WoodWorks

A Design Example of a Wood Cantilever Diaphragm

Carbon 12, Portland, OR
PATH Architecture

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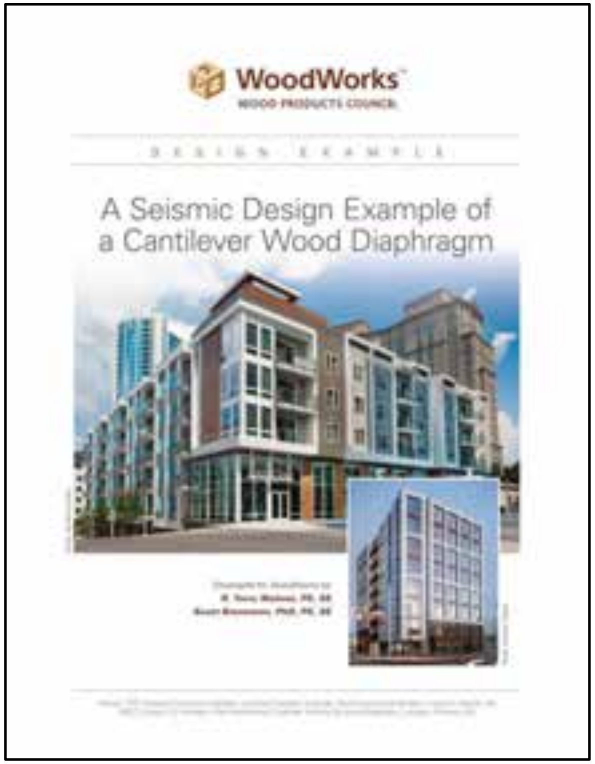
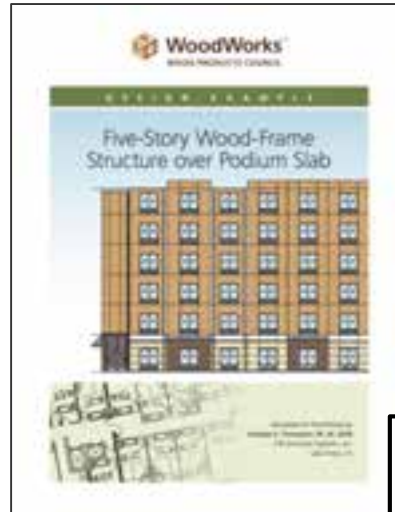
In Your Folders





- **Colored flow chart**
- **The Analysis of Irregular Shaped Diaphragms**

Paper: **Education tab/Presentation Slide Archive/**

https://www.woodworks.org/wp-content/uploads/design_examle-Design-Example-of-a-Cantilever-Wood-Diaphragm.pdf

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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
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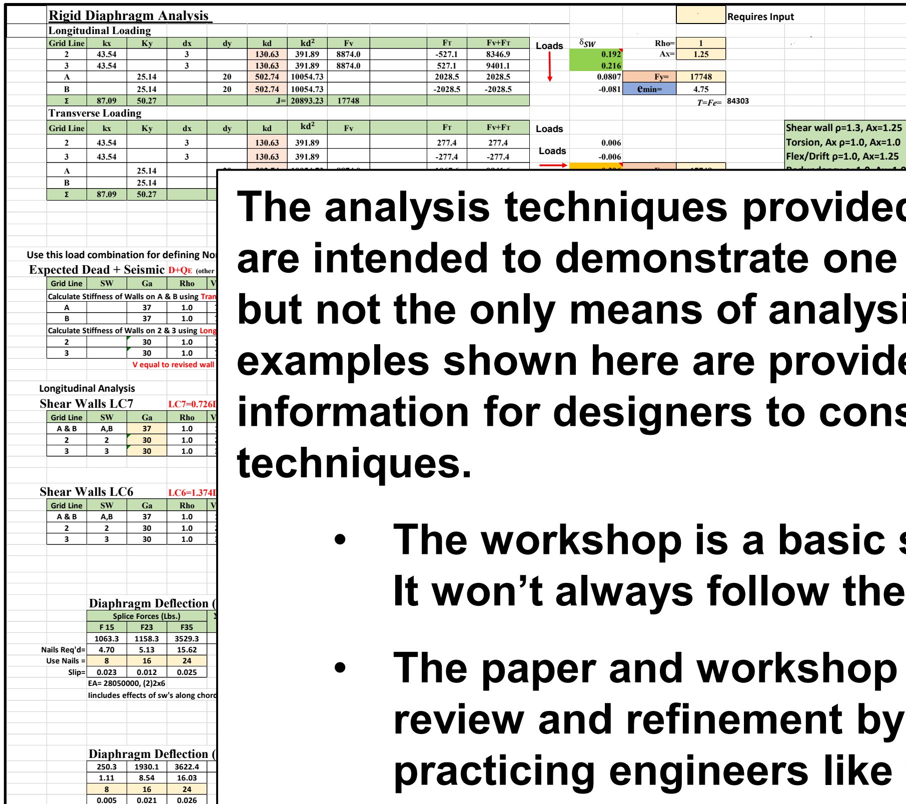
Codes and Standards

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.



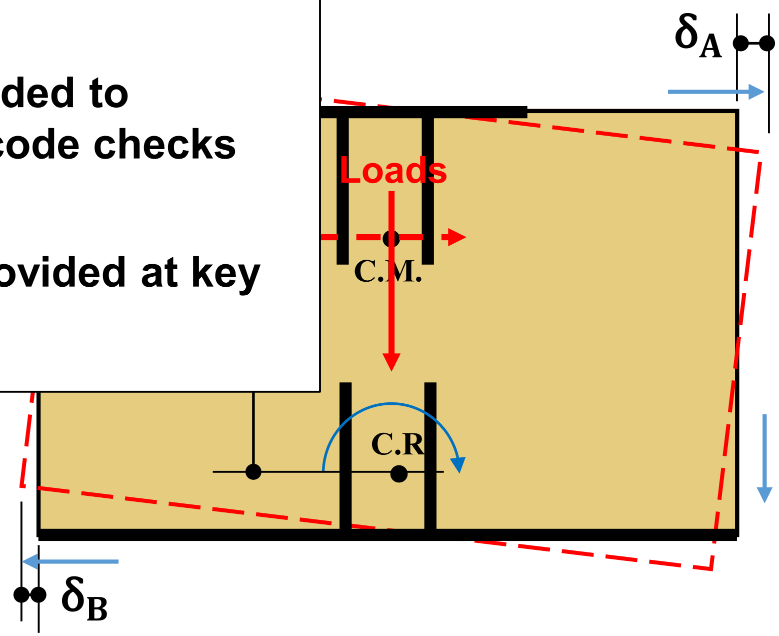
The analysis techniques provided in this presentation are intended to demonstrate one method of analysis, but not the only means of analysis. The techniques and examples shown here are provided as guidance and information for designers to consider to refine their own techniques.

- 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.

$$= \sum kd_x^2 + kd_y^2$$

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

$$F_T = T\frac{\ldots}{\sum kd_x^2 + kd_y^2}$$



Workshop Content

Part 1-Background:

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

15 minute break

Part 2-Design Example :

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

15 minute break

Part 3-Design Example (cont.):

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

Lunch

Part 4-Design Example (cont.):

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

Part 1 Content

Part 1-Background:

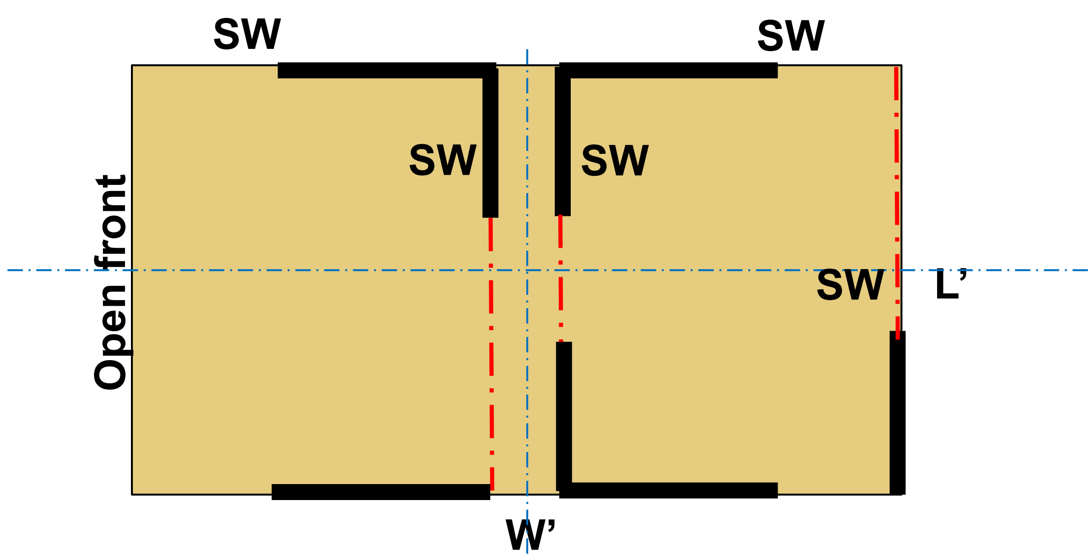
- Introduction
- Questions needing resolution
- Horizontal distribution of shear and stiffness issues
- 2015 SDPWS open-front requirements-review
- Introduction to open-front example

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?

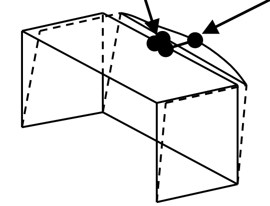
Horizontal Distribution of shear and Stiffness Issues

- 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:



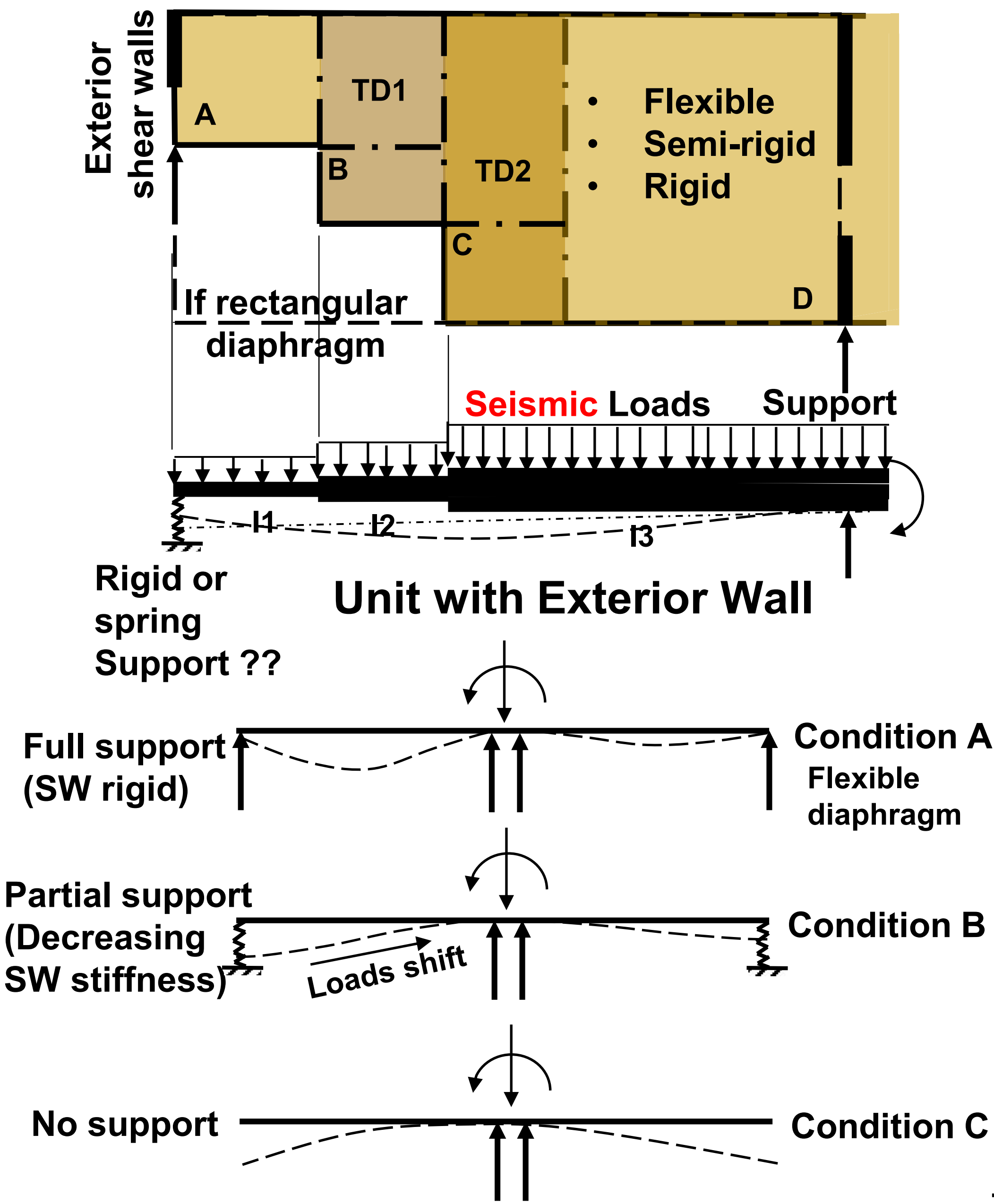
Maximum diaphragm deflection (MDD) >2x average story drift of vertical elements, using the ELF Procedure of Section 12.8?

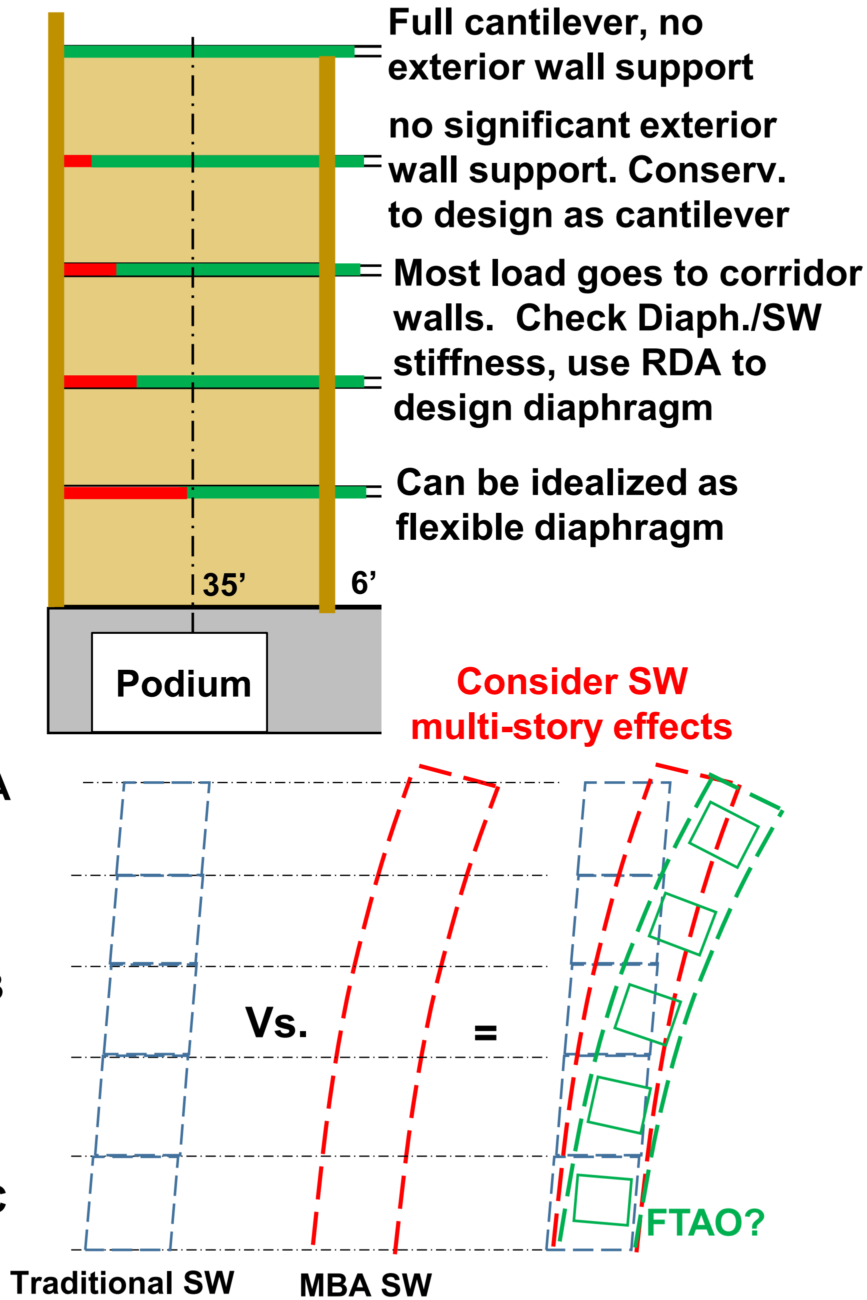
Calculated as Flexible

- **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.

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

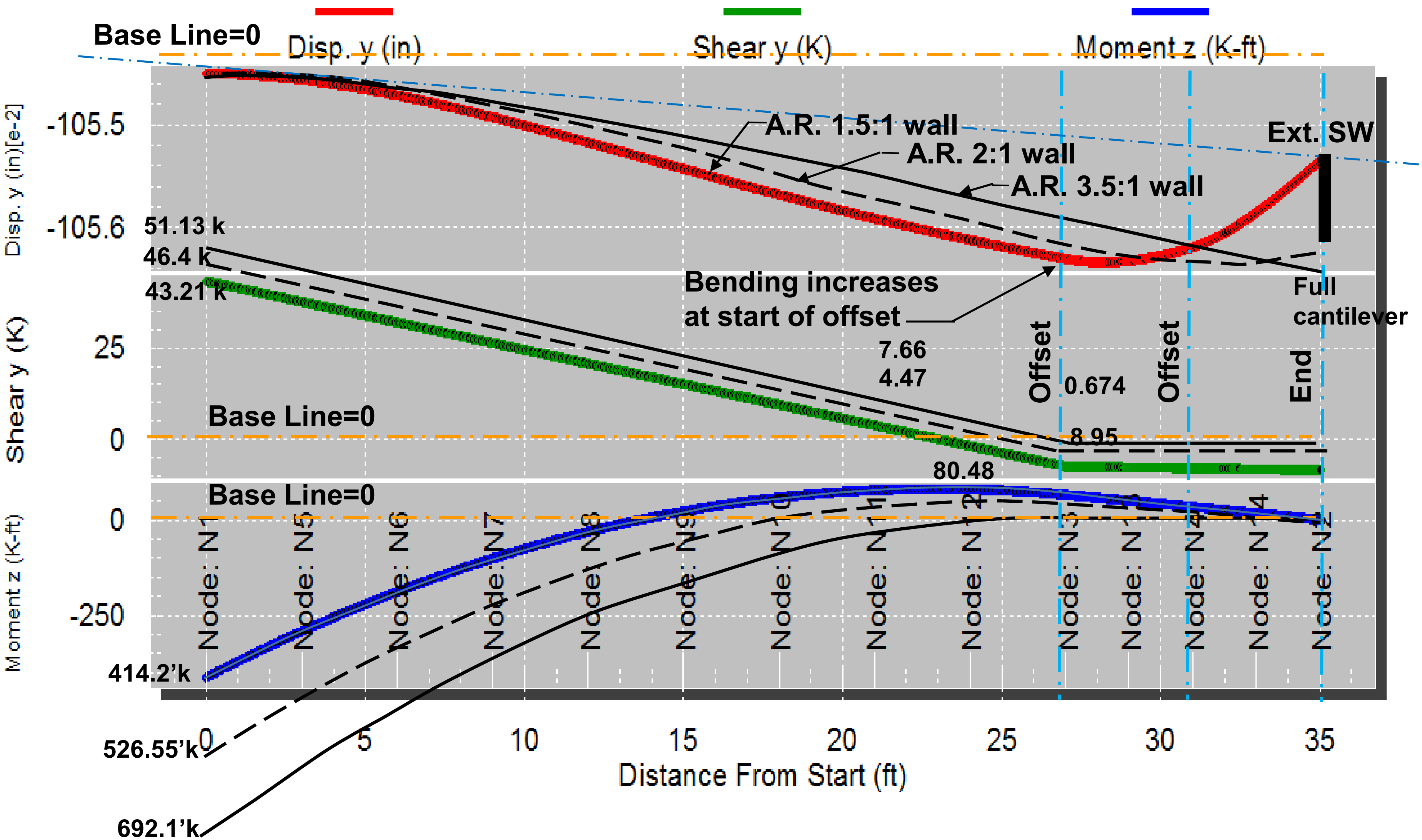
Force Distribution Due to Diaphragm/SW stiffness





