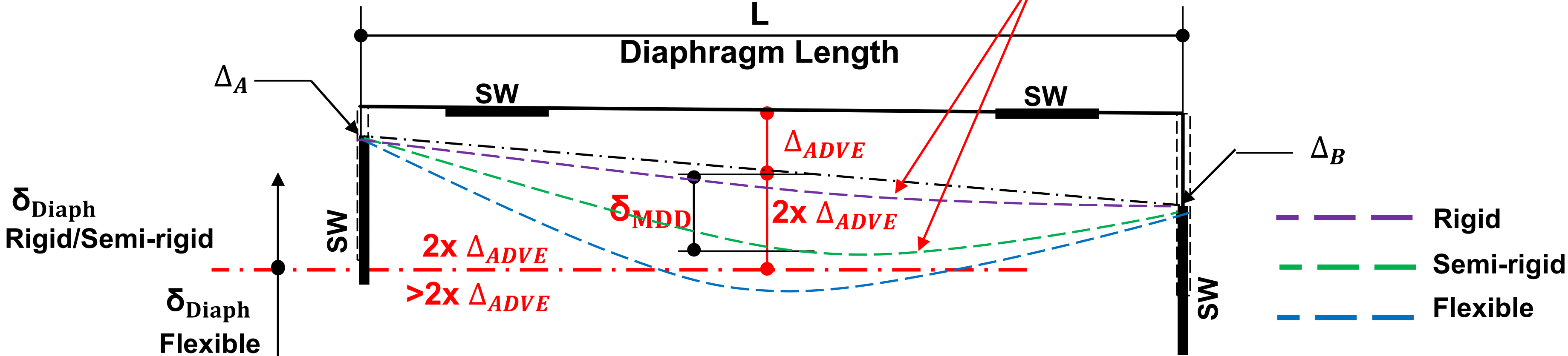
Diaphragm Flexibility

- **12.3.1.1 Flexible Diaphragm Condition.**
  - **Untopped steel decking or wood structural panels**
  - **Permitted to be idealized as flexible under certain conditions.**
- **12.3.1.2 Rigid Diaphragm Condition.**
  - **Concrete slabs or concrete-filled metal deck (No mention of wood)**
  - **Span-to-depth ratios of 3 or less with no horizontal irregularities**
  - **Permitted to be idealized as rigid.**
- **12.3.1.3 Calculated Flexible Diaphragm Condition. (No calculated Rigid condition)**
  - **Diaphragms not satisfying the conditions of Sections 12.3.1.1or 12.3.1.2**
  - **Permitted to be idealized as flexible provided: $\delta_{MDD} > 2\Delta_{ADVE}$.**

- **2018 IBC Section 1604.4:**
  - **A diaphragm is rigid when $\delta_{MDD} \leq 2\Delta_{ADVE}$.**

- **2015 SDPWS 4.2.5 Horizontal Distribution of Shear**
  - **Idealize as rigid when computed $\delta_{MDD} \leq 2\Delta_{ADVE}$**

To What Degree, Rigid or Semi-rigid?

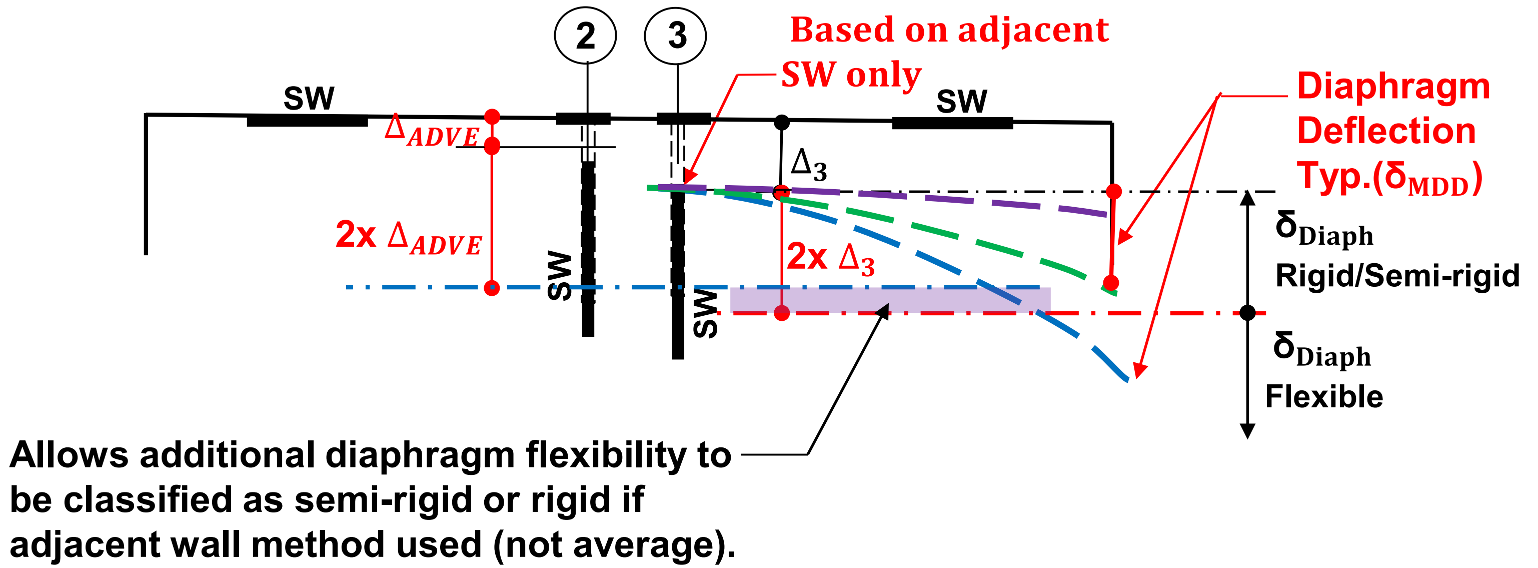


(a) ASCE 7-16 Figure 12.3-1

Simple Span Diaphragm

Determination of Cantilever Diaphragm Flexibility (Question 3):

Page 42



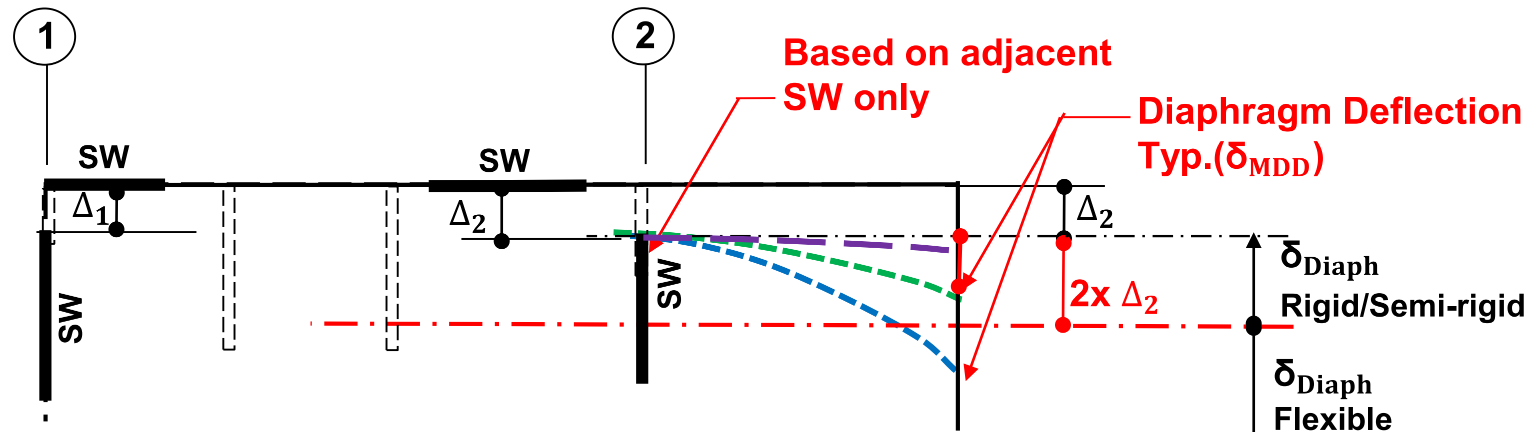
(b) Corridor Walls Only

Preferred Method

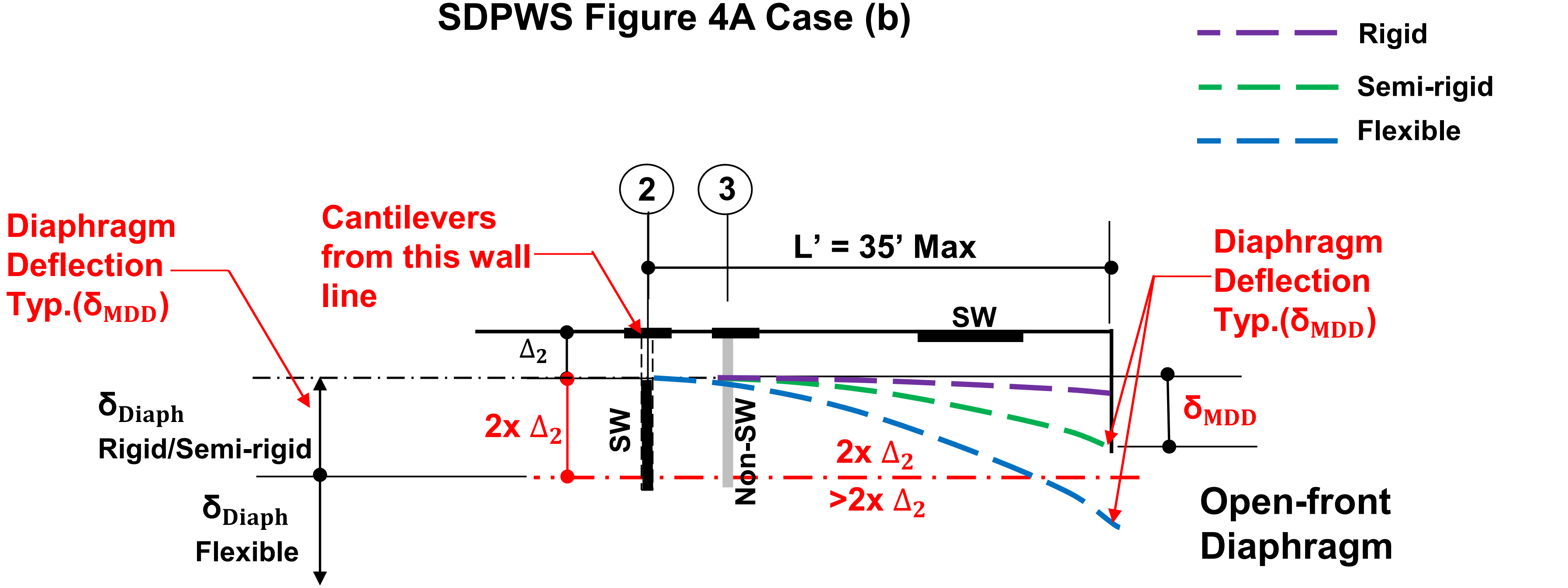
Can require engineering judgement

ATC WDSC -Proposed language

Cantilevered diaphragms shall be permitted to be idealized as rigid when the calculated maximum in-plane deflection of the diaphragm itself under lateral load is less than or equal to two times the deflection of vertical elements of the lateral force-resisting system of the associated story used to determine the cantilever length, L' (See Figure 4A).