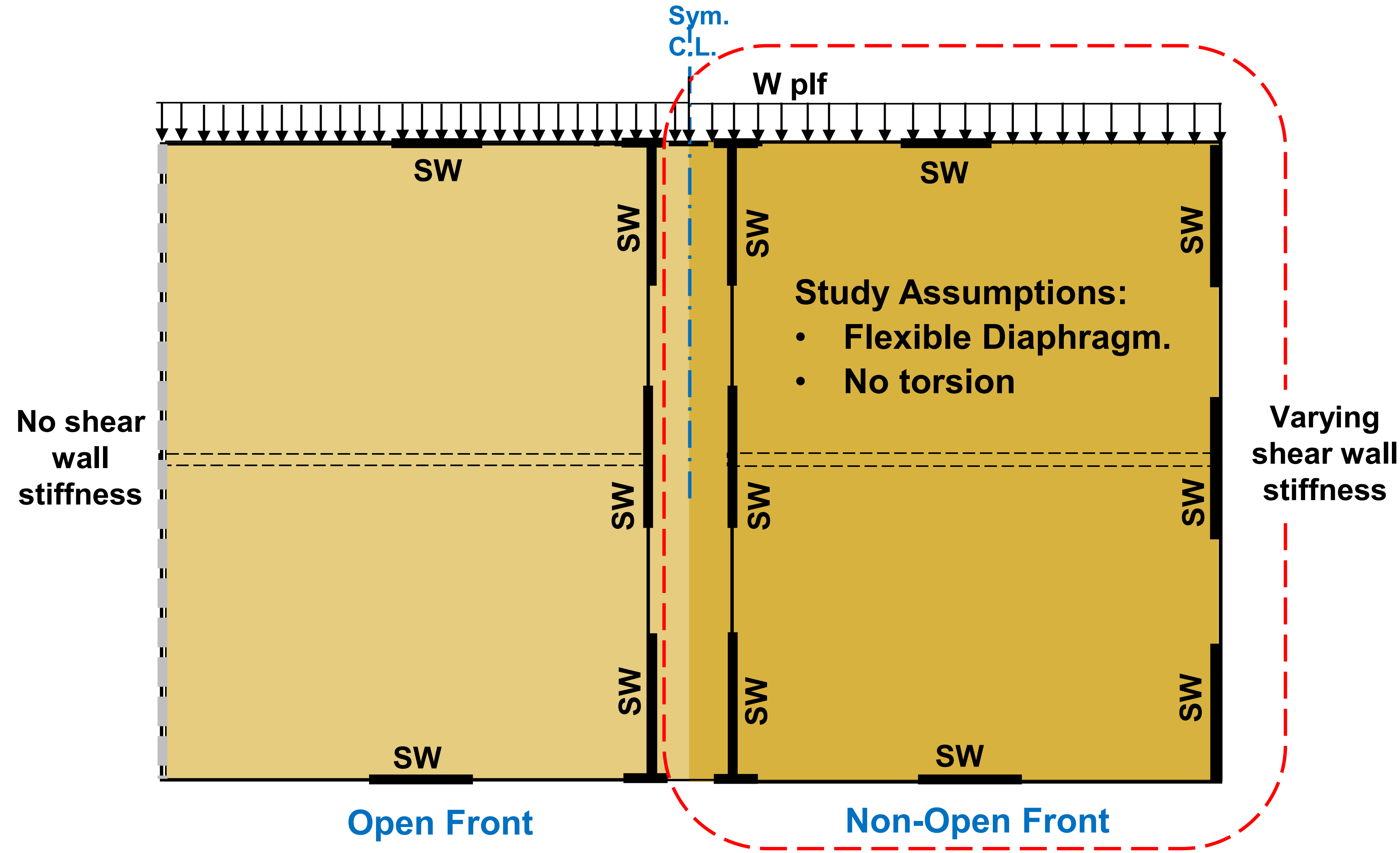
Review Stiffness at Offsets

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

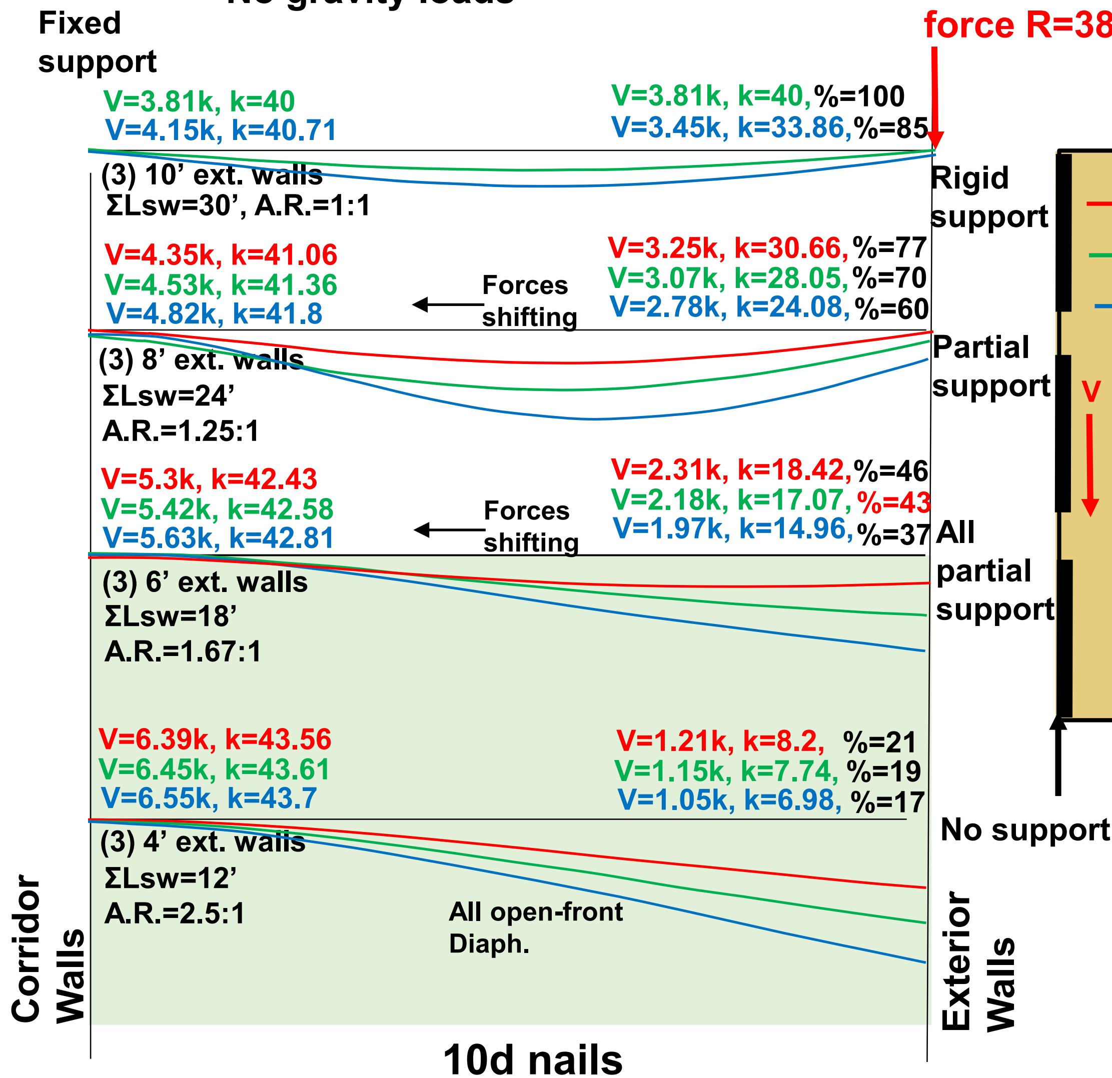
35’ RDA Force Distribution-SW displ.

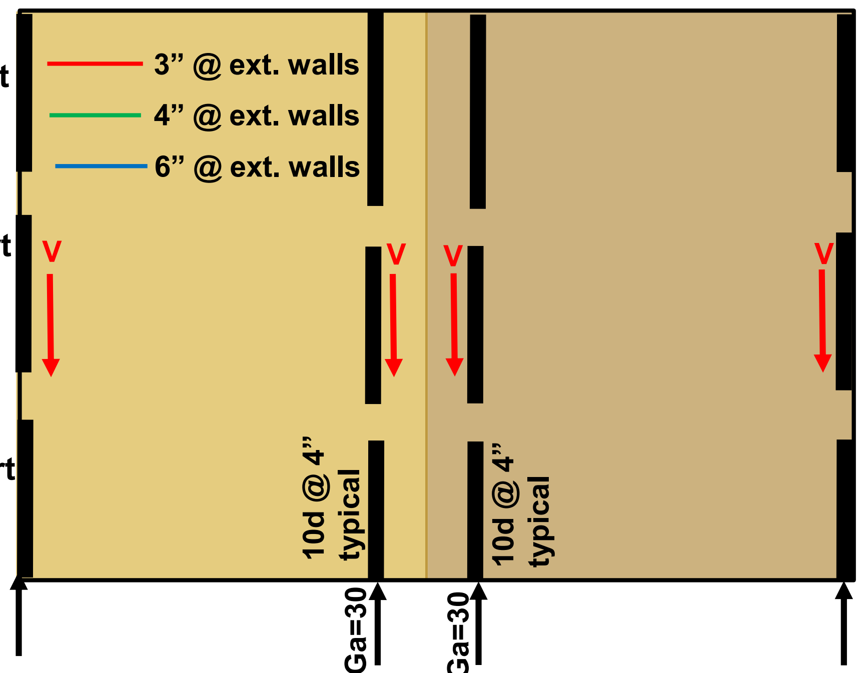
- Diaphragm stiffness flexible
- Shear wall stiffness-variable
- Seismic STR. Forces
- No torsion
- No gravity loads

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

Open-front effect

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



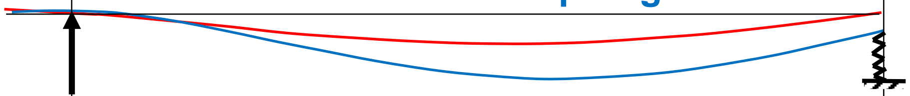


Prelim conclusion (This example only):

- If walls near 44% or if k ≤ 20 consider open-font
- Magic 20’ SW

Condition A

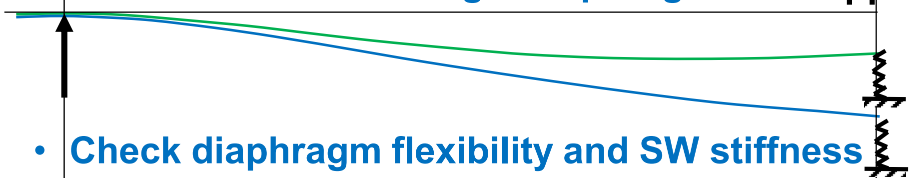
Flexible diaphragm



- Check diaphragm shear and chord forces.
- Check story drift limits at shear wall line.

Condition B

Semi-rigid diaphragm



- Check diaphragm flexibility and SW stiffness
- RDA check of forces to walls
- Check diaphragm shear and chord forces.
- Check story drift limits at shear wall lines.
- Check torsional irregularities
- Check Redundancy

Condition C

Open-front diaphragm

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)

A matter of Stiffness

Seismic:

ASCE 7-16 Section 12.3.1- Diaphragm flexibility-The structural analysis shall <u>consider</u> 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 <u>consider</u> 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

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

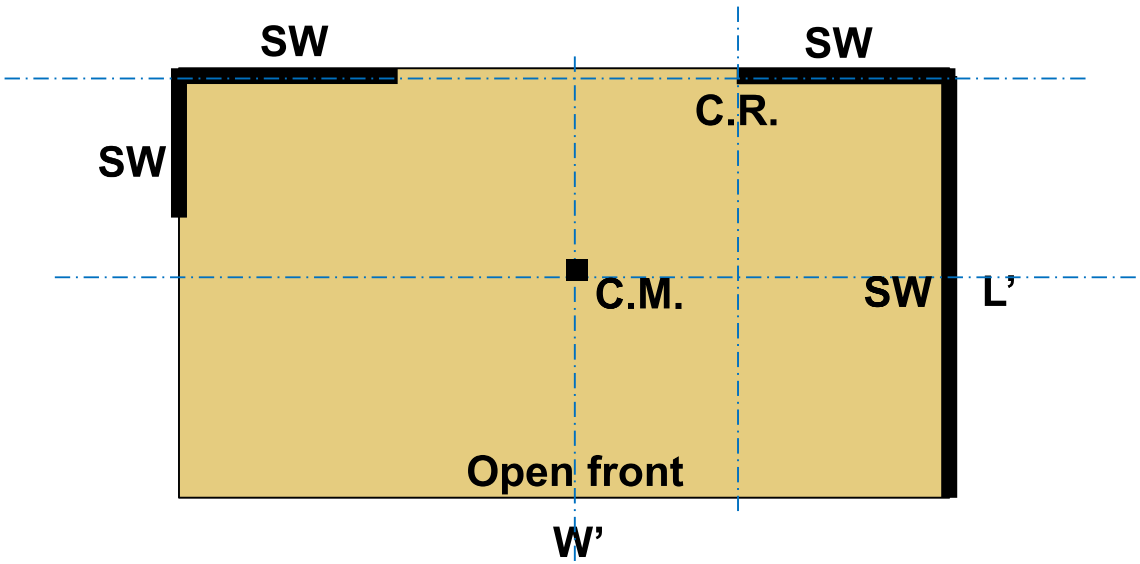
An Engineered Structure?

No shear walls

Possible Soft Story

2015 SDPWS Open-front Diaphragm Requirements

Open-Front Diaphragms



Relevant 2015 SDPWS Sections

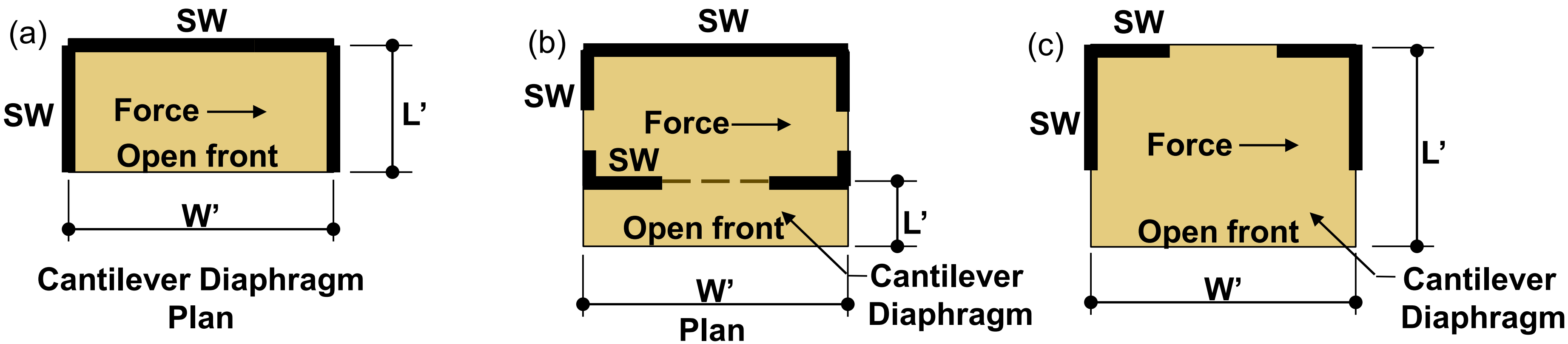
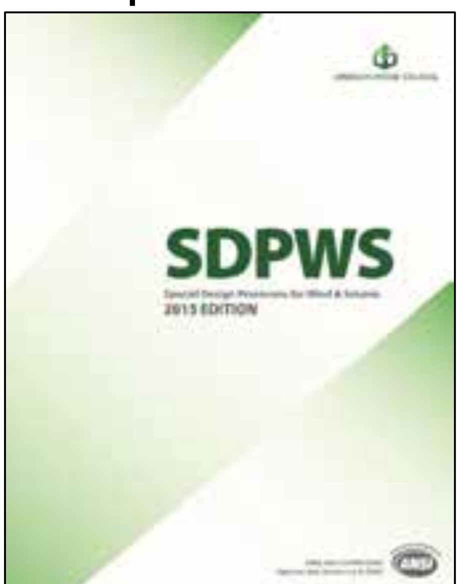


Figure 4A Examples of Open Front Structures

4.2.5.2 Open Front Structures:

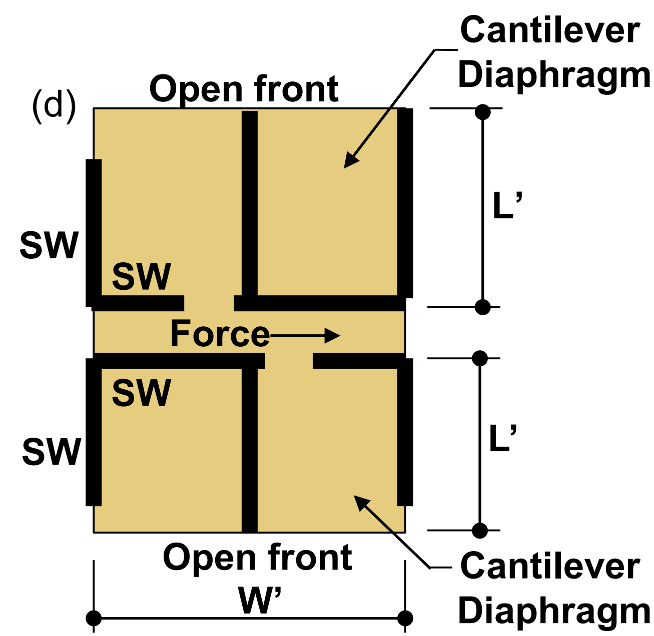


New definitions added:

- 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 <u>seismic</u> loads, wood-frame diaphragms in open front structures shall comply with all of the following requirements:

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

2. **The drift at <u>edges</u> 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 <u>over one</u> story in height, and 1:1 for structures <u>one</u> story in height.**

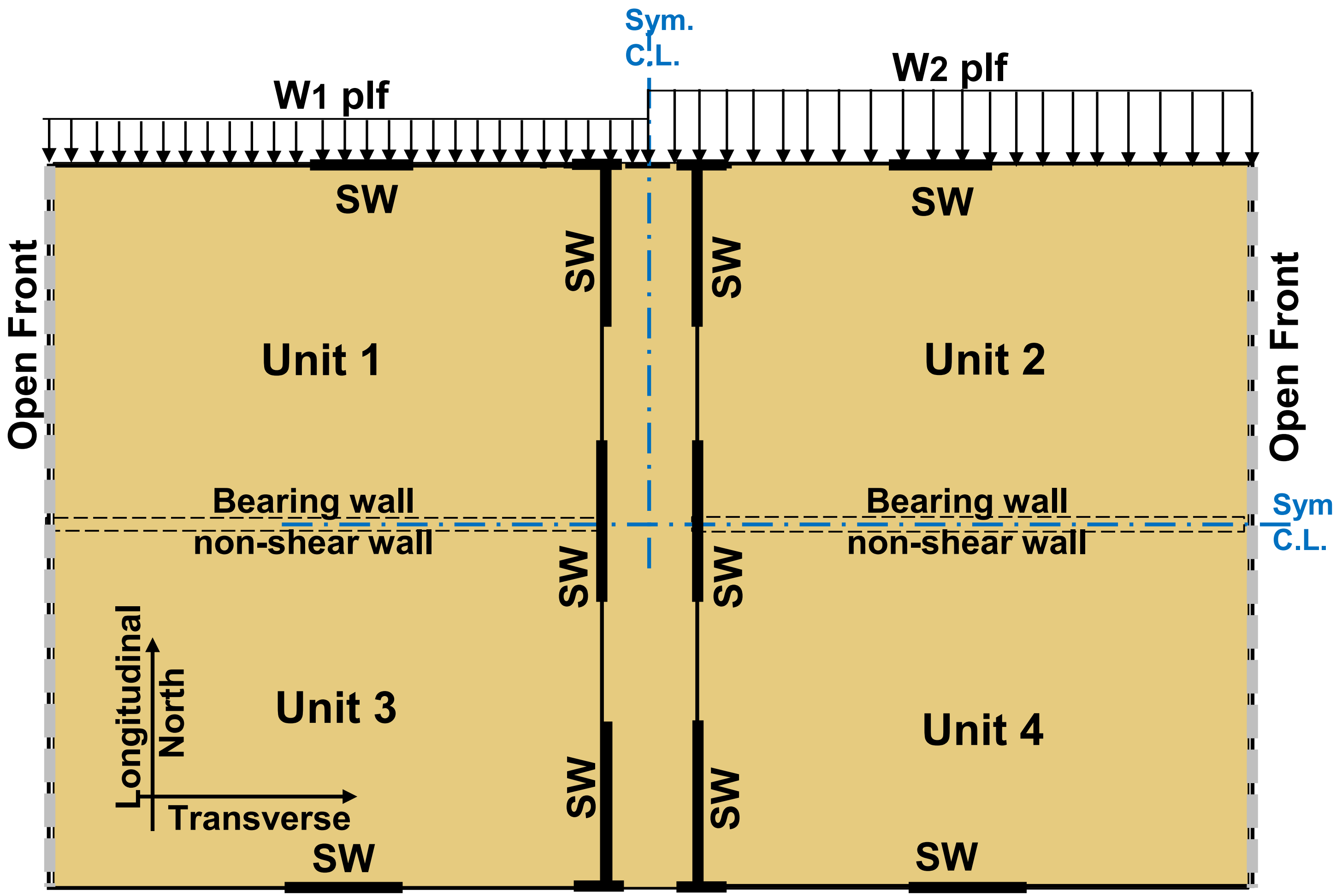
4. **For loading parallel to open side:**
   a. **Model as semi-rigid (min.), shall include shear and bending deformation of the diaphragm, or idealized as rigid.**

5. **The diaphragm length, L', (normal to the open side) does not exceed 35 feet. (2008 SDPWS: L'_{max}=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 <u>not</u> intended to serve as recommendations or as the only method of analysis available.