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



(d) Diaphragm flexibility Shear Wall One Side

Cantilever Diaphragm Deflection Equations (Question 2):

AWC WDSC Ballot 4

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.2

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

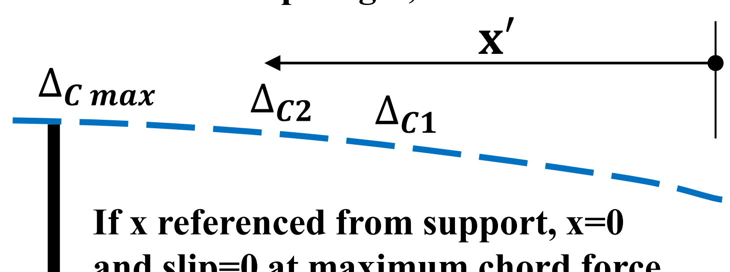
X' = distance from chord splice to the free edge of the diaphragm, ft

Δ_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., ρ=1.0, Ax=1.25										
Grid Line	kx	Ky	dx	dy	kd	kd^2	Fv		F_T	Fv+F_T
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

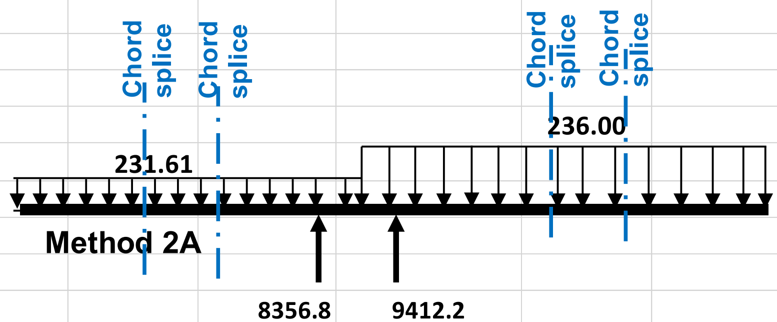
Diaphragm Deflection (STR)

Rt. Cantilever

	Splice Forces (Lbs.)			Σδ_slip	v unif.	v conc.	Ga	L'	W'	δDiaph Unif	δDiaph conc	Total δ
	F 15	F23	F35	In.	plf	plf	k/in.	Ft.	Ft.	In.	In.	In.
	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

Iincludes effects of sw's along chord line



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, ρ=1.0, Ax=1.25

$$\delta_{Diaph\ Unif} = \frac{3v_{max}L'^3}{EAW'} + \frac{0.5v_{max}L'}{1000G_a} + \frac{\Sigma A_C X_C}{W'}$$ **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”

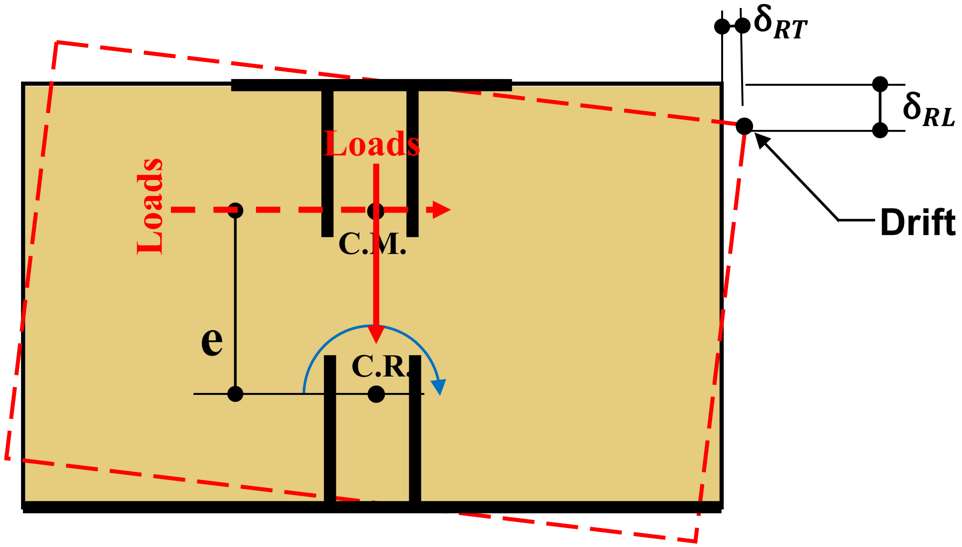
$2 \text{ x } \Delta_3 = 0.432$“

0.265” < 0.432” ∴ Diaphragm can be idealized as Rigid

Diaphragm Flexibility – Wind

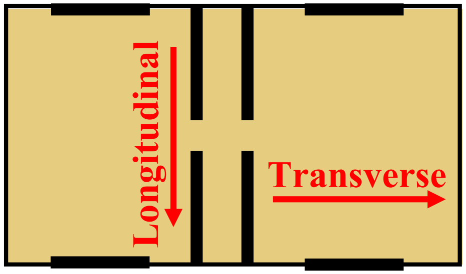
- **ASCE 7-16, Chapter 27, Section 27.5.4-DIAPHRAGM FLEXIBILITY-requires that the structural analysis shall consider the stiffness of diaphragms and vertical elements of the main wind force resisting system (MWFRS).**
- **Section 26.2 - Definitions, DIAPHRAGM, diaphragms constructed of WSP are permitted to be idealized as flexible.**
- **ASCE 7 drift limits requirements for wind design can be found in ASCE 7-16 Commentary Appendix CC, Serviceability Considerations**
  Section CC2.2 Drift of Walls and Frames

Story Drift, ρ=1.0, Ax=1.25



Analysis Flow

Longitudinal Design



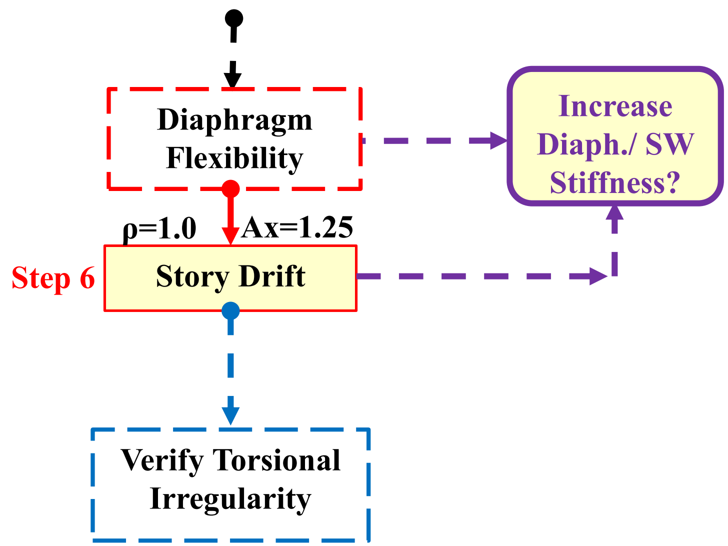
Example Plan

Legend

- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg., ρ, Ax

ASD Design

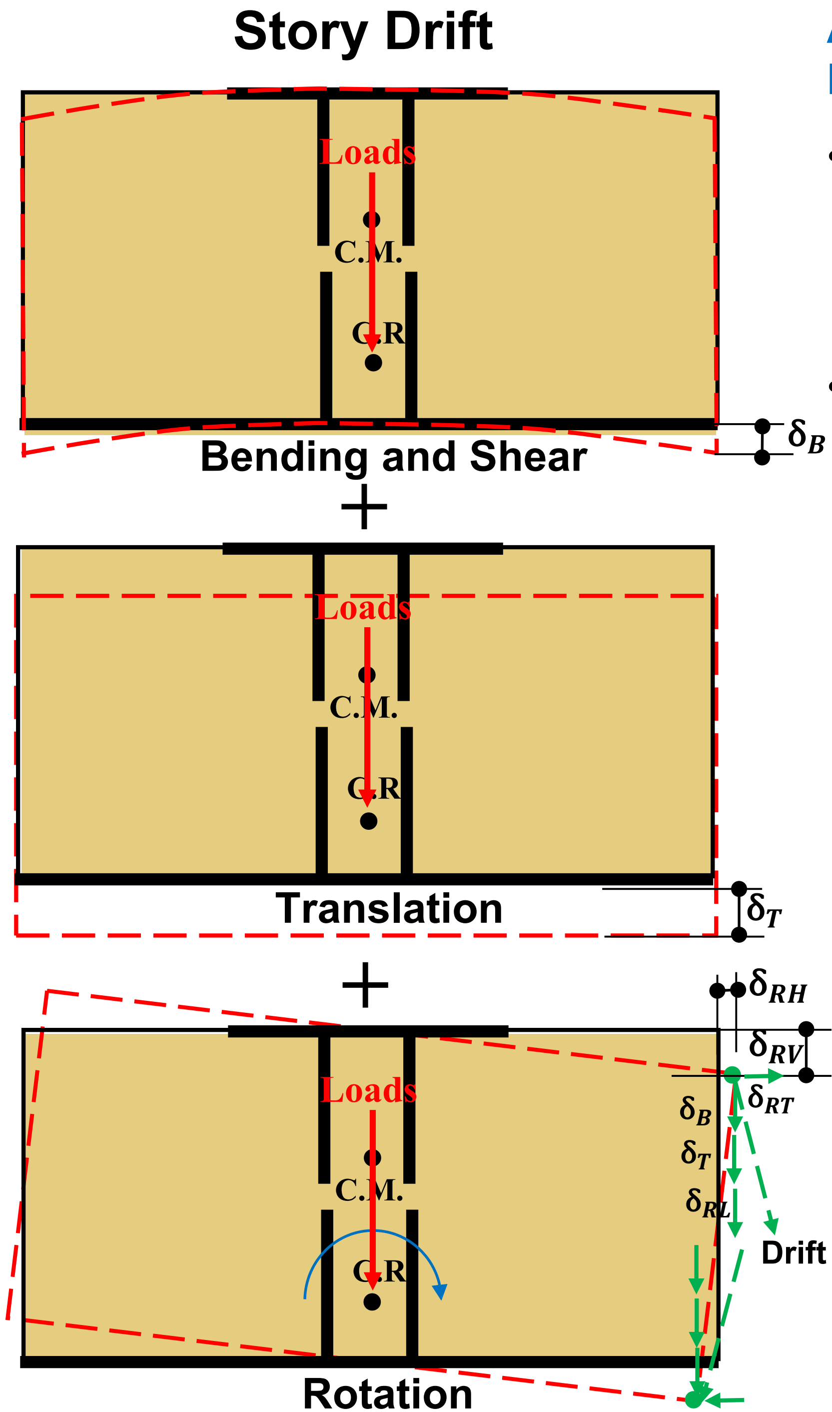
STR Design



Check Story Drift

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

Page 44



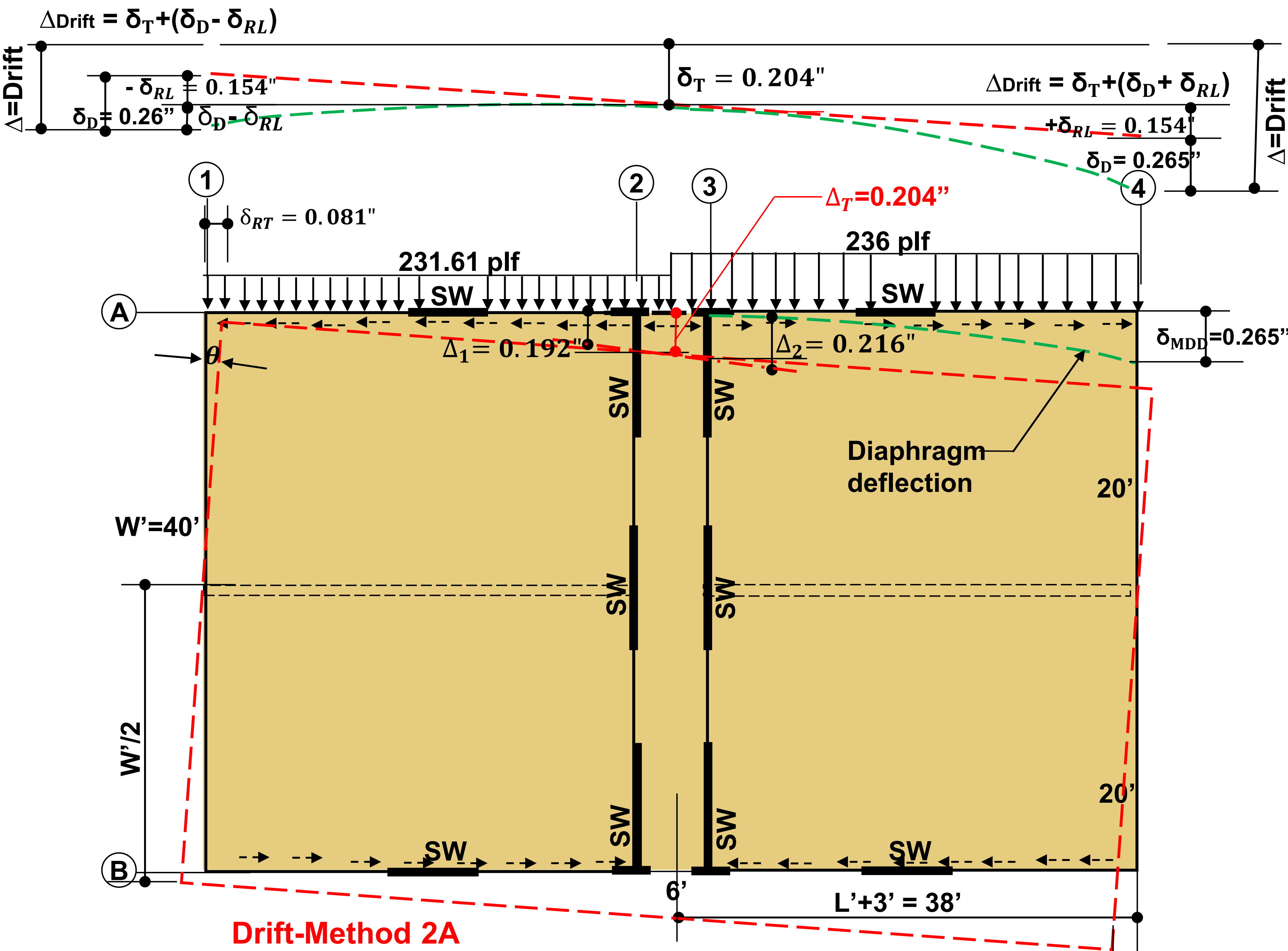
ASCE 7-16 Section 12.8.6-Story Drift Determination Regular structures:

- **Story drift (Δ) shall be computed as the difference of the deflections <u>at the centers of mass</u> at the top and bottom of the story under consideration (Fig. 12.8-2).**
- **For structures assigned to SDC C, D, E, or F that have <u>horizontal irregularity Type 1a or 1b</u> of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration <u>along any of the edges</u> of the structure.**

SDPWS Section 4.2.5.2 (4): Open-front structures, loading parallel to the open side:

- **Maximum story drift <u>at each edge</u> of the structure ≤ ASCE 7-16 allowable story drift (Seismic) including torsion and accidental torsion and shall include shear and bending deformations of the diaphragm computed - strength level basis amplified by C_d .**

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



ΔDrift = δT+(δD- δRL)
Δ=Drift
- δRL = 0.154"
δD= 0.26"
δD- δRL
δT = 0.204"
ΔDrift = δT+(δD+ δRL)
+δRL = 0.154"
δD= 0.265"
Δ=Drift
1
2
3
4
ΔT=0.204"
δRT = 0.081"
231.61 plf
236 plf
SW
A
Δ1= 0.192"
Δ2= 0.216"
δMDD=0.265"
θ
Diaphragm deflection
20'
W'=40'
W'/2
20'
B
6'
L'+3' = 38'
Drift-Method 2A
δRT = 0.081"

Drift-Method 2A (Seismic) ρ=1.0, Ax=1.25

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

δ_2 = 8.357 k / 43.54 k/in = 0.192 in,

δ_3 = 9.412 k / 43.54 k/in = 0.216 in

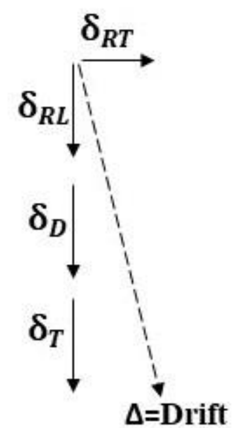
δ_A = 2.031 k / 25.14 k/in = 0.081 in,

δ_B = -2.031 k / 25.14 k/in = -0.081 in

Δ_{Diaph}= 0.265”

$\Delta_{Average}$= 0.204” (Translation)

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



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

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

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

Cd = 4, Ie = 1

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

δ_{RT} = Transverse component of rotation

δ_{RL} = Longitudinal component of rotation

δ_D=Diaphragm displacement

δ_T = Translational displacement

Table 12.12-1 Allowable Story Drift, Δ_a

Structure	Risk Category I or II	Risk Category III	Risk Category 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 exterior wall systems that have been designed to accommodate the story drifts.	0.025hsx	0.020hsx	0.015hsx
Masonry cantilever shear wall structures	0.010hsx	0.010hsx	0.010hsx
Other masonry shear wall structures	0.007hsx	0.007hsx	0.007hsx
All other structures	0.020hsx	0.015hsx	0.010hsx

- 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.
- 0.025hsx 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.

0.025hsx = 0.025(10)(12) = 3.0” > 2.51” ∴ drift O.K.

0.02hsx = 0.02(10)(12) = 2.4” < 2.51” ∴ drift not O.K. for 2% drift

Solutions if drift is exceeded: 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 Ga**

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 other panel edges (cases 1, 2, 3 & 4)															
					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.)		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
				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

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, Ga, 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 Δa) to lessen rod elongation and wall rotation.
- Increase the number of boundary studs (decrease bearing perpendicular to grain stresses, crushing). Optional LVL or LSL plates.
- 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:

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

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

Gvtv =35000 lb/in depth, 4-ply **SDPWS Table C4.2.2A**

v = 233.2 plf

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

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

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 ρ or Ax decreases (See *Section 7.6.1*).**