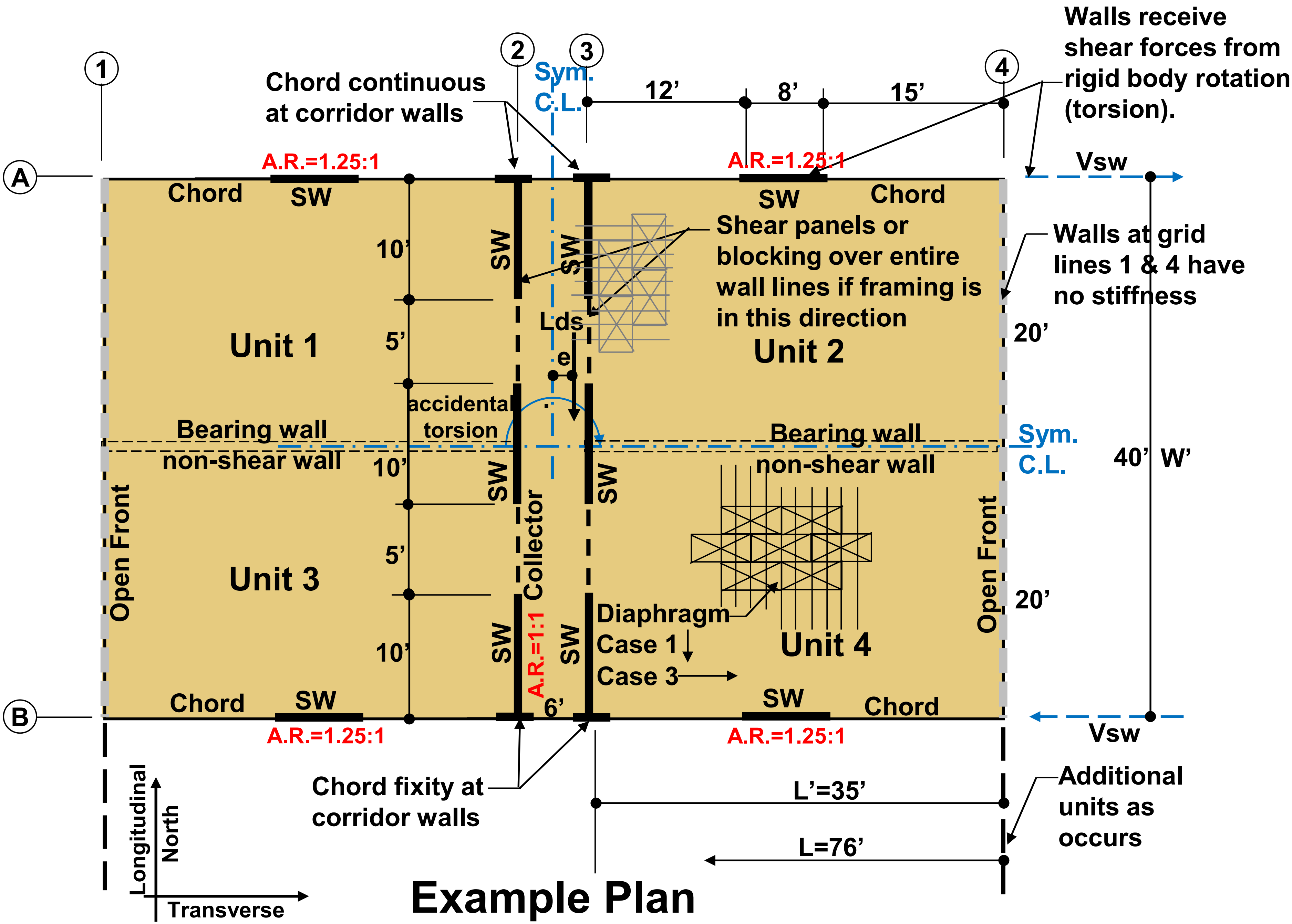
Open Front Structures Code Checks:

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

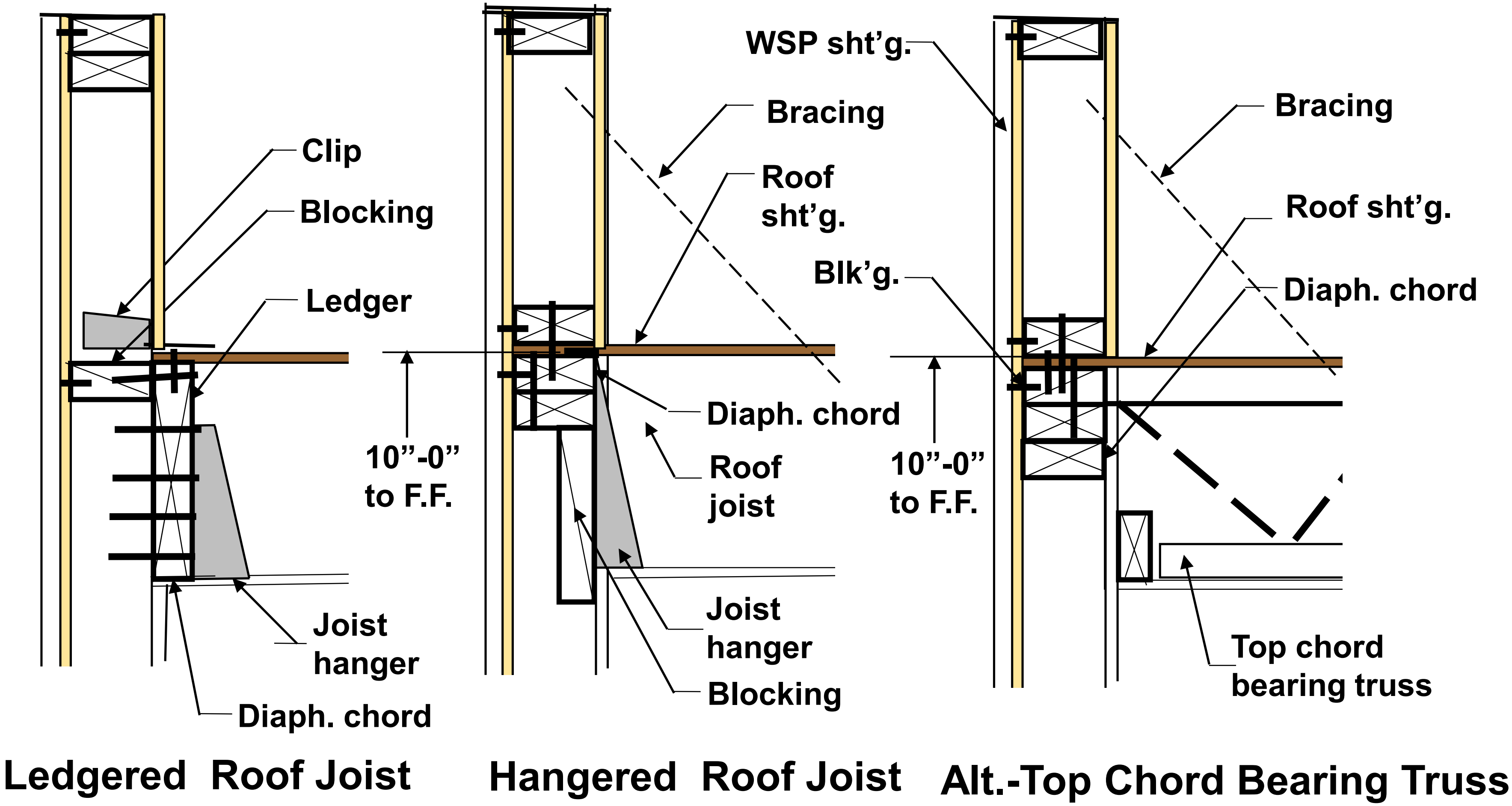
1. **Check stiffness of diaphragm and shear walls** — **ASCE 7 12.3.1, SDPWS 4.2.5.2 (3)**
2. **Verify aspect ratio** — **SDPWS 4.2.7.1- 4.2.7.3**
3. **Check drift at edges** — **ASCE 7 12.12.1, SDPWS 4.2.5.1**
4. **Check for torsional irregularity** — **ASCE 7 12.3.2, SDPWS 4.2.5.1**
   - **Inherent 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. **Show that the resulting drift at the edges of the structure can be tolerated.**
3. **Recommend Following SDPWS 4.2.5.2 (not required by code). Considered good engineering practice.**

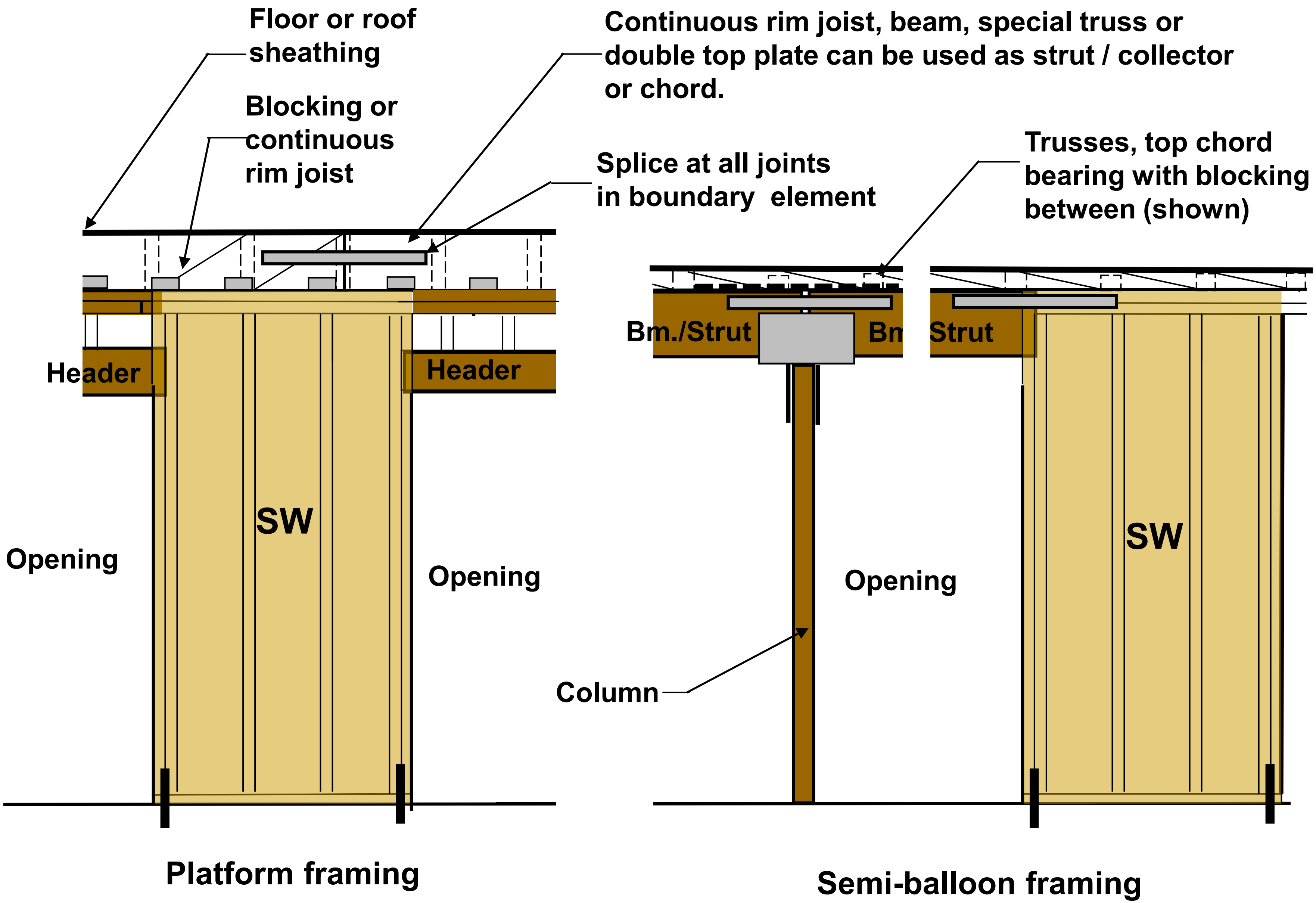


Example Plan

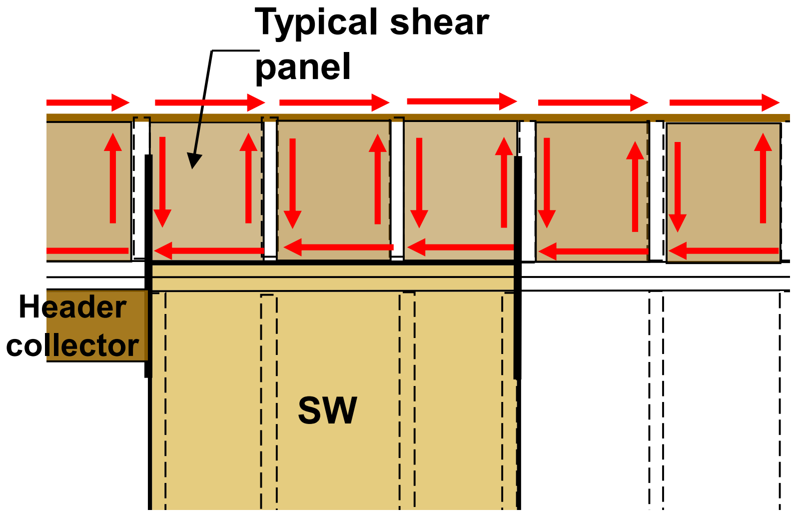


(Platform framing not shown)

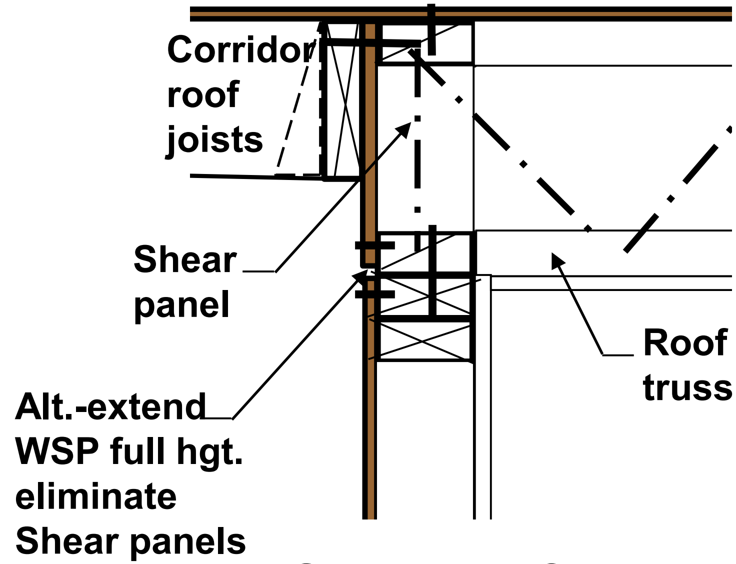
Typical Exterior Wall Sections



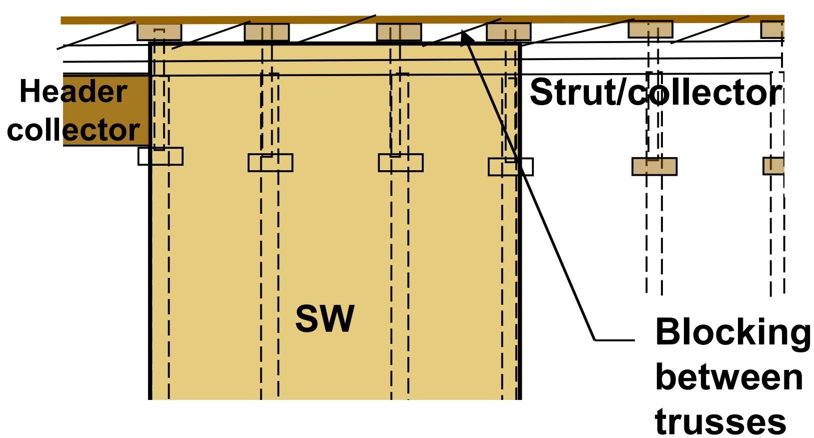
Typical Exterior Wall Elevations at Grid Lines A and B



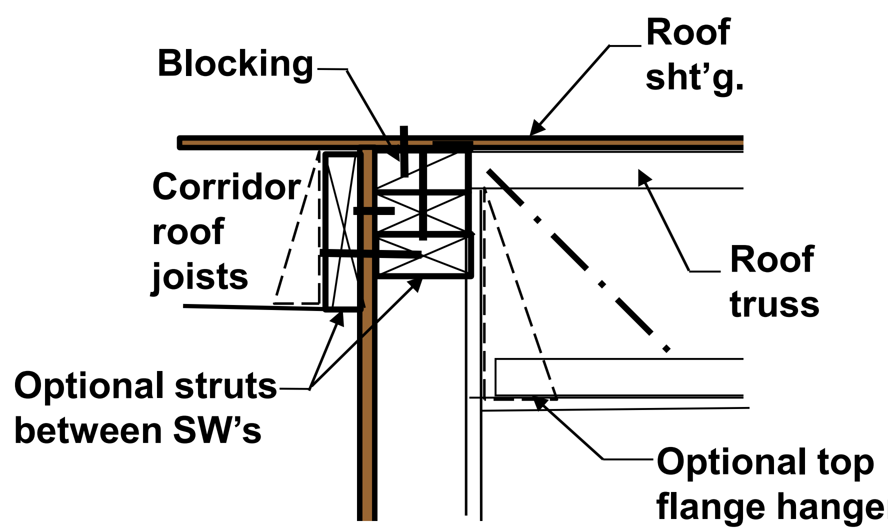
Platform Framing at Corridor



Section at Corridor



Semi-balloon Framing at Corridor



Section at Corridor
(Similar to example)

Typical Wall Sections at Corridor Walls

Let's Take a 15 Minute Break

Part 2 Content

Part 2-Design Example :

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

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, 12.3.1.1 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 ρ=1.3 unless conditions of ASCE 7-16 Section 12.3.4.2 are met to justify ρ=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 (Mt) plus the accidental torsional moments (Mta)**
- **Accidental torsional moment (Mta) = 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 (Mta) 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.**

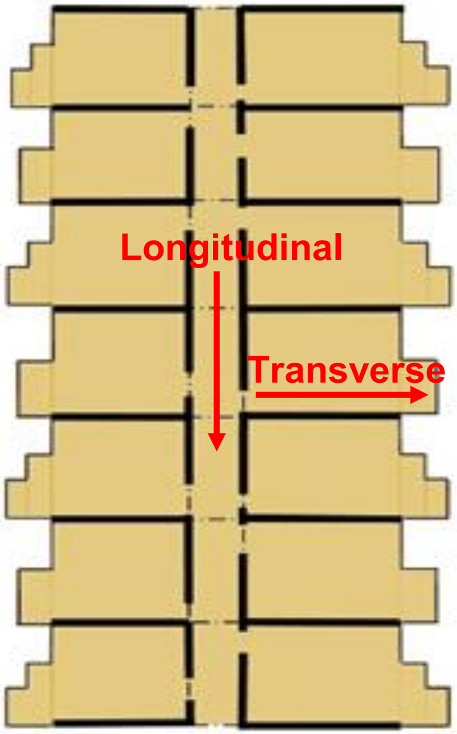
Exceptions:

- **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.**

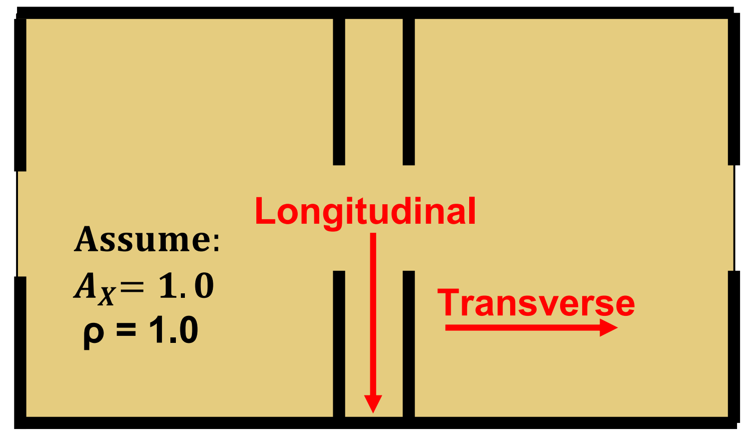
Structures assigned to SDC C, D, E, or F, where Type 1a or 1b torsional irregularity shall have the effects accounted for by multiplying Mta at each level by a torsional amplification factor (Ax)

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

Preliminary Assumptions-Redundancy / Irregularity Issues



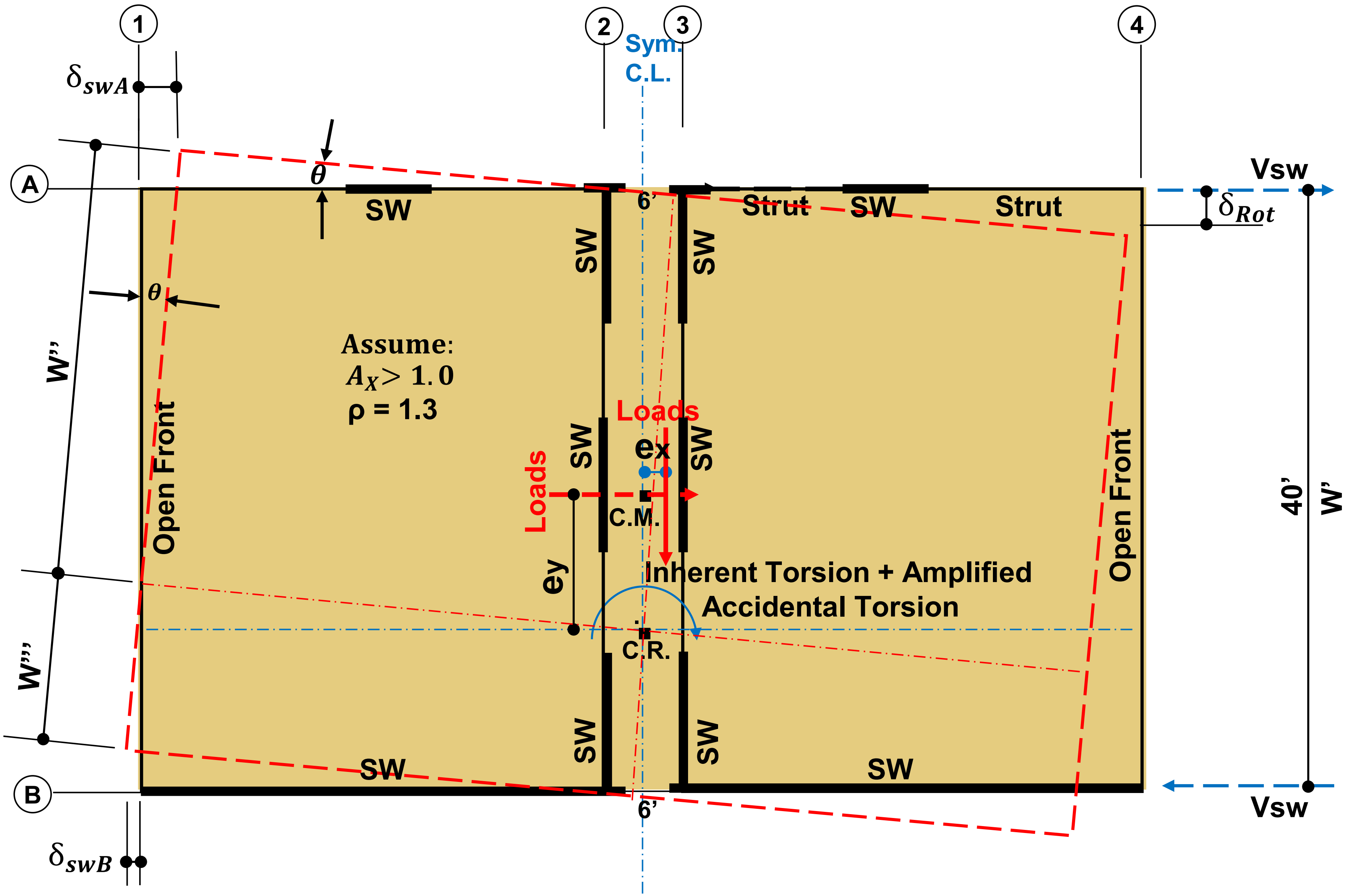
Assume:
$A_X = 1.0$
ρ = 1.0



A_X = Amplification of accidental torsion if torsional irregularity exists
ρ = Redundancy

Regular Plans

Questionable Plans-Unsymmetrical Plan Layouts



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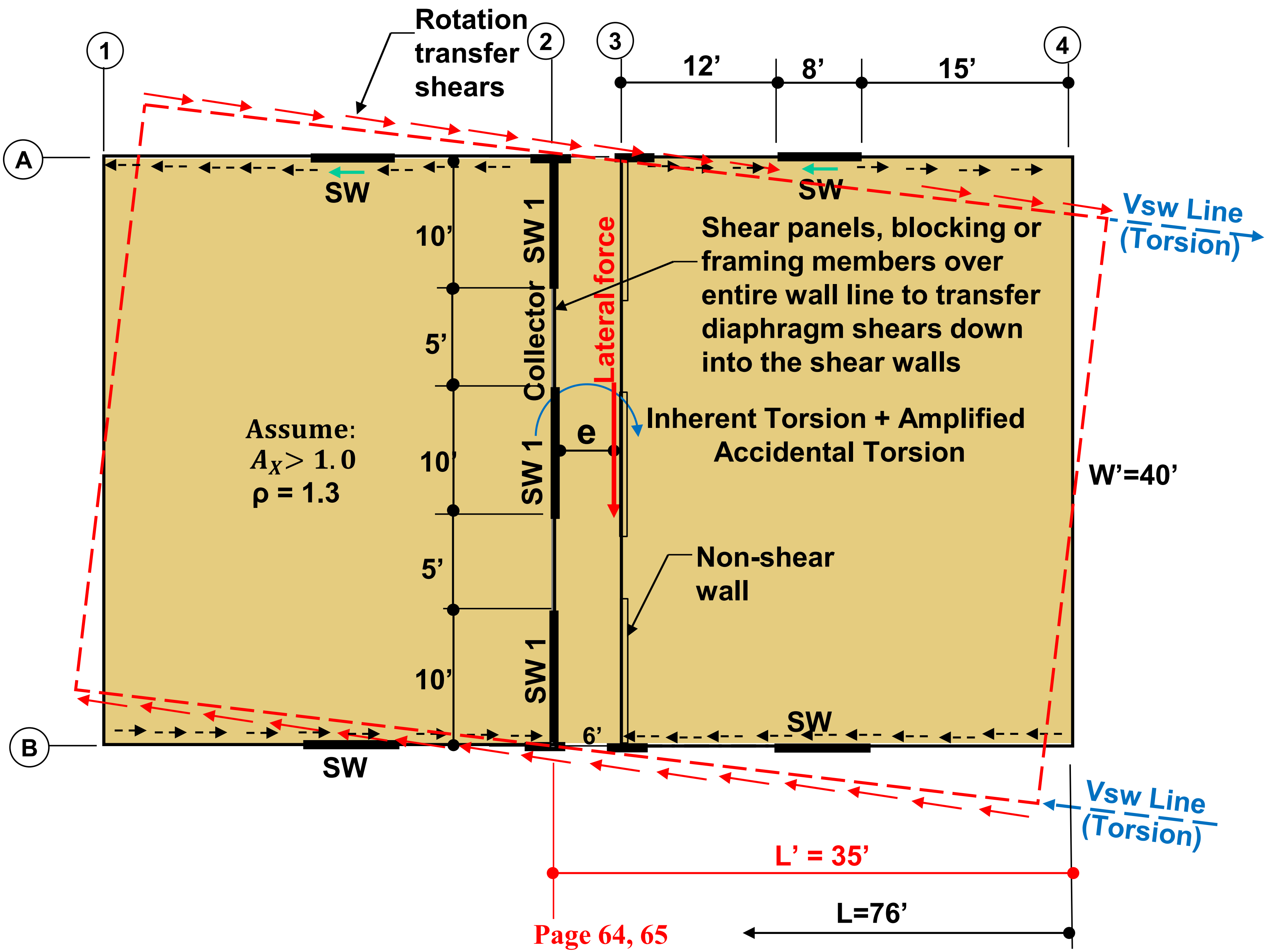
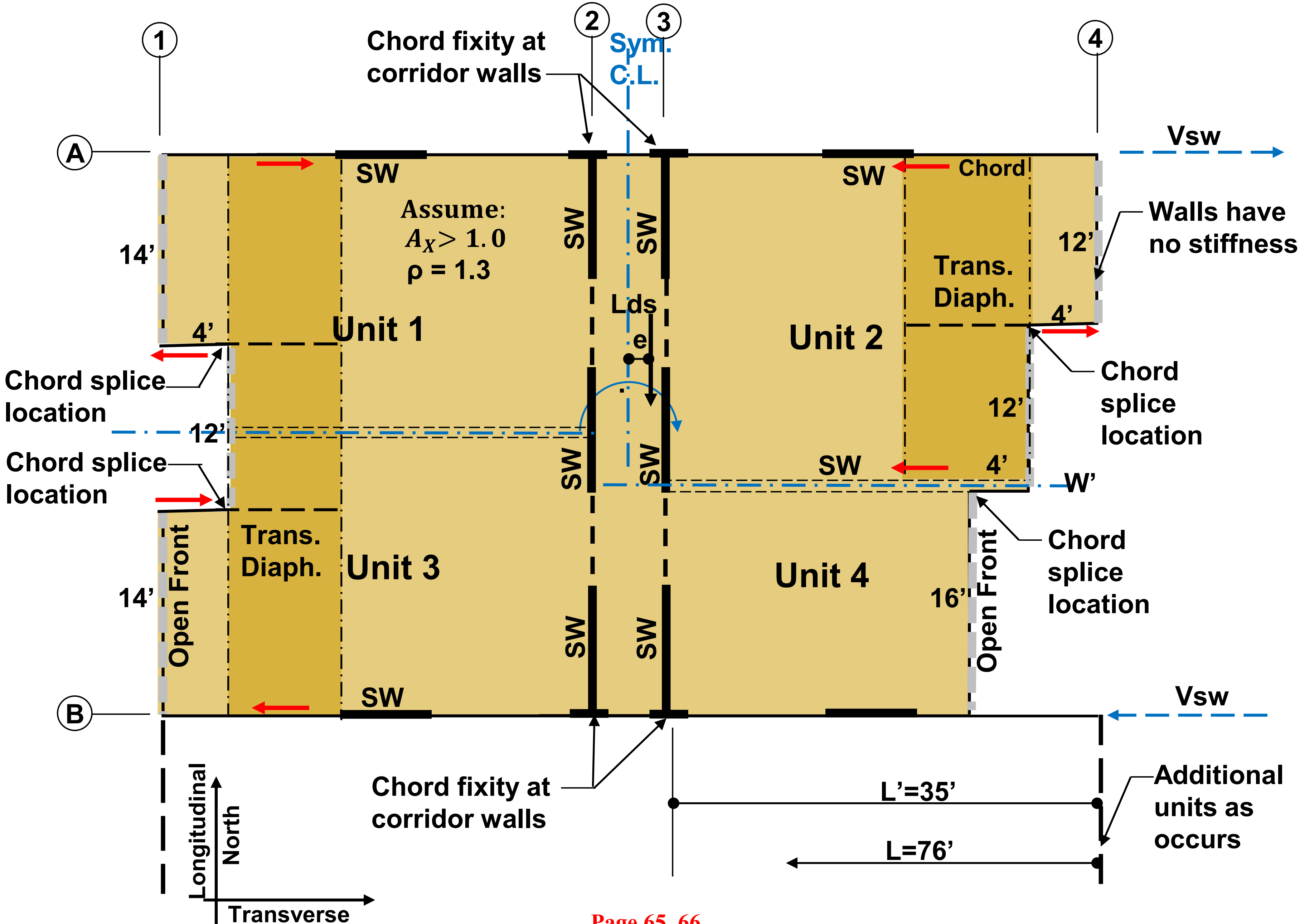
Questionable Plans-Corridor Walls One Side Only



Questionable Plans-Complex Plans-horizontal offsets



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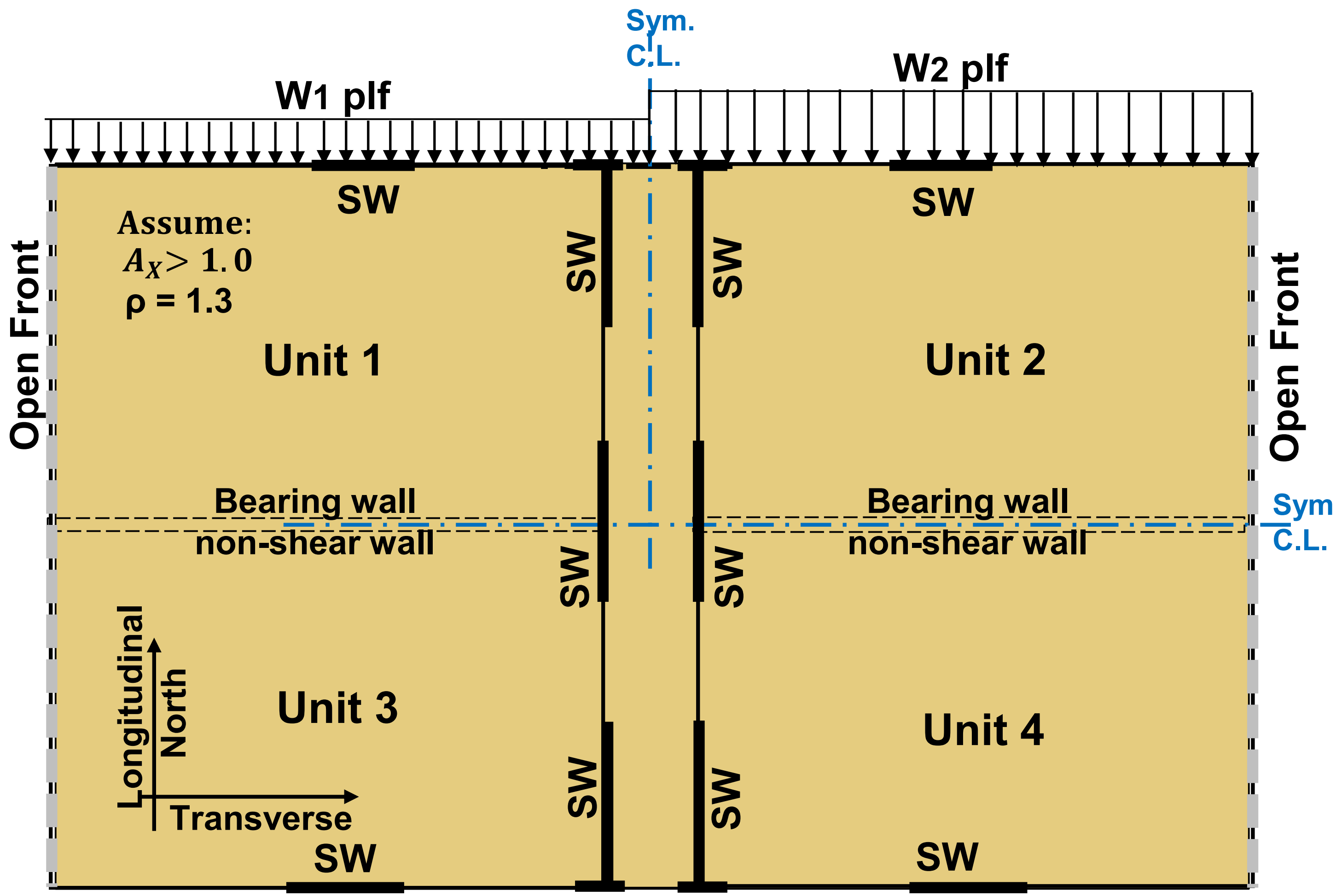
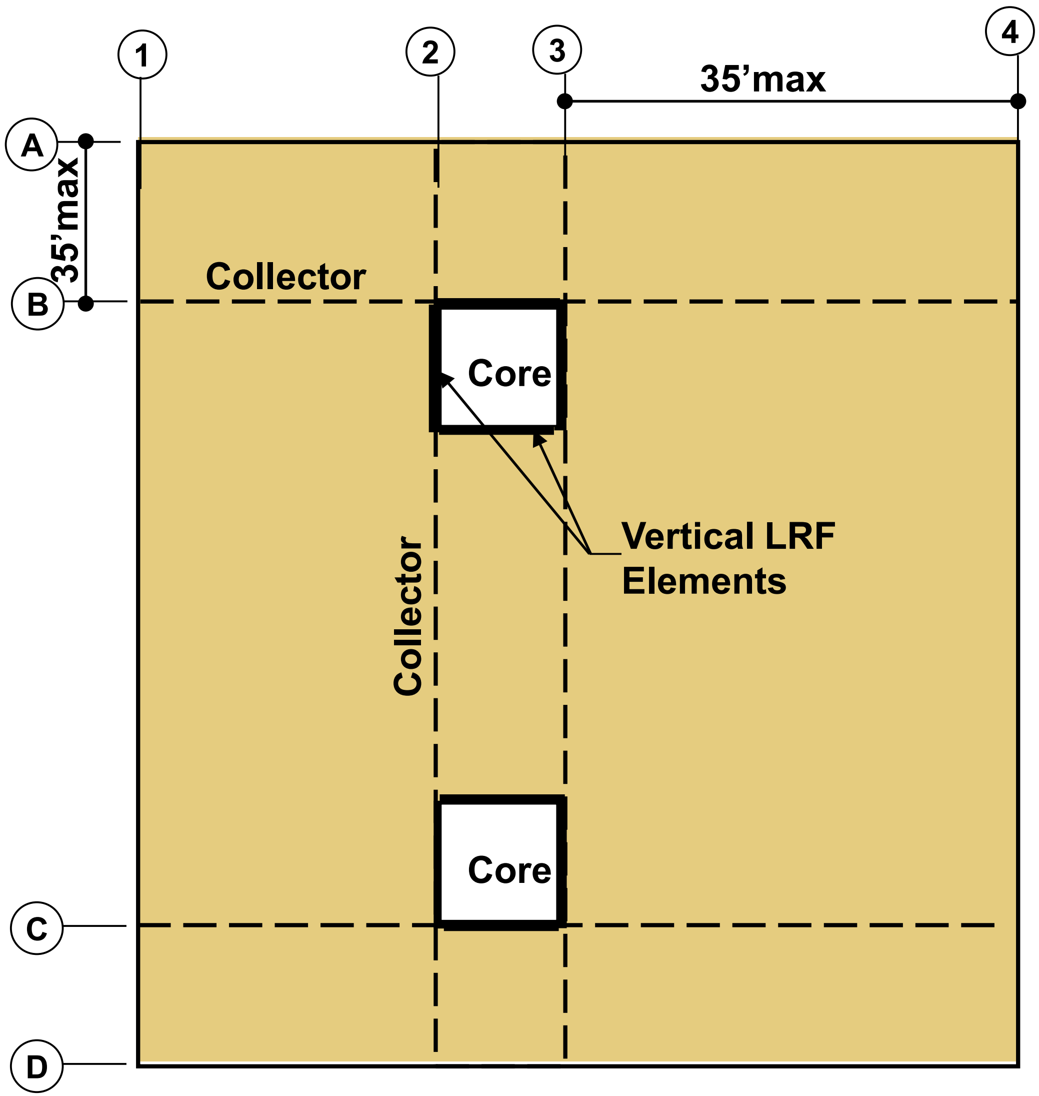
Questionable Plans-Design Example