Check for Wind Drift

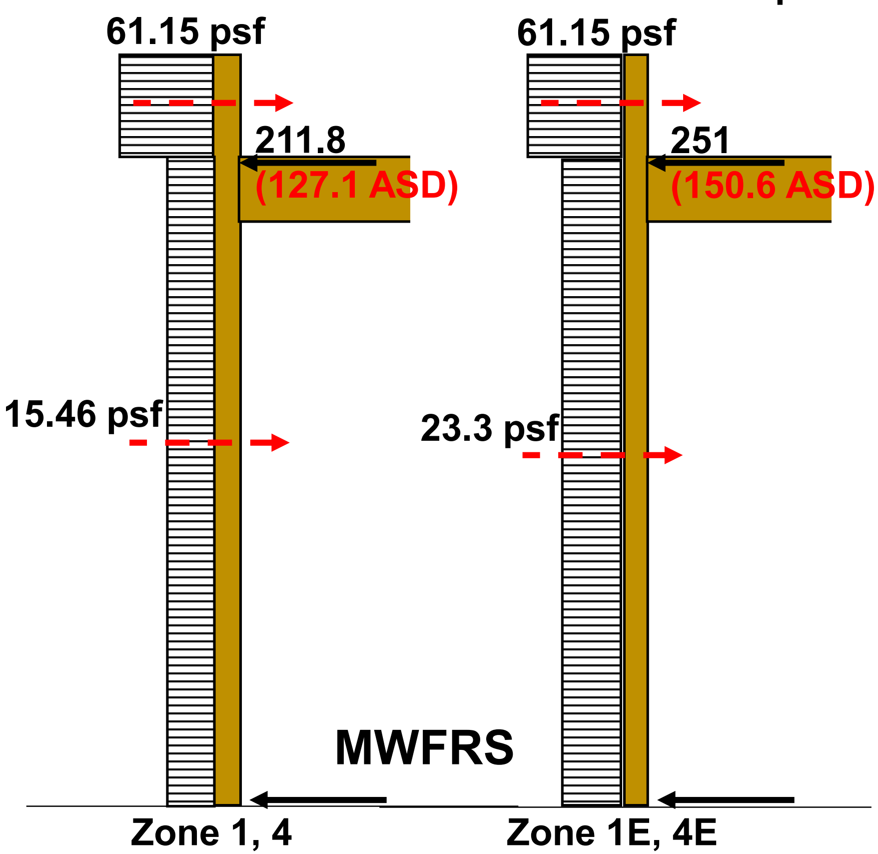
Simplified Procedure Chapter 28, Part 1 Low-rise Buildings, Enclosed

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

Risk Category II, V_{ult}=115 MPH — Figure 26.5-1B

Exposure C — 26.7, 26.7.2

$P=Q_h[(GC_{pf})-(GC_{pi})]$ MWFRS — 28.3.1 Design wind pressure



K_d=0.85 — Wind directionality factor — 26.8

GC_{pi}=+/-0.18 — Internal pressure coeff. — 26.13

$K_z= 2.01\left(\frac{15}{z_g}\right)^{\left(\frac{2}{\alpha}\right)}$ — Velocity pressure exp. coeff. — 26.10-1

K_z=0.78 @ h=10'

$Q_h=0.00256K_zK_{zt}K_dV^2$=22.4 psf — 26.10-1

Figure 28.3-1

Surface	1	4	1E	4E
GCpi	0.4	-0.29	0.61	-0.43
P (psf)	8.96	6.5	13.66	9.63
	15.46 psf		23.3 psf	

Parapet

$P_p=Q_p(GC_{pn})$ — 28.3-2

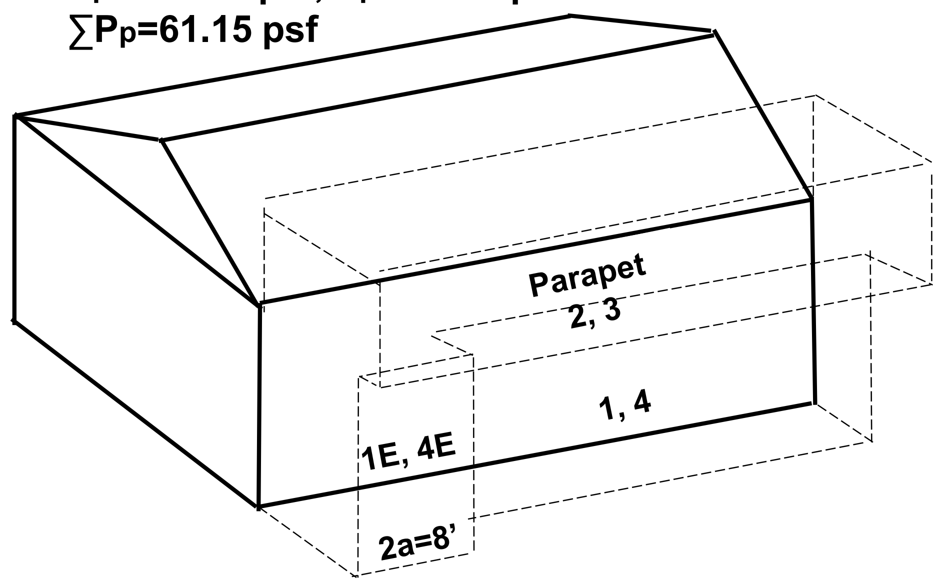
K_z=0.85 @ 12' Top of parapet

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	kd²	Fv		FT	Fv+FT	Loads δ_{SW}
2	43.54		3		130.63	391.89	4923.8		-40.0	4883.8	0.112
3	43.54		3		130.63	391.89	4923.8		40.0	4963.8	0.114
A		25.14		20	502.74	10054.72756			153.8	153.8	0.0061
B		25.14		20	502.74	10054.72756			-153.8	-153.8	-0.006
Σ	87.09	50.27			J=	20893.23102	9847.6				

Rho= 1   Ax= 1   Fy= 9847.6   e= 34   T= 6392.0

2a= 8   Net= 23.5   W1,4= 127.1   W1E,4E= 150.6

Transverse Loading

Grid Line	kx	Ky	dx	dy	kd	kd²	Fv		FT	Fv+FT	Loads δ_{SW}
2	43.54		3		130.63	391.89			18.8	18.8	0.000
3	43.54		3		130.63	391.89			-18.8	-18.8	0.000
A		25.14		20	502.74	10054.72756	4923.8		72.4	4996.2	0.199
B		25.14		20	502.74	10054.72756	4923.8		-72.4	4851.4	0.193
Σ	87.09	50.27			J=	20893.23102	9847.6				