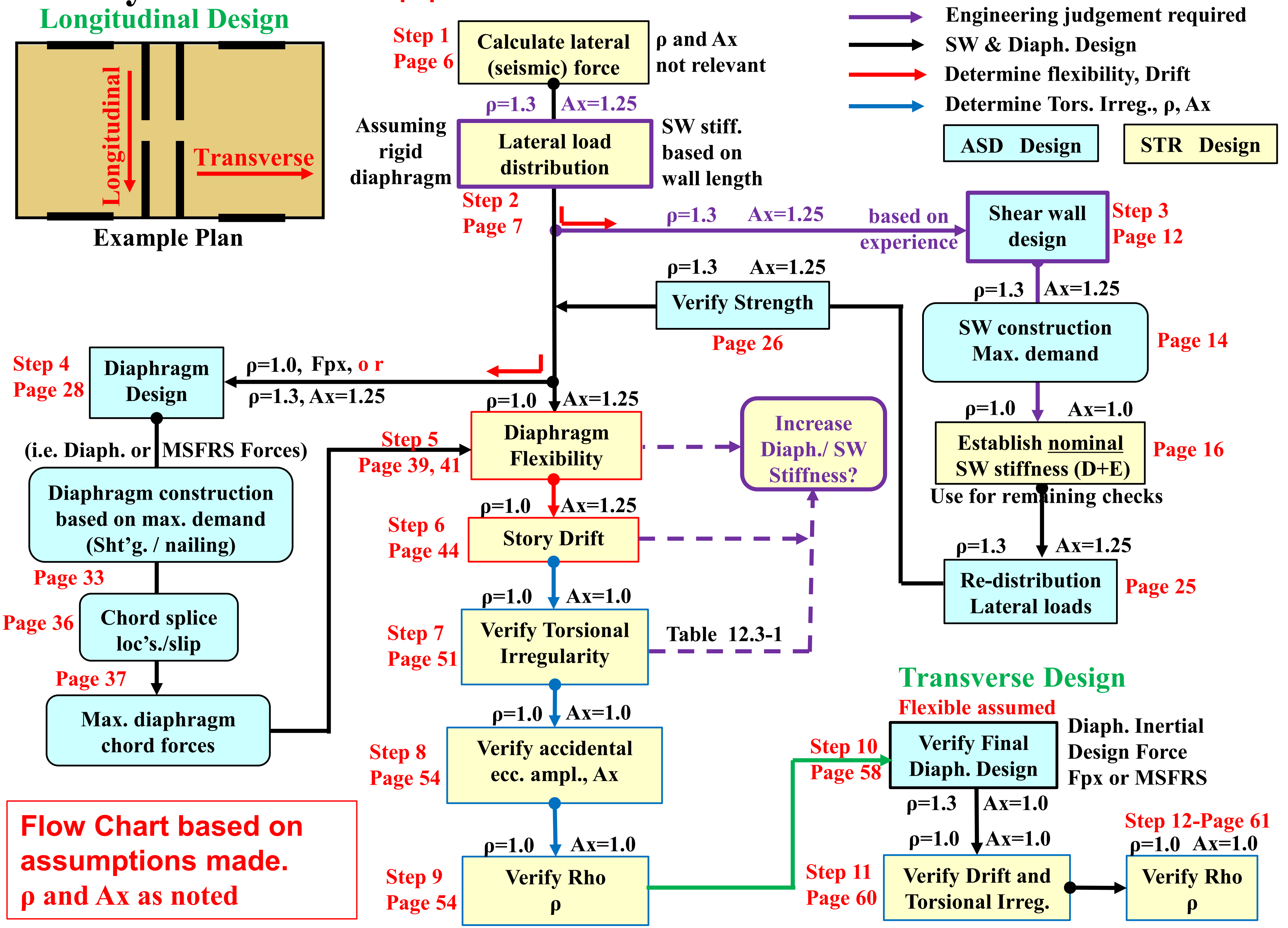
Questionable Plans-Core Structures

- Can be simple-symmetrical
- Can be complex-different eccentricities



- Light framed
- CLT

Analysis Flow- Not in paper



Typical Spreadsheet

Rigid Diaphragm Analysis

Longitudinal Loading

Grid Line	kx	Ky	dx	dy	kd	kd^2	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			

Loads

δ_{SW}		
0.192	Rho=	1
0.216	Ax=	1.25
0.0807	Fy=	17748
-0.081	emin=	4.75
	T=Fe=	84303

Requires Input

Input ρ, Ax

Input or calculate base shear

Transverse Loading

Grid Line	kx	Ky	dx	dy	kd	kd^2	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

Loads

0.006		
-0.006		
0.396	Fx=	17748
0.311	emin=	2.5
	T=Fe=	44370

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 ASCE 41-13, equation 7-3) ρ=1.0, Ax=1.0

Nominal wall stiffness

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}		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																	25.14
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							43.54

Calculate nominal stiffness by 3-term or 4-term deflection equation. K=F/δ

Longitudinal Analysis

Shear Walls LC7 LC7=0.726D+ρQE

Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\delta_{SW=F/Keff}$
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

δsw=F/K

Shear Walls LC6 LC6=1.374D+ρQE+0.2S

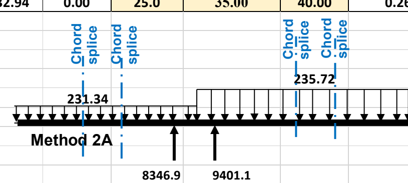
Grid Line	SW	Ga	Rho	V on wall	v	T	C	$\delta_{SW=F/Keff}$
A & B	A,B	37	1.0	1014.3	126.8	-4085.04	7128.71	0.081
2	2	30	1.0	2782.3	278.2	1477.15	4305.90	0.192
3	3	30	1.0	3133.7	313.4	1847.03	4675.78	0.216

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

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.
	1063.3	1158.3	3529.3	0.075	232.94	0.00	25.0	35.00	40.00	0.265	0.00	0.265
Nails Req'd=	4.70	5.13	15.62									
Use Nails =	8	16	24									
Slip=	0.023	0.012	0.025									

EA= 28050000, (2)2x6
Iincludes effects of sw's along chord line



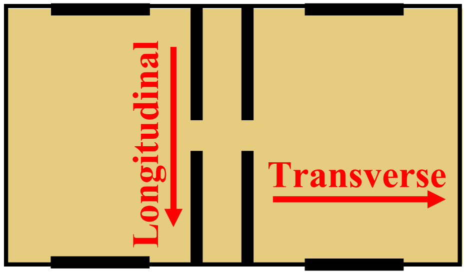
W2	W1
233.53	233.53
2.19	-2.19
235.72	231.34

Diaphragm Deflection (STR) Lft. Cantilever

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									

Analysis Flow

Longitudinal Design



Example Plan

Legend

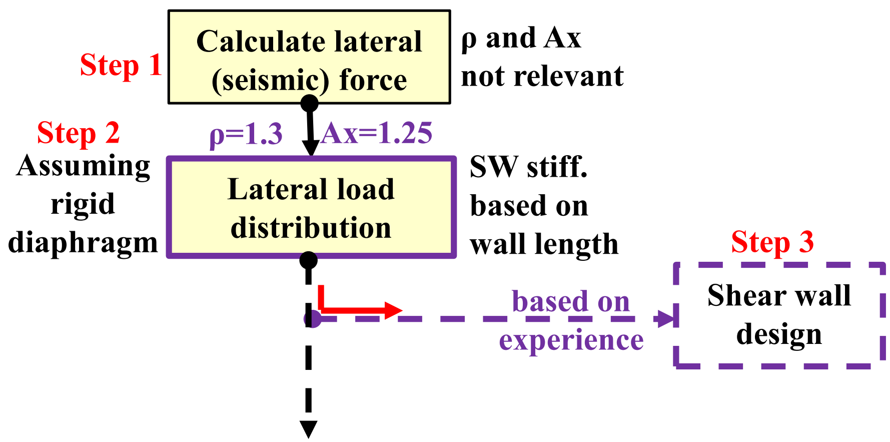
- Engineering judgement required
- SW & Diaph. Design
- Determine flexibility, Drift
- Determine Tors. Irreg., ρ, Ax

ASD Design

STR Design

Assumptions Made: Page 8

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



Force Distribution to Shear Walls

Seismic- ρ=1.3, Ax=1.25

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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**
- **Ss = 1.355 g, S1 = 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, Ω_0=3, Cd=4, Maximum height for shear wall system=65'.

Seismic Force Calculation results:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.167$$ **short period controls** 12.8-2

Basic lateral force MSFRS

V = C_SW = 0.167(35)(76)(40) = 17769 lbs. STR
17769(0.7) = 12438 lbs. ASD

Rigid Diaphragm Analysis- ρ=1.3, Ax=1.25

Initial wall stiffness will be based on wall length.

The final wall <u>Nominal stiffness's</u> are used for all final analysis checks.

<u>RDA Equations</u>

$$T = V(e)(Ax)(\rho) \text{ ft. lbs.}$$

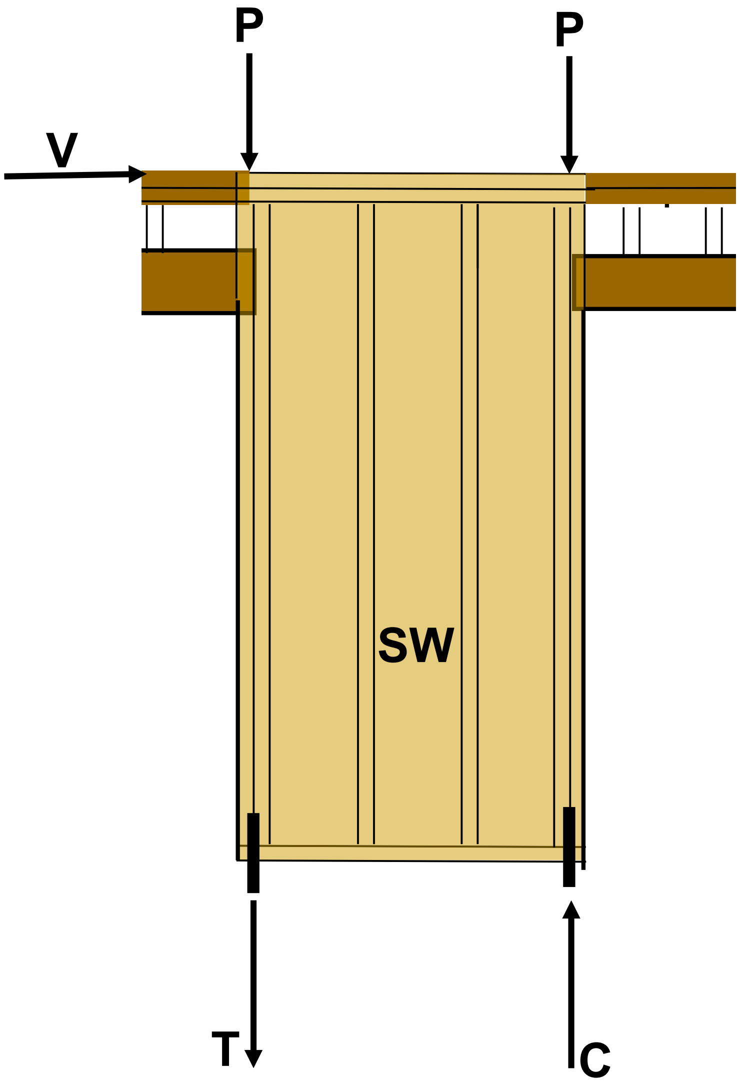
$$F_T = T\frac{kd}{\sum kd_x^2 + kd_y^2}$$

$$F_{sw} = F_V + F_T$$

$$J = \sum kd_x^2 + kd_y^2$$

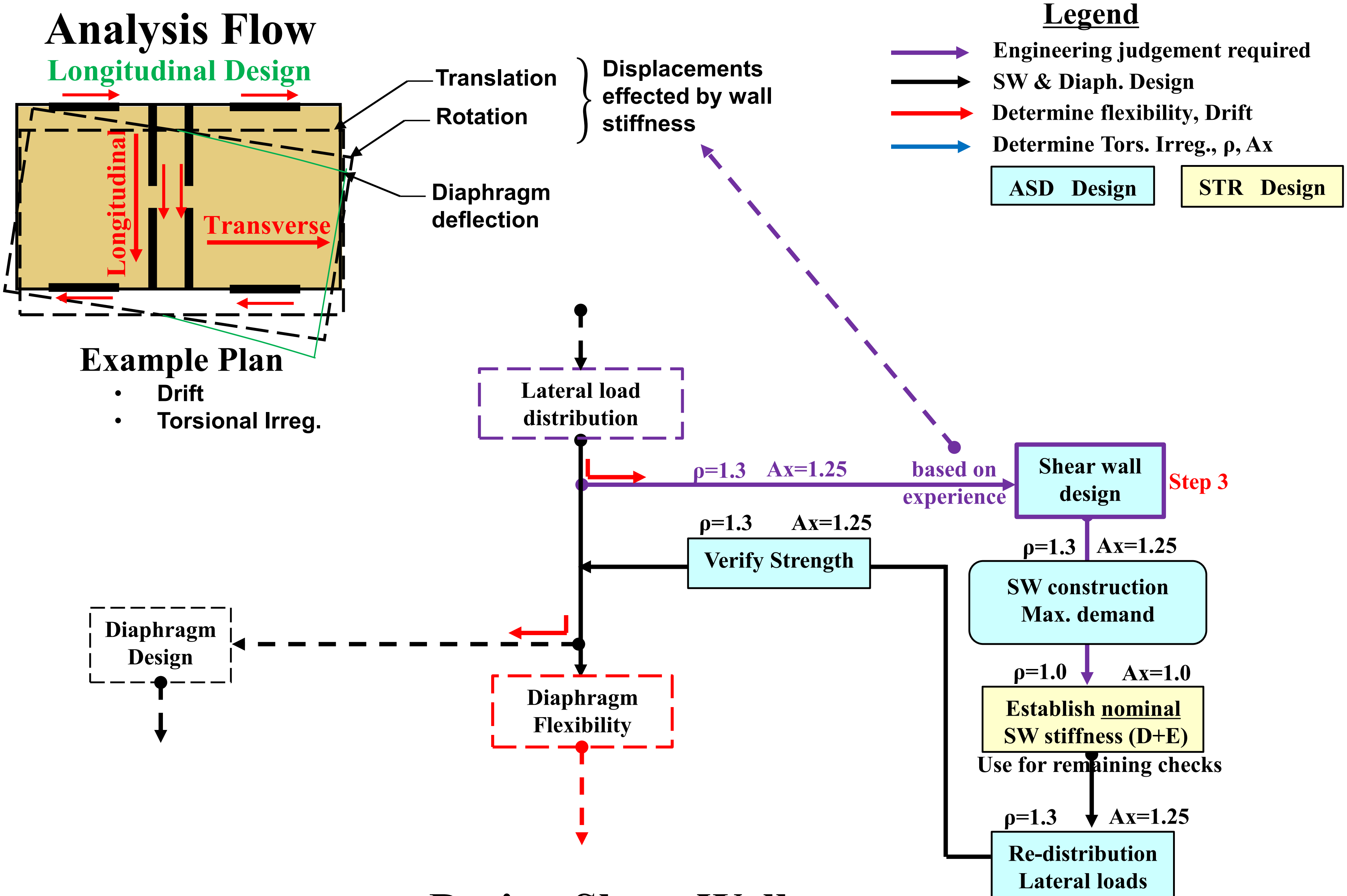
$$F_V = F_x\frac{k}{\sum k}$$

Preliminary Shear Wall Design



Analysis Flow

Longitudinal Design



Design Shear Walls

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

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

SW Design Checks

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

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

- 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.

- Calculate wall deflection

- **Shear Wall Deflection-calculated using:**

Traditional 4 term deflection equation

SDPWS combines

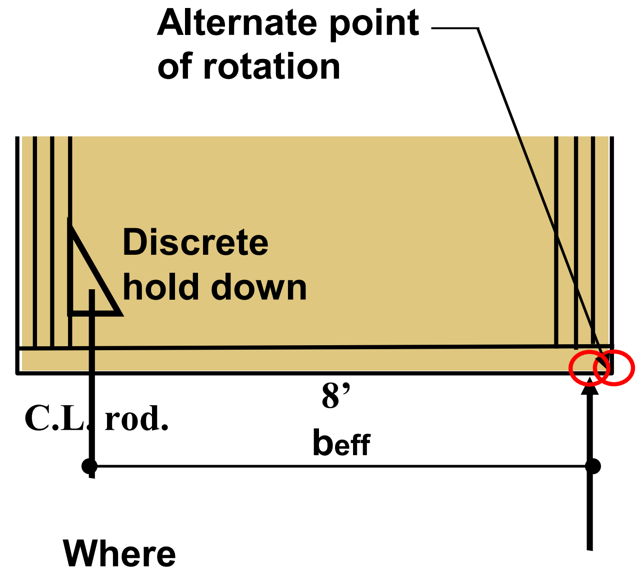
$$\delta_{SW} = \frac{8vh^3}{EAb} + \underbrace{\frac{vh}{G_v t_v} + 0.75he_n}_{\text{SDPWS combines}} + \frac{h\Delta_a}{\text{beff}} \qquad \text{C4.3.2-1}$$

Bending

Shear

Nail slip

Rod elongation (Wall rotation)



SDPWS 3 term deflection equation

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

Bending

Vertical elongation
- Device elongation
- Rod elongation

Apparent shear stiffness
- Nail slip
- Panel shear deformation

Where

v=wall unit shear (plf)

h=wall height (ft.)

beff =Wall rotation width (ft.)

b=Wall width (ft.)

Ga=apparent shear stiffness (k/in.)

Δ_a=Sum of vertical displacements at anchorage and boundary members (in.)

Note:

Calculate wall deflection as: $\delta_{sw\ A,B} = \frac{F}{k}$

after Nominal stiffness has been established

Causes of Wall Rotation

- 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

- 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.

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

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

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

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

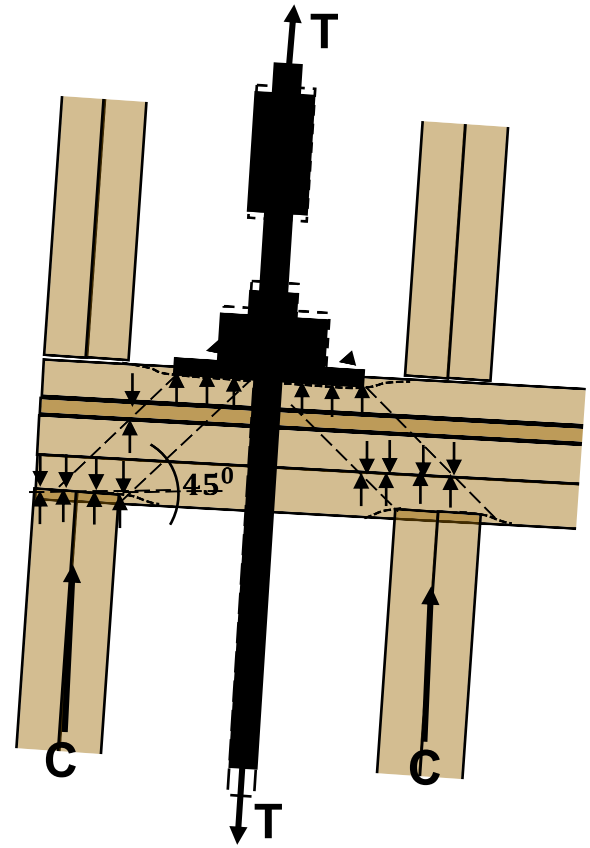
When $F_{c\perp 0.02}" \leq f_{c\perp} \leq F_{c\perp 0.04}$"

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

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

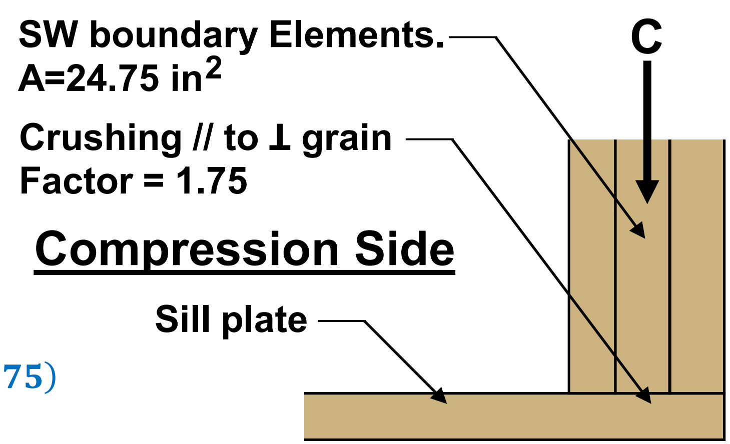
$$\Delta_{crush} = 0.04\left(\frac{f_{c\perp}}{F_{c\perp 0.04}}\right)^3$$

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



Tension Side

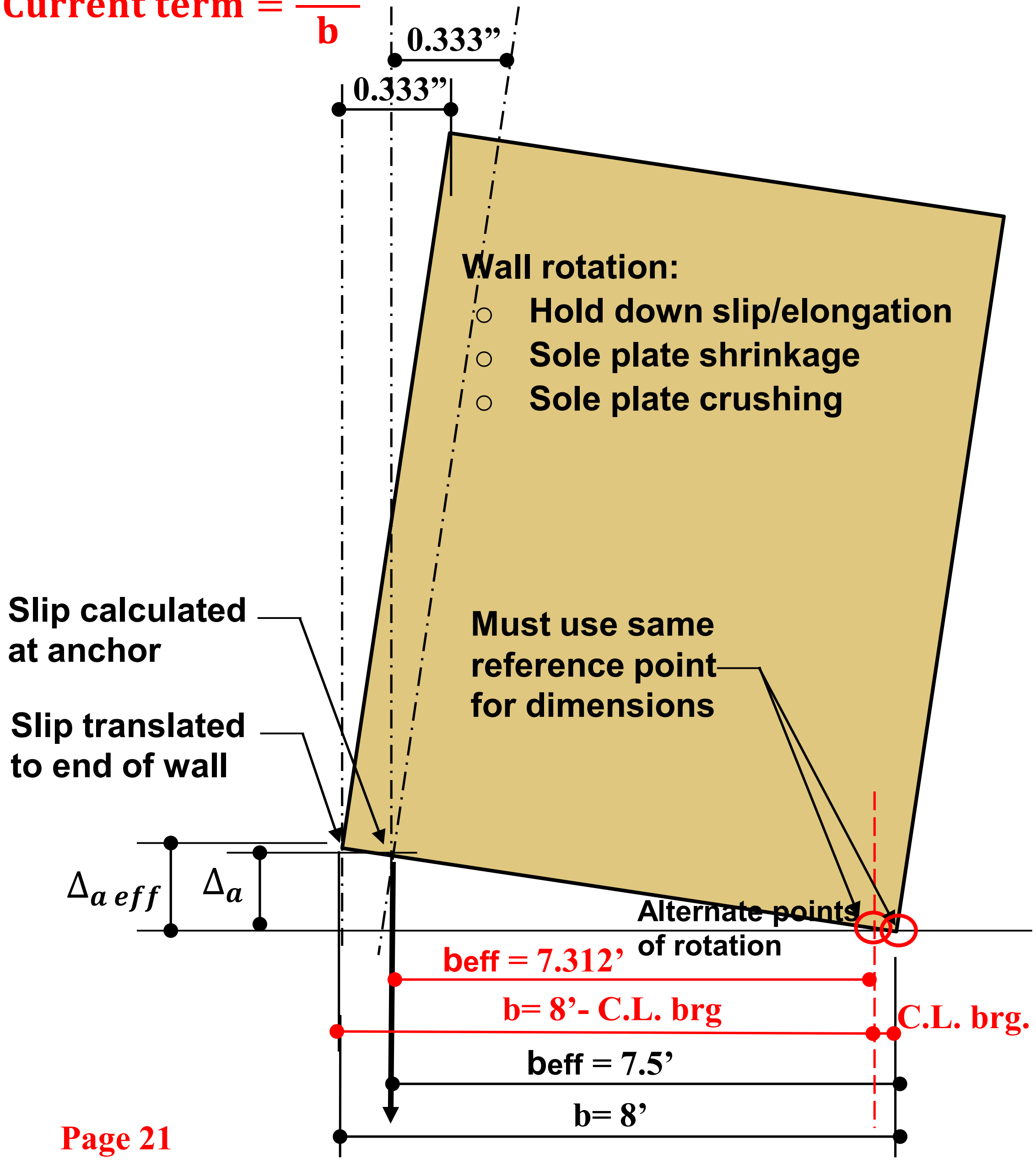
If cont. tie rod

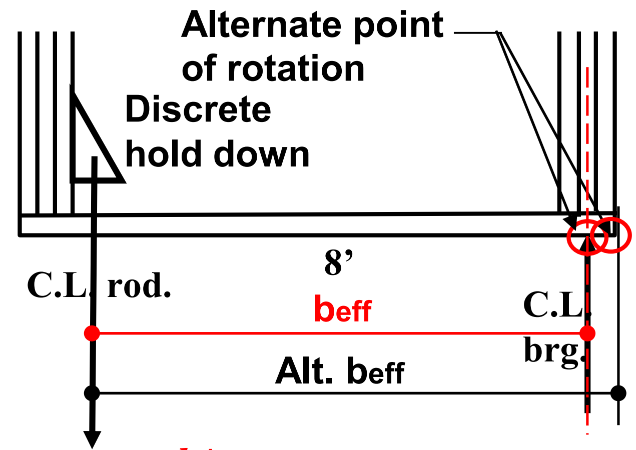


Shear Wall Rotation

Proposed nomenclature of next edition of SDPWS

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





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

Where

h=wall height (ft.)

beff =Wall rotation arm (ft.)

b=Wall width (ft.)

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

Δ_a=Sum of vertical displacements at tension edge of wall

Δ_a= 0.25”

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

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

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

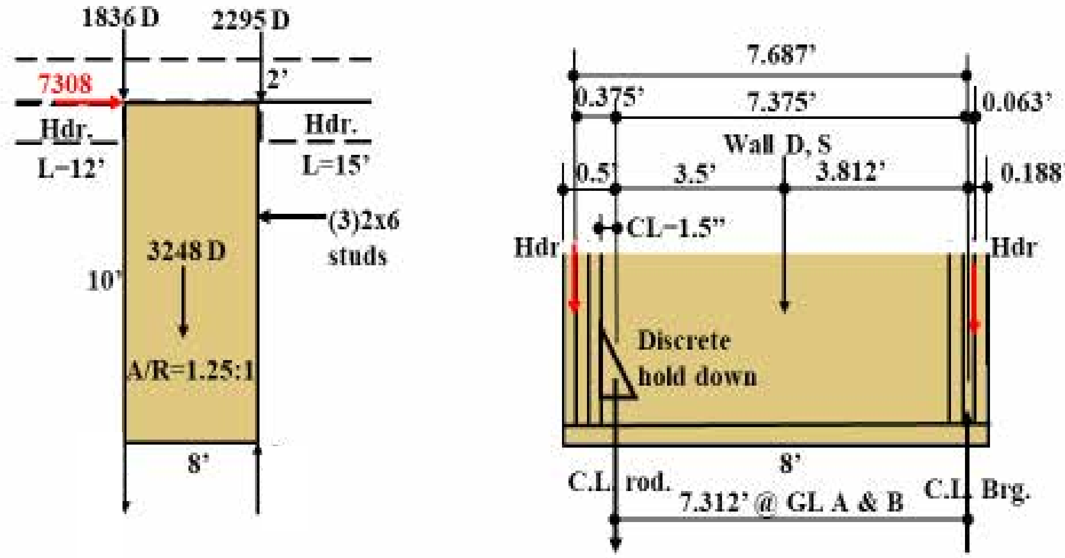
Load Combinations (ASD):

LC8 $= 1.152D + 0.7\rho Q_E$

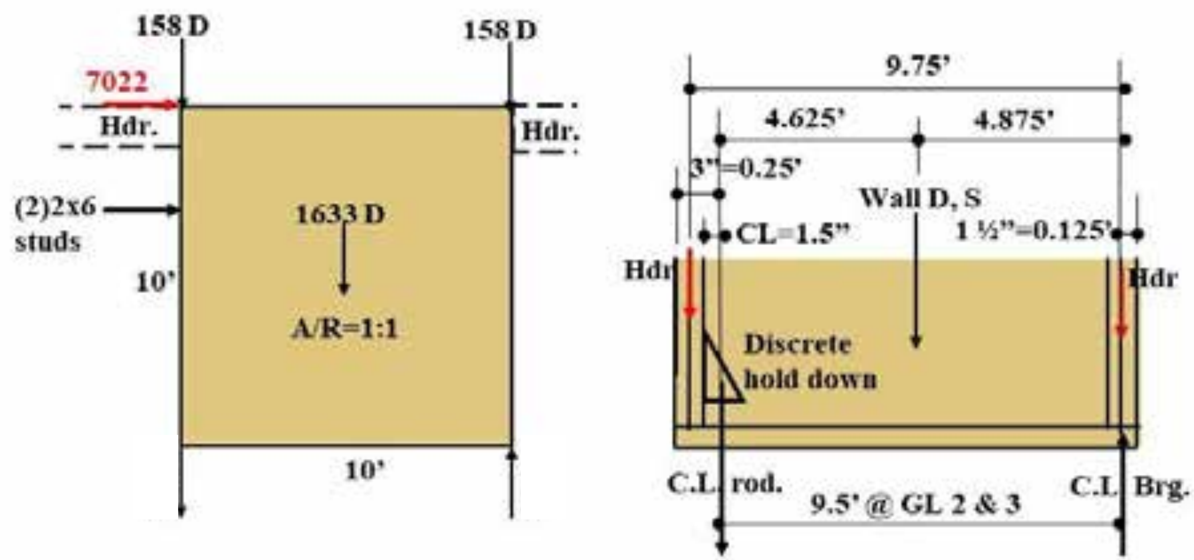
LC9 $= 1.114D + 0.525\rho Q_E + 0.75S$

LC10 $= 0.448D + 0.7\rho Q_E$

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, ρ=1.3, Ax=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^2	Fv Lbs.	F_T 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

ΣKy=60 ΣKx=32

J=16169.8

Corridor Walls at Grid lines A & B Walls

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

SW Line	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd^2	Fv Lbs.	F_T 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

ΣKy=60 ΣKx=32

J=16169.8

Corridor Walls at Grid lines A & B Walls

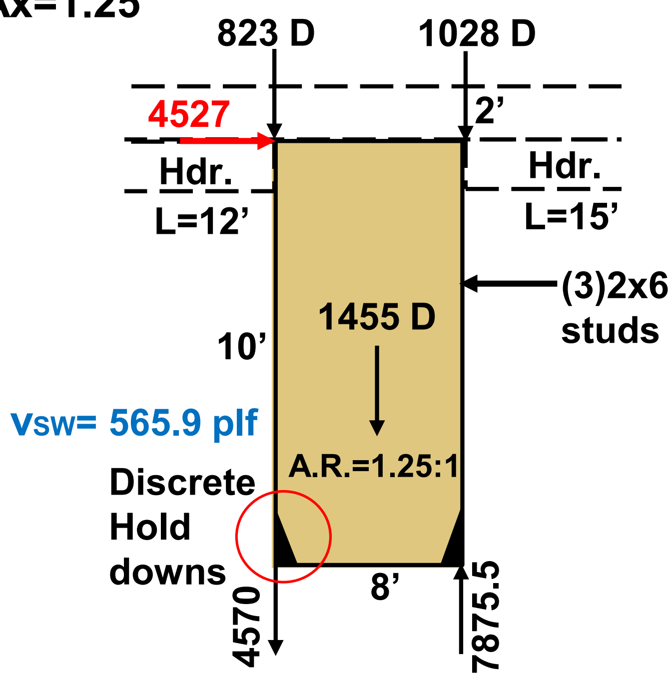
Preliminary Shear Wall Design-Distribution based on wall lengths

Adding Gravity Loads to Shear Walls

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

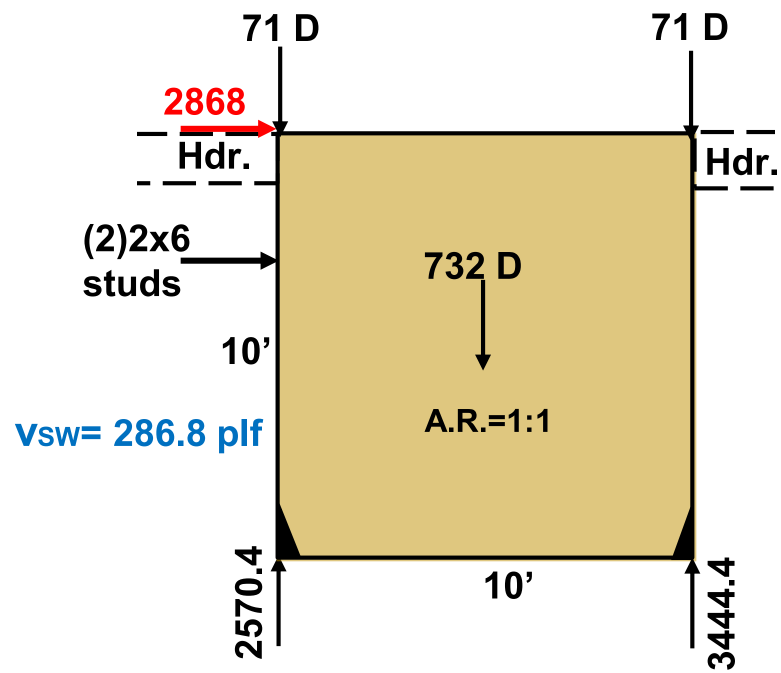
ASD Load Combination: LC10 = 0.448D+0.7ρQ_E

ρ=1.3, Ax=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

$V_{SW\ A,B}$ = 565.9 plf
$V_{SW\ 2,3}$ = 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[4]											
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 Panel Edge Fastener Spacing (in.)				B Wind Panel Edge Fastener Spacing (in.)			
				6 (plf) (kips/in.)	4 (plf) (kips/in.)	3 (plf) (kips/in.)	2 (plf) (kips/in.)	6 (plf)	4 (plf)	3 (plf)	2 (plf)
				Vs Ga OSB PLY	Vs Ga OSB PLY	Vs Ga OSB PLY	Vs Ga OSB PLY	Vw	Vw	Vw	Vw
Wood Structural Panels-Sheathing [4,5]	15/32	1-3/8	8d	520 13 10	760 19 13	980 25 15	1280 39 20	730	1065	1370	1790
	15/32			620 22 14	920 30 17	1200 37 19	1540 52 23	870	1290	1680	2155
	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.

$SW_{A,B}$: Use 15/32" OSB w/ 10d@3" o.c., v_s= (1200)/2 = 600 plf, Ga=37
$SW_{2,3}$: Use 15/32" OSB w/ 10d@4" o.c., v_s= (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.

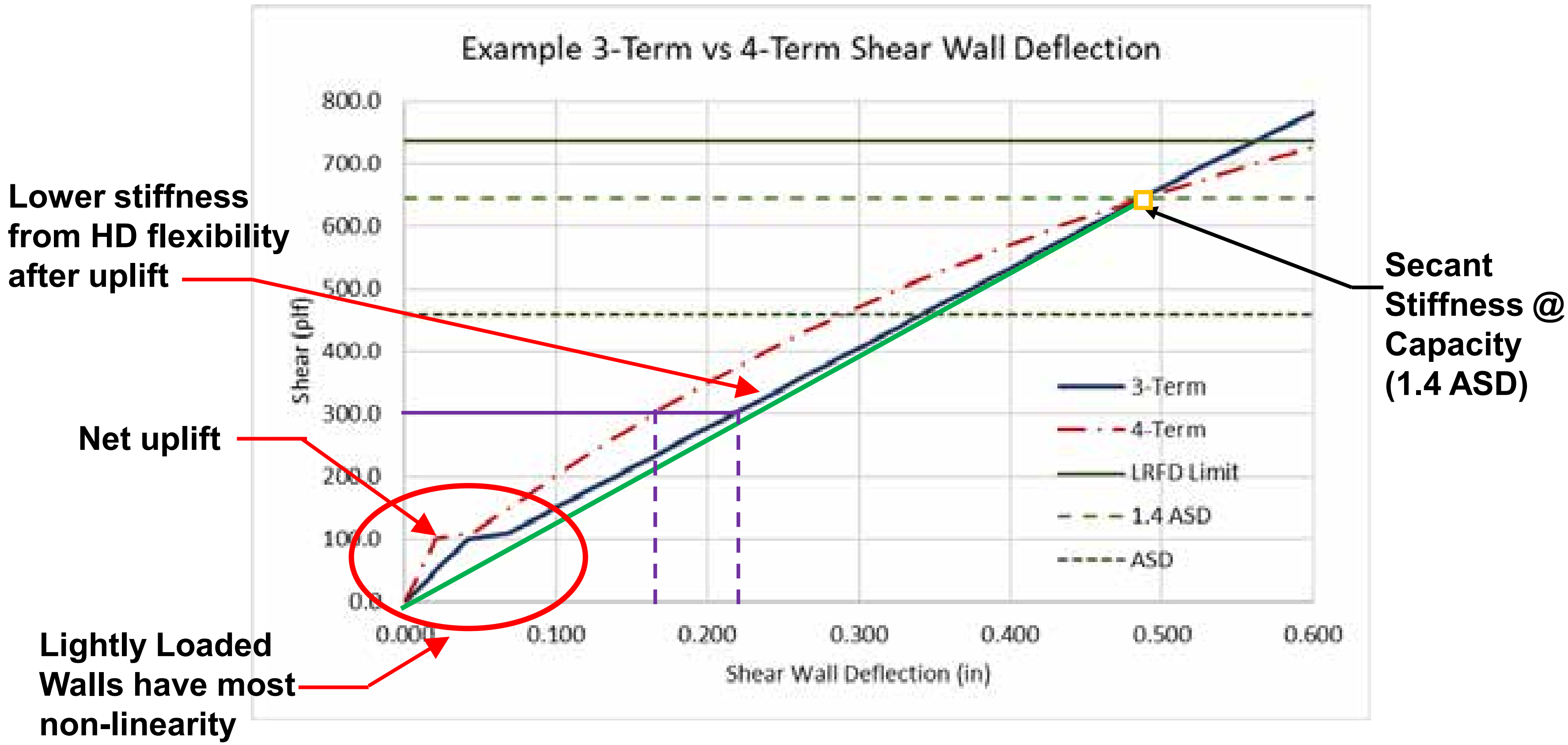
Sources of non-linearities:

- **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

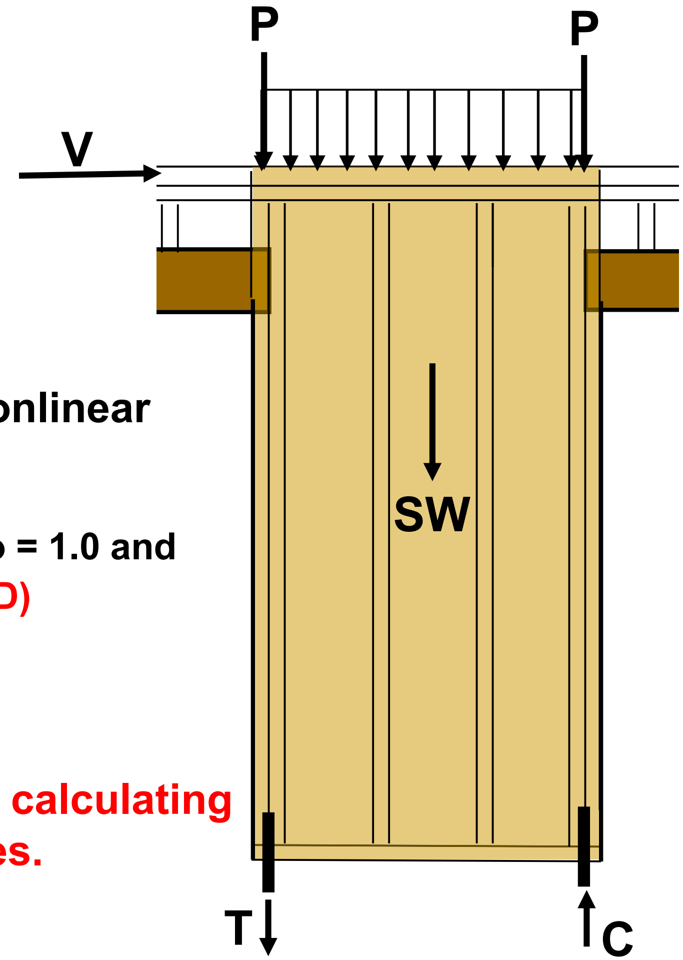
- **3-term equation is a linear simplification of the 4-term equation, calibrated to match the applied load at 1.4 ASD.**
- **This simplification removes the non-linear behavior of e_n.**
- **Similar approach can be used to remove non-linear effects of Δ_a by calculating the wall stiffness at strength 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 ρ = 1.0 and Ax = 1.0. Vertical seismic loading not included. (E_V=0.2S_{DS}D)**
>
> **For multi-story buildings, suggest 1.0D+ αL 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 ρ=1.0 and Ax=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.**

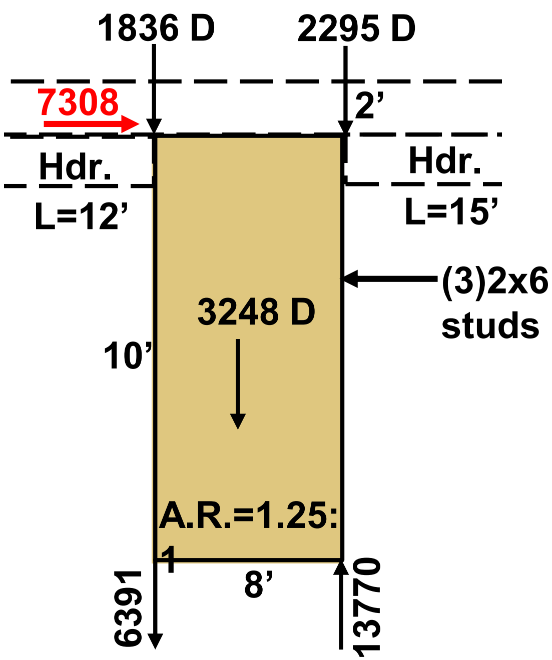
Nominal Shear Wall Stiffness's (STR) ρ=1.0, Ax=1.0

Load Combination: $1.0D + Q_E$

Grid Line	Ga	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 LRFD 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