Fx= 9847.6   emin= 16   T= 3008.0

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

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

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

Expected Dead + Seismic D+QE (other terms if "expected" gravity loads as per AS ρ=1.0, Ax=1.0

Grid Line	SW	Ga	Rho	V on wall	v	T	C	Δ_a	$F_{c\perp}$	Crush.	Shrink	δ_B	δ_S	δ_{Rot}	δ_{SW}
Calculate Stiffness of Walls on A & B using Transverse 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
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
Calculate Stiffness of Walls on 2 & 3 using Longitudinal 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
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
		V equal to revised wall force based on HD STR (design) capacity							625 Max.	Add stud					

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

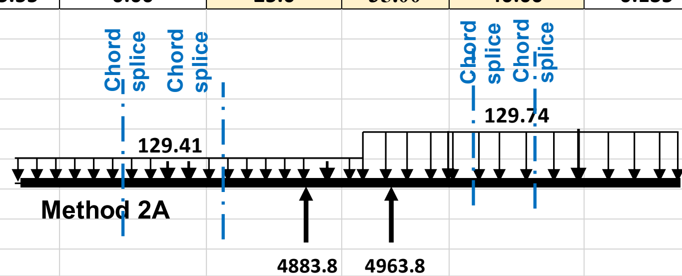
Diaphragm Deflection (STR)

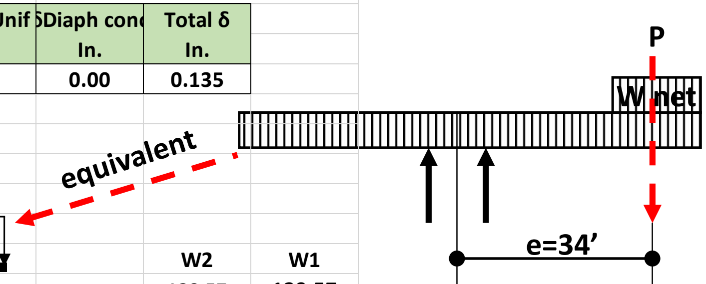
$$\delta_{Diaph\ Unif} = \frac{3vL'^3}{EAW'} + \frac{0.5v_{max}L'}{1000G_a} + \frac{\Sigma\Delta_c X_c}{W'}$$

Rt. Cantilever

Splice Forces (Lbs.)			Σδ_slip	v unif.	v conc.	Ga	L'	W'	δDiaph Unif	δDiaph conc	Total δ
F 15	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
Iincludes effects of sw's along chord line





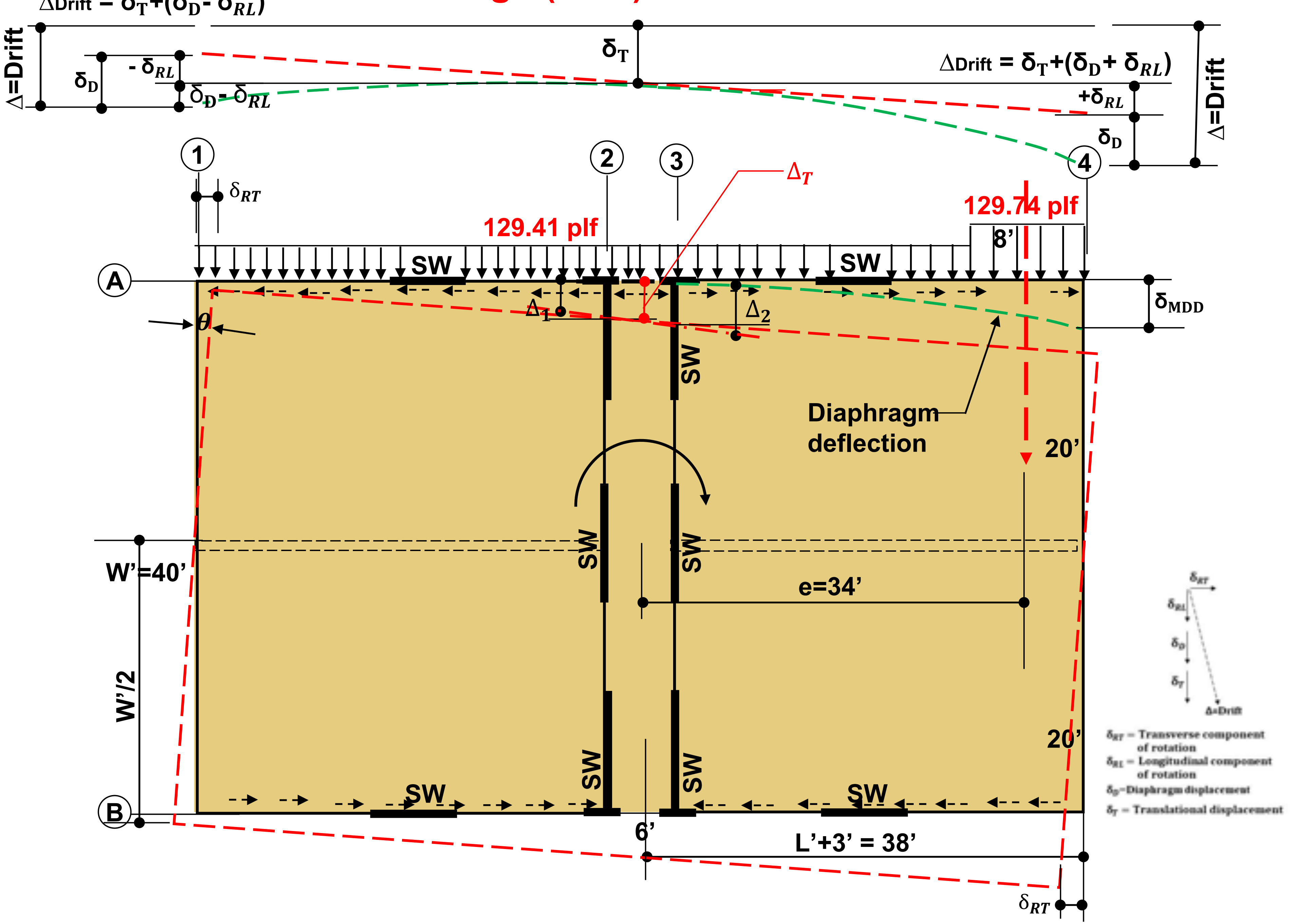
W2	W1
129.57	129.57
0.17	-0.17
129.74	129.41