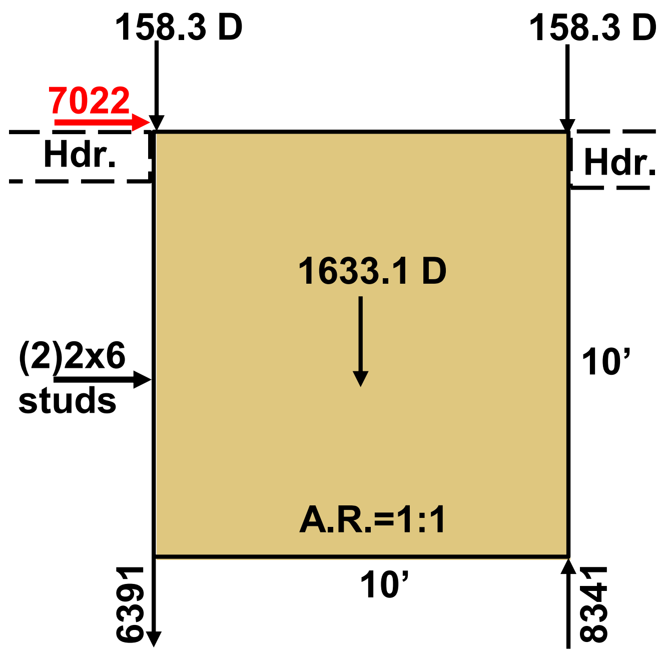
Wall Capacity based on hold down

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



Shear wall Grid A and B
Trib. = 10'

Transverse Loading
Nominal Strength



Shear wall Grid 2 and 3
Trib. = 2'

Longitudinal Loading
Nominal Strength

Max. capacity check (STR):

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

$H.D._{A,B,2,3}$=6391 lbs.(STR),
Δa=0.154"

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

V $_{allow. A,B}$= 7308 lbs. controls
V $_{allow. 2,3}$= 7022 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. ρ=1.3, Ax=1.25

SW Line	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd^2	Fv Lbs.	FT Lbs.	Total Lbs.	
A	-------	25.14	--------	20	502.8	10056	0	1848.1	1848.1	Walls at Grid lines A & B
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	Corridor Walls
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. ρ=1.3, Ax=1.25

SW	Ky k/in	Kx k/in	Dx Ft.	Dy Ft.	Kd	Kd^2	Fv Lbs.	FT Lbs.	Total Lbs.	
A	-------	25.14	--------	20	502.8	10056	8084.9	972.7	9057.6	Walls at Grid lines A & B
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	Corridor Walls
3	43.54	--------	3	--------	130.62	391.86	0	-252.7	-252.7	

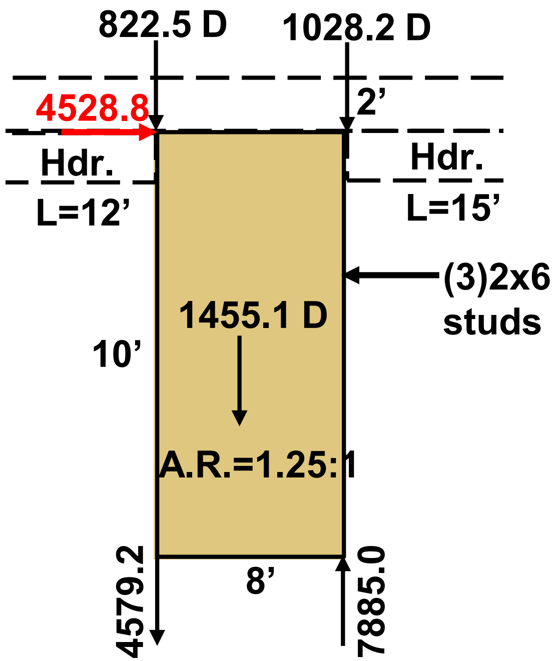
ΣKy=87.08 ΣKx=50.28

J=20895.72

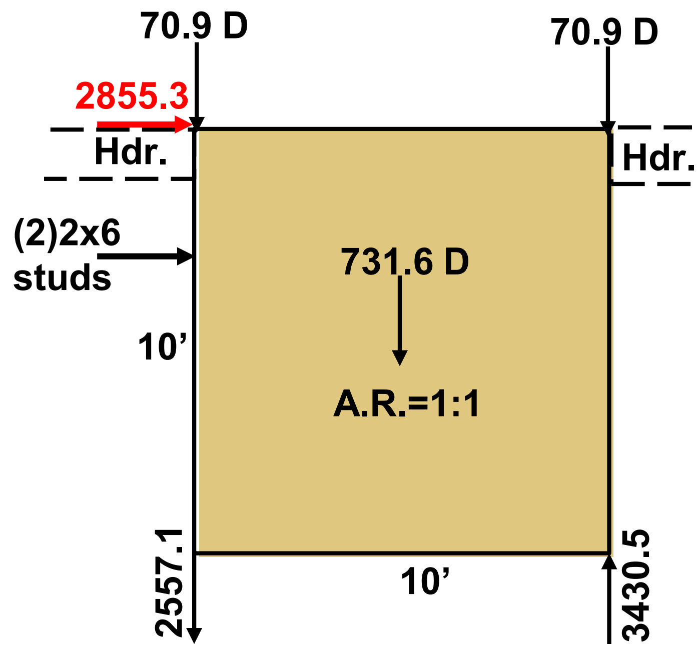
Nominal stiffness values used

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

ρ=1.3, Ax=1.25



Shear wall Grid A and B



Shear wall Grid 3

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

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

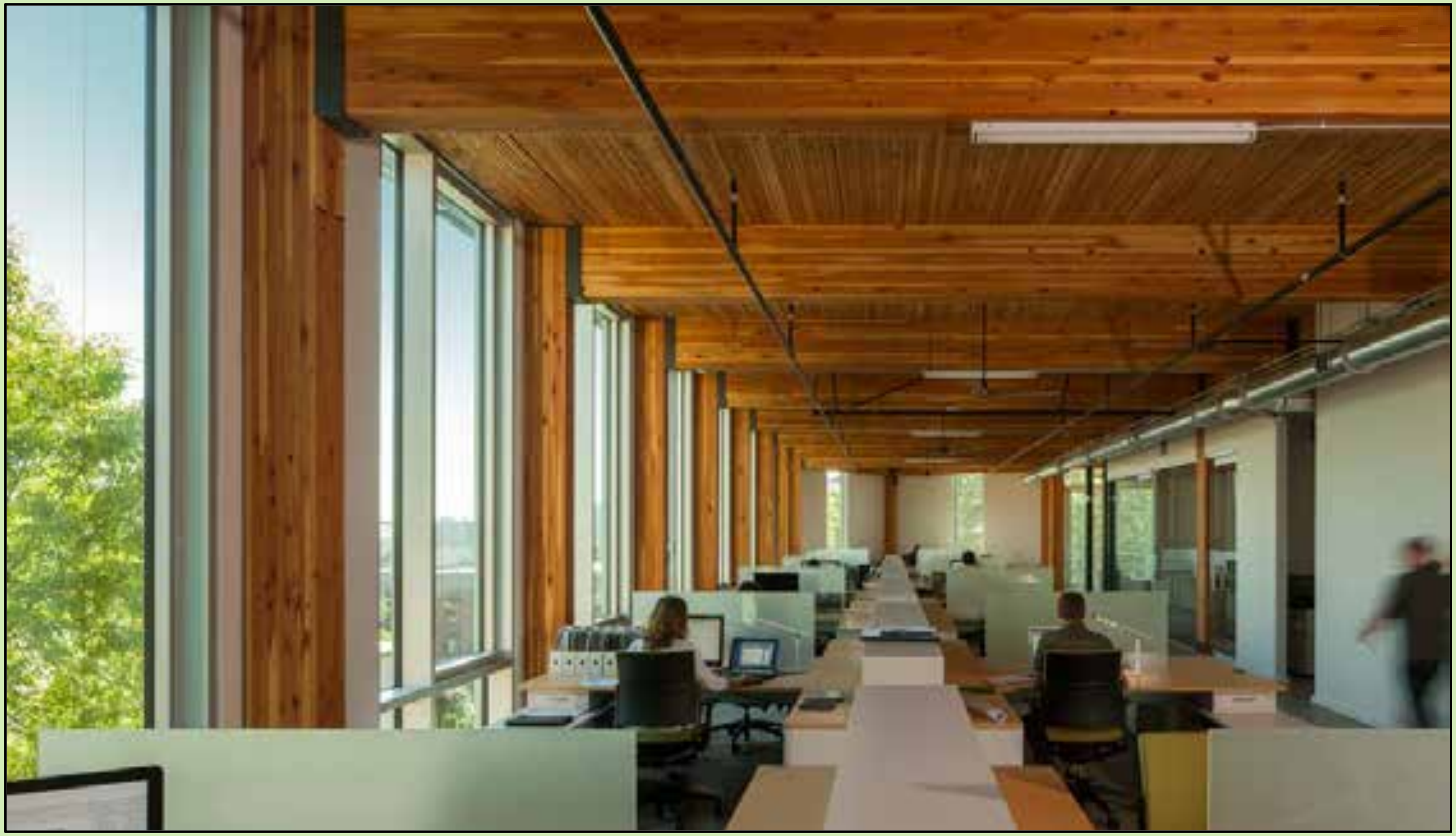
vs = $\frac{4528.8}{8}$ = **566.1** plf <**600** plf allowed ∴ o.k.

T= **4579.2** lbs. ≈ **4565** lbs. allowed, **0.3% over**
∴ hold down o.k. –check later

vs = $\frac{2855}{10}$ = **285.5** plf. < **460** plf allowed ∴ o.k.

T = **2557.1** lbs. < **4565** lbs. allowed
∴ hold down o.k.

Let’s Take a 15 Minute Break



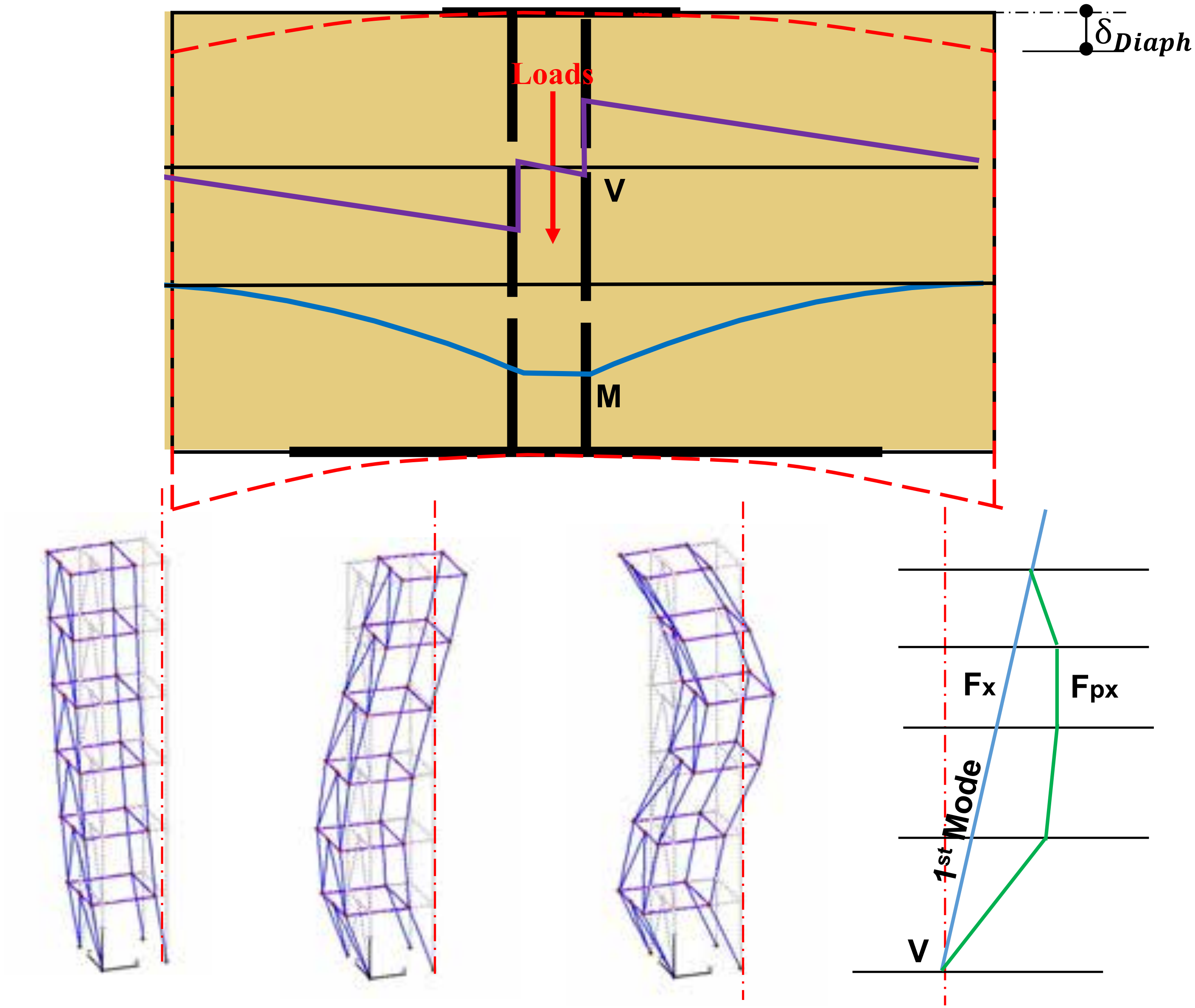
Mass Timber Project

Part 3 Content

Part 3-Design Example (cont.):

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

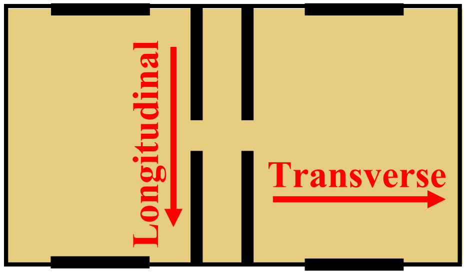
Diaphragm Design



Diaphragm Design Forces: MSFRS or F_{px}

Analysis Flow

Longitudinal Design

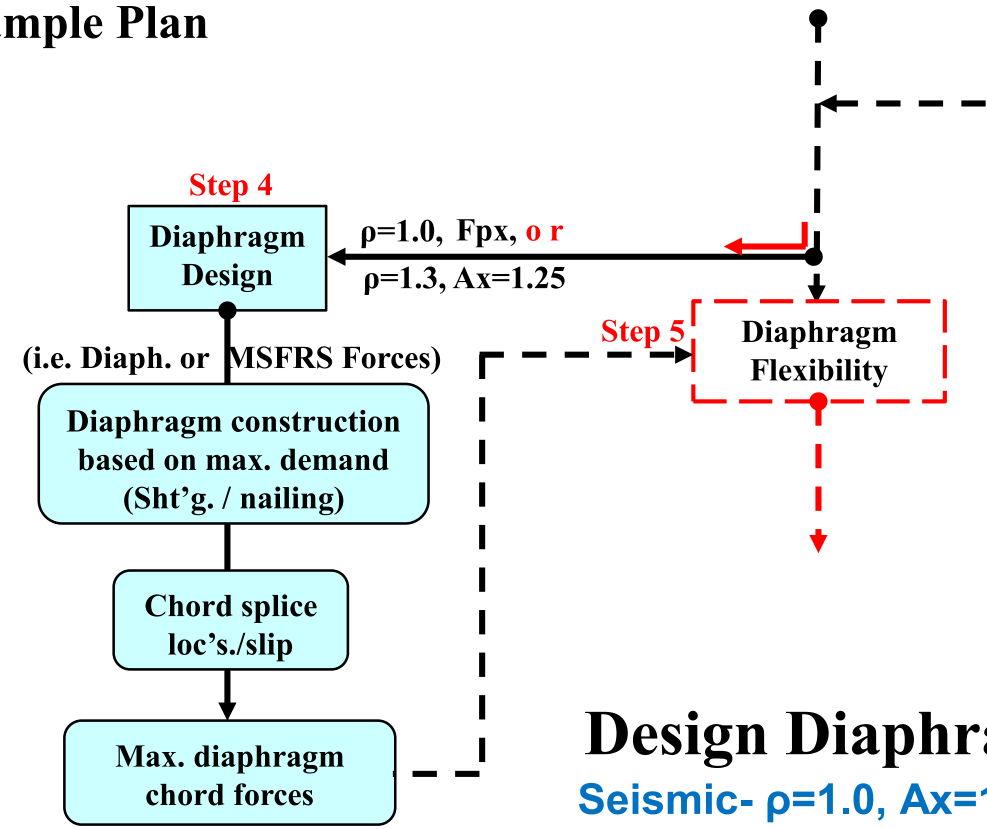


Example Plan

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

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:**

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

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, $\rho=1.0$ per ASCE 7-16 Section 12.3.4.1, Item 7.

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

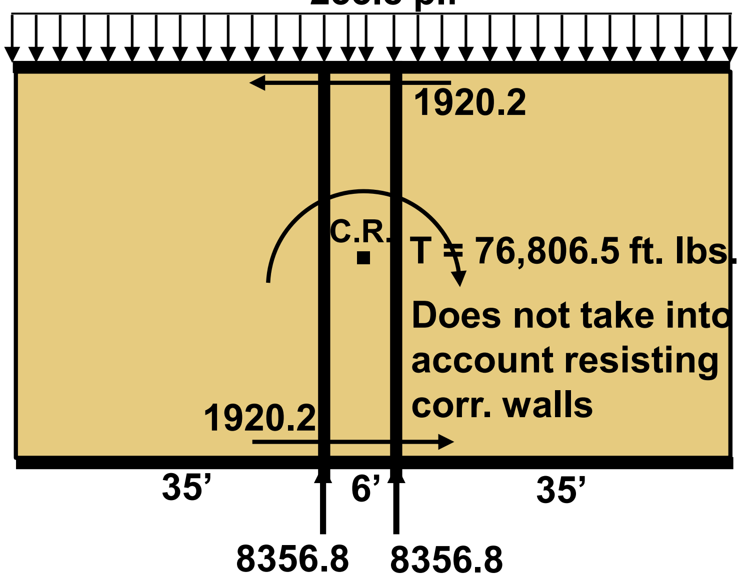
Torsional Distribution-Not mandatory

(Question 4) ρ=1.3, Ax=1.25

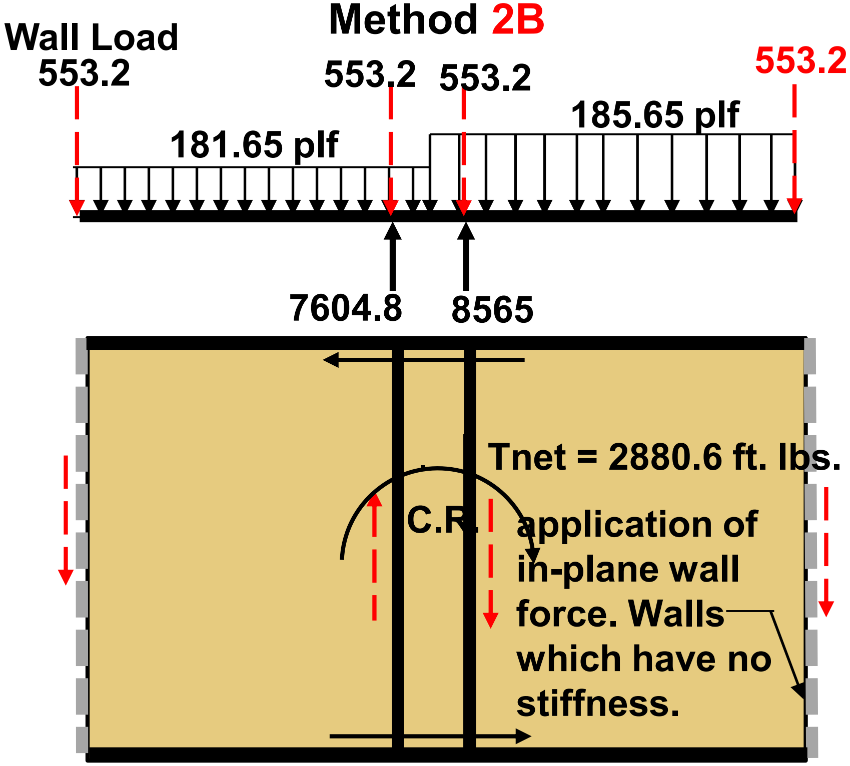
Method 2B will be used for diaphragm design
(To answer questions 5 and 6)