Diaphragm Deflection (STR)

Lft. Cantilever

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									

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



Drift-Method 2A (Wind) ρ=1.0, Ax=1.25

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

δ_2 = 4.884 k / 43.54 k/in = 0.112 in,

δ_3 = 4.964 k / 43.54 k/in = 0.114 in

δ_A = 0.154 k / 25.14 k/in = 0.0061 in,

δ_B = -0.154 k / 25.14 k/in = -0.0061 in

Δ_{Diaph}= 0.135”

$\Delta_{Average}$= 0.113” (Translation)

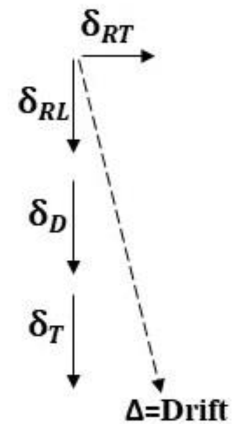
$$\delta_{RL} = \frac{2\Delta_{SW\ A,B}(L'+3')}{W'} = \frac{2(0.0061)(35'+3')}{40} = 0.0116'', \ \delta_{RT} = 0.0061$$

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

$$\text{Drift } \Delta_4 = \sqrt{(0.113 + 0.135 + 0.0116)^2 + (0.0061)^2} = 0.26''$$

$$\text{Drift } \Delta_1 = \sqrt{(0.113 + 0.135 - 0.0116)^2 + (0.0061)^2} = 0.24''$$

Drift Δ_4=0.26”



δ_{RT} = Transverse component of rotation

δ_{RL} = Longitudinal component of rotation

δ_D=Diaphragm displacement

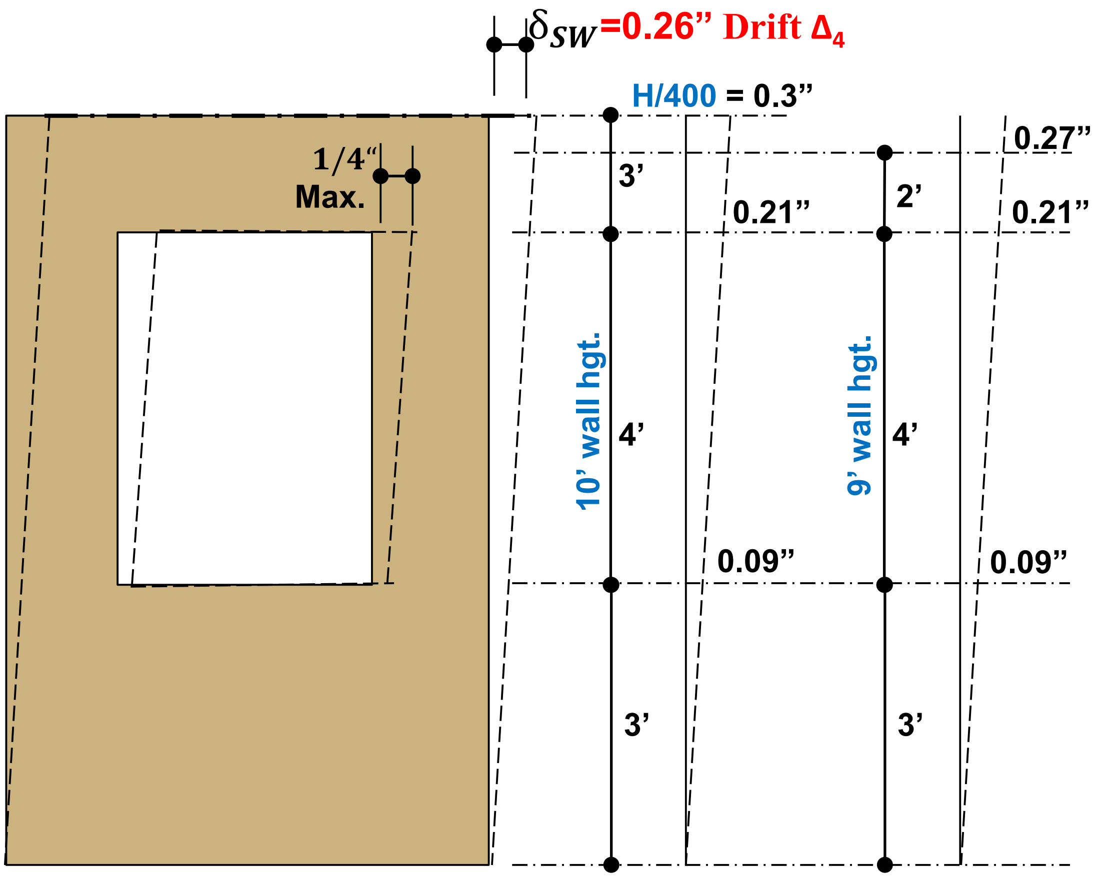
δ_T = Translational displacement

Flexibility check:

$\delta_{MDD} < 2\ \delta_{ADVE}$, 0.135”<2(0.113”)= 0.226”

Diaphragm is rigid or semi-rigid

Allowable Drift Wind H/600, H/400



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
Drift Δ_4 =0.26"<0.3"
∴ drift OK

Maximum displacement at top of window at allow defl.=0.21"<0.25"
∴ OK

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:

- ASCE 7 drift limits requirements for wind design can be found in ASCE 7-16 Commentary Appendix CC, Serviceability Considerations
  Section CC2.2 Drift of Walls and Frames

Damage to nonstructural partitions, cladding, and glazing may occur if the story drift exceeds about 3⁄8" unless special detailing practices are made to tolerate movement

Questions?

This concludes Woodworks Presentation on:

Part 3- Cantilever Diaphragm Design, And Flexibility and Drift Checks

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

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Thank You

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