Method 2A will be used for all other checks

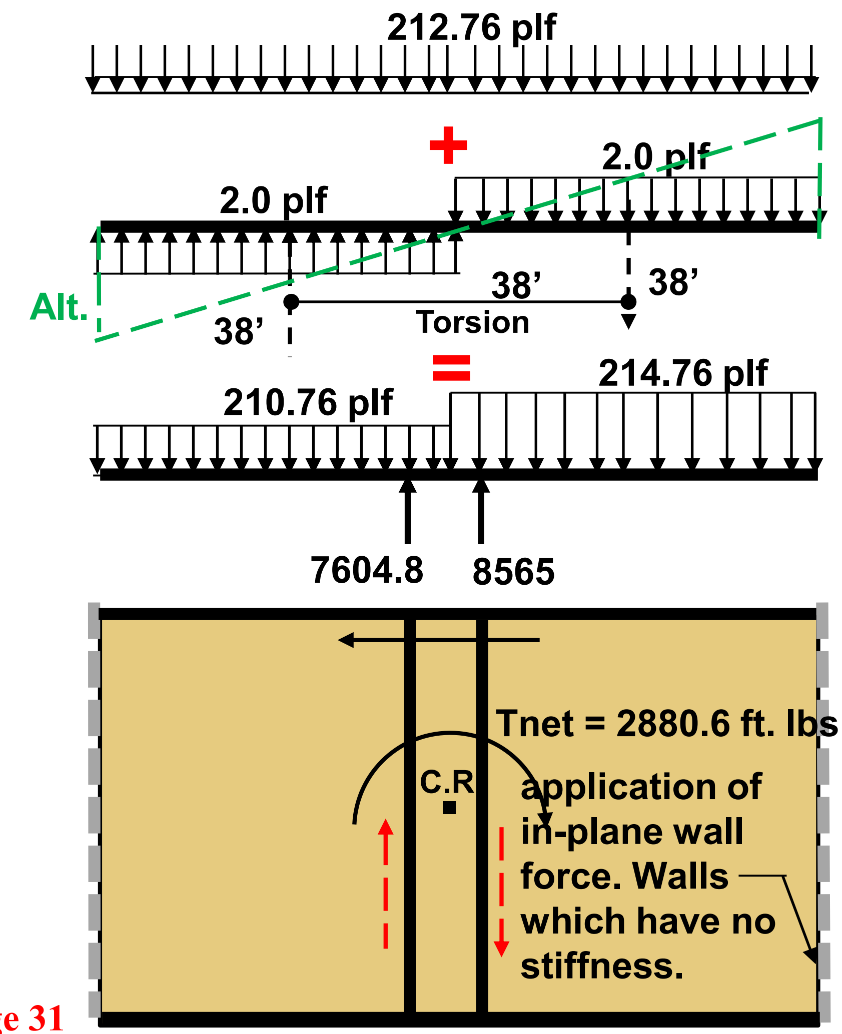
Method 1



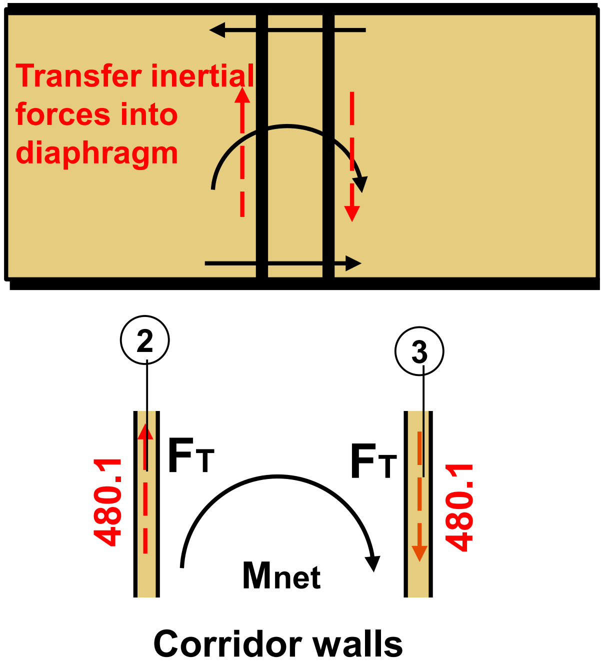
Method 2B



Method 2A



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

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

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

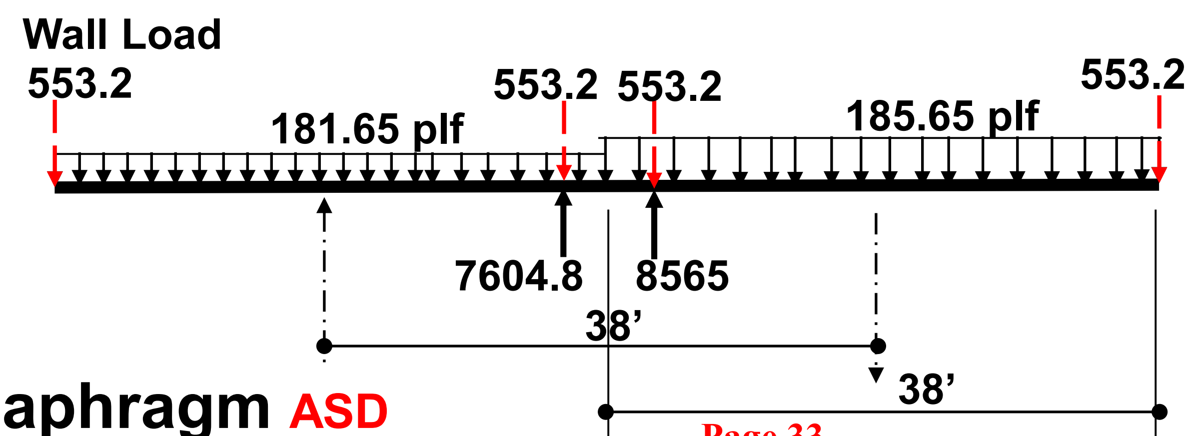
$V_{net} = V_{base} - F_{1,2,3,4}$ =12438.3(1.3) - 4(553.2) = 13957 lbs.

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

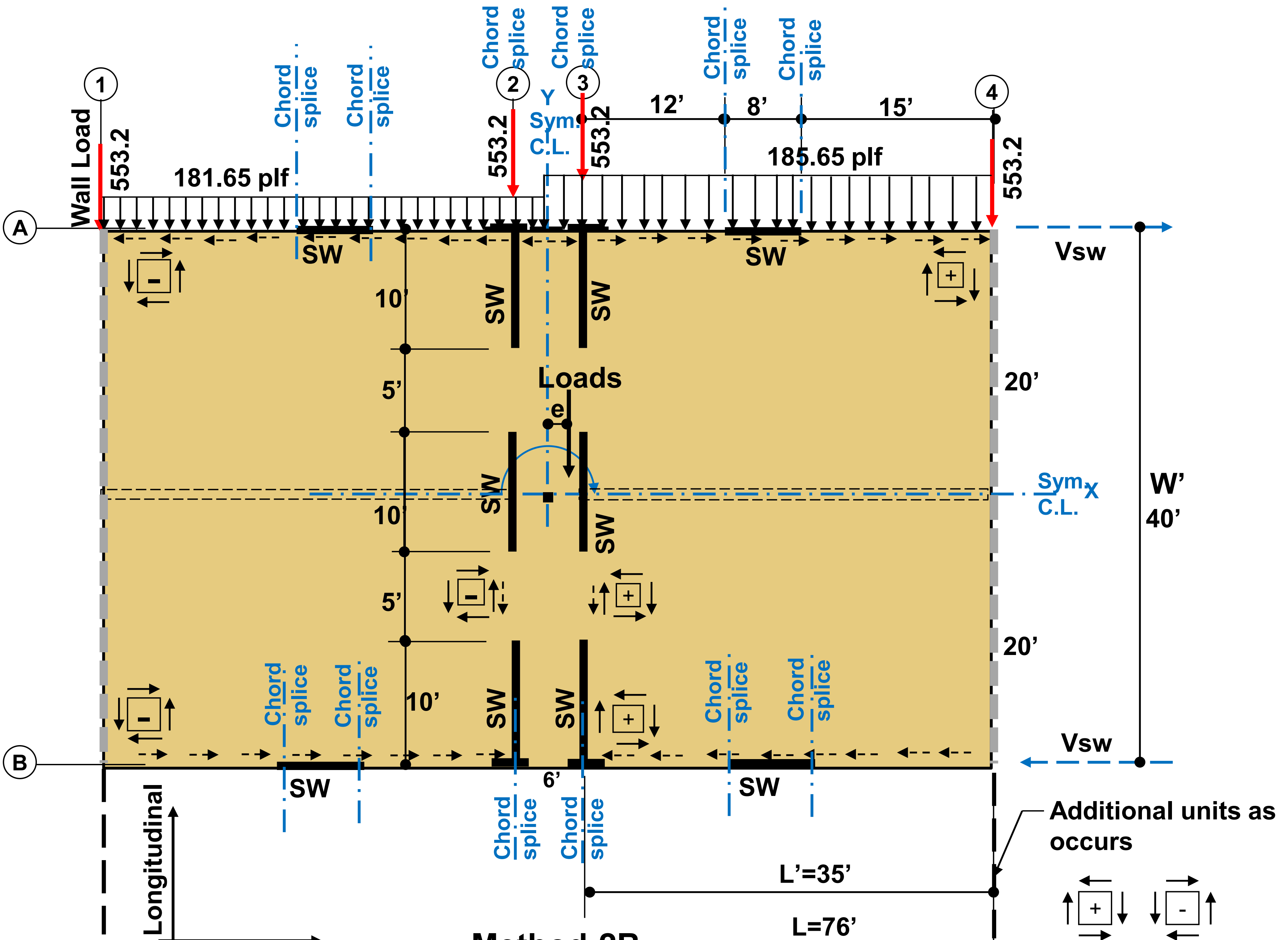
$W_T = \frac{2880.6}{38(38)}$ = 2.0 plf: equivalent uniform torsional load acting as M_{net}

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

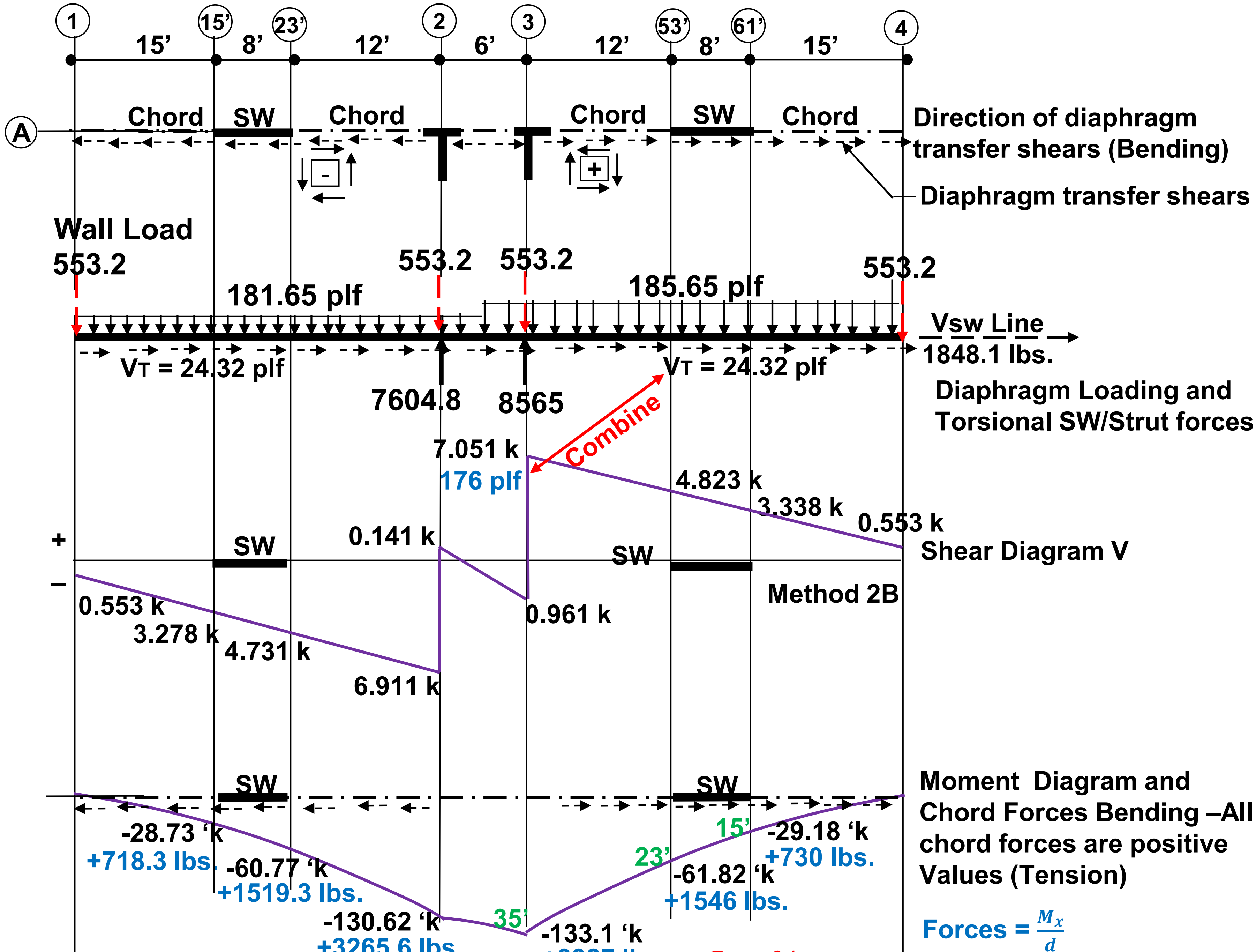


Calculate Loads to Diaphragm ASD



Wall Load
553.2
181.65 plf
185.65 plf
Chord splice
Y
Sym.
C.L.
12'
8'
15'
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'
23'
2
3
53'
61'
4
15'
8'
12'
6'
12'
8'
15'
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
Combine
Diaphragm Loading and Torsional SW/Strut forces
7.051 k
176 plf
4.823 k
3.338 k
0.553 k
+
SW
0.141 k
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 < vs = 0.5(580) = 290 plf. o.k.

G_a = 25, blocked

Visual Aid-Shear

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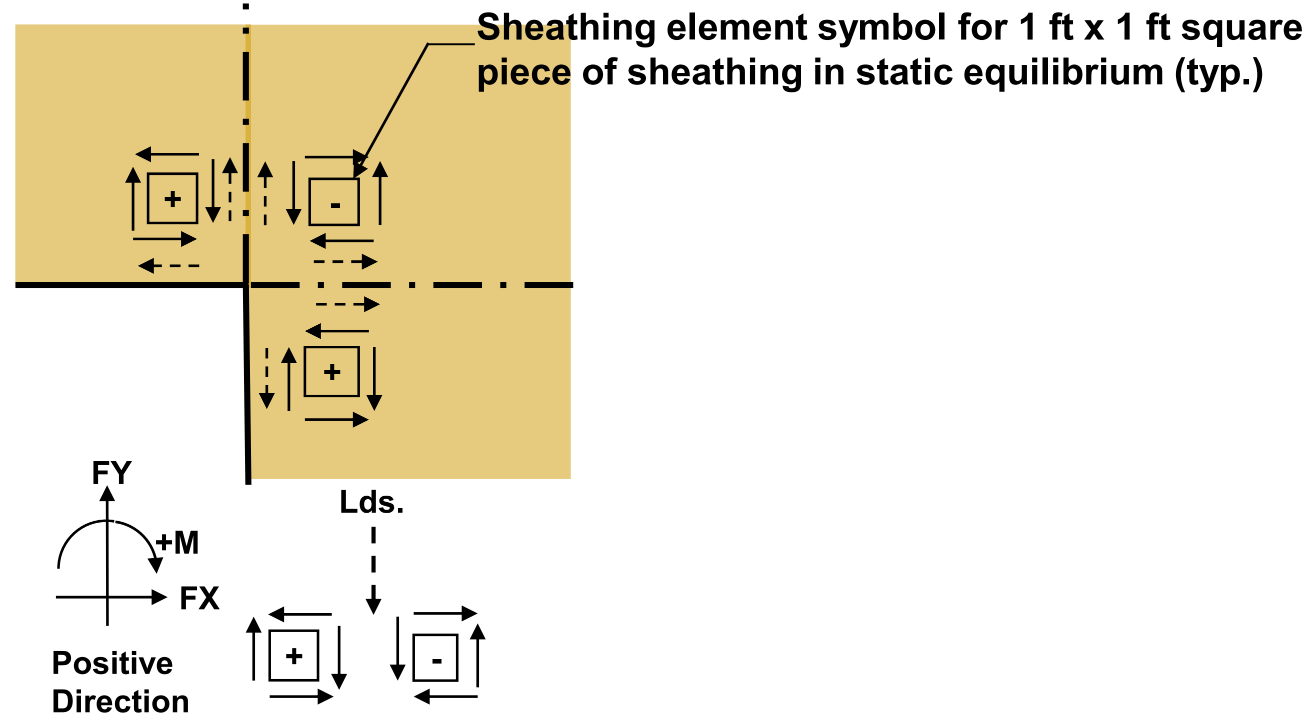
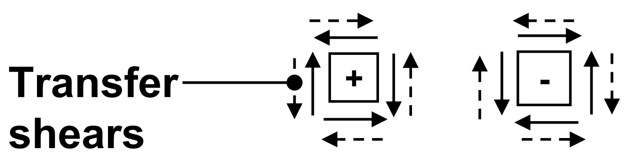


Longitudinal Direction (shown)

Shears Applied to Sheathing Elements

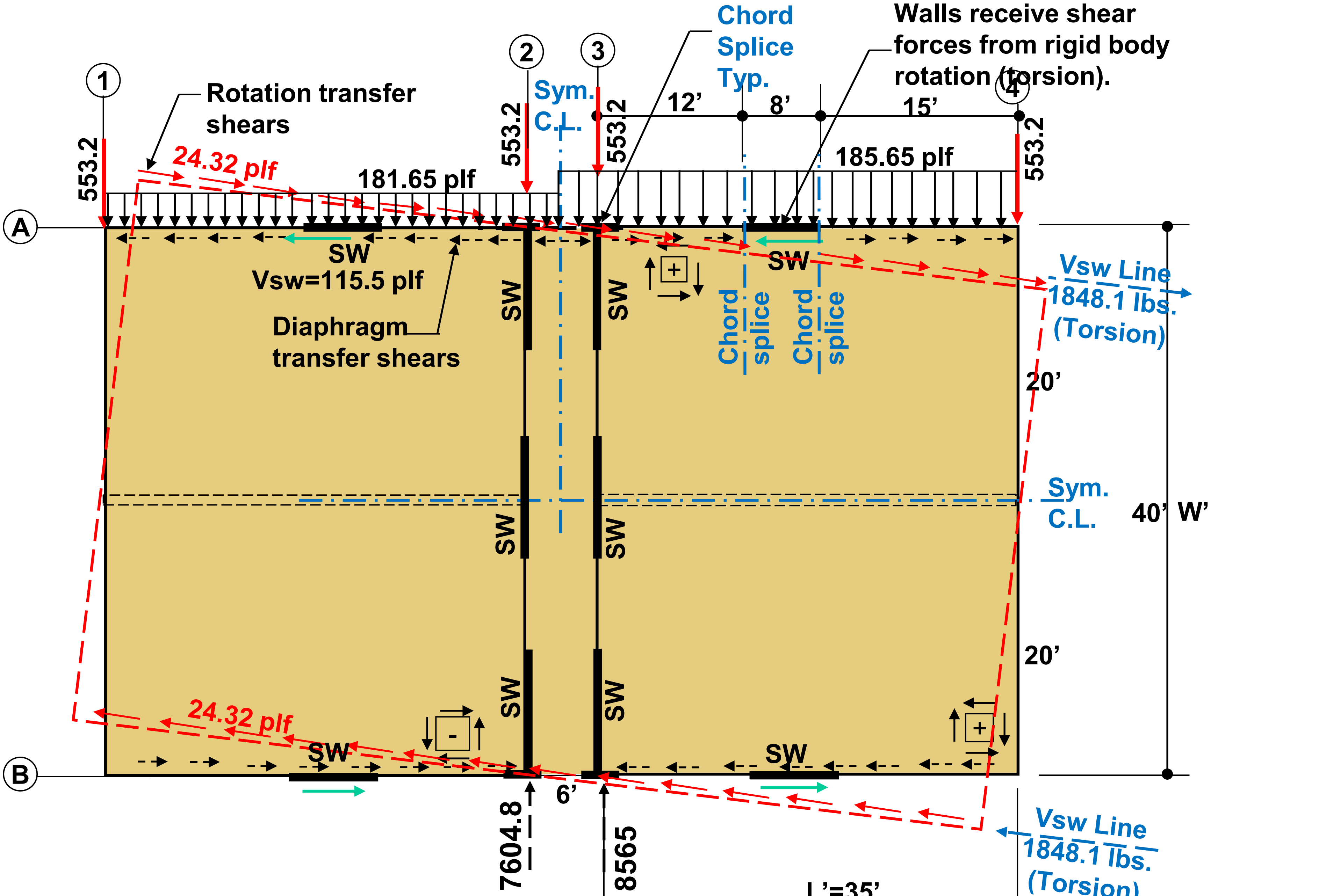


Unit shear acting on sheathing element (plf)

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

Shears Transferred Into Boundary Elements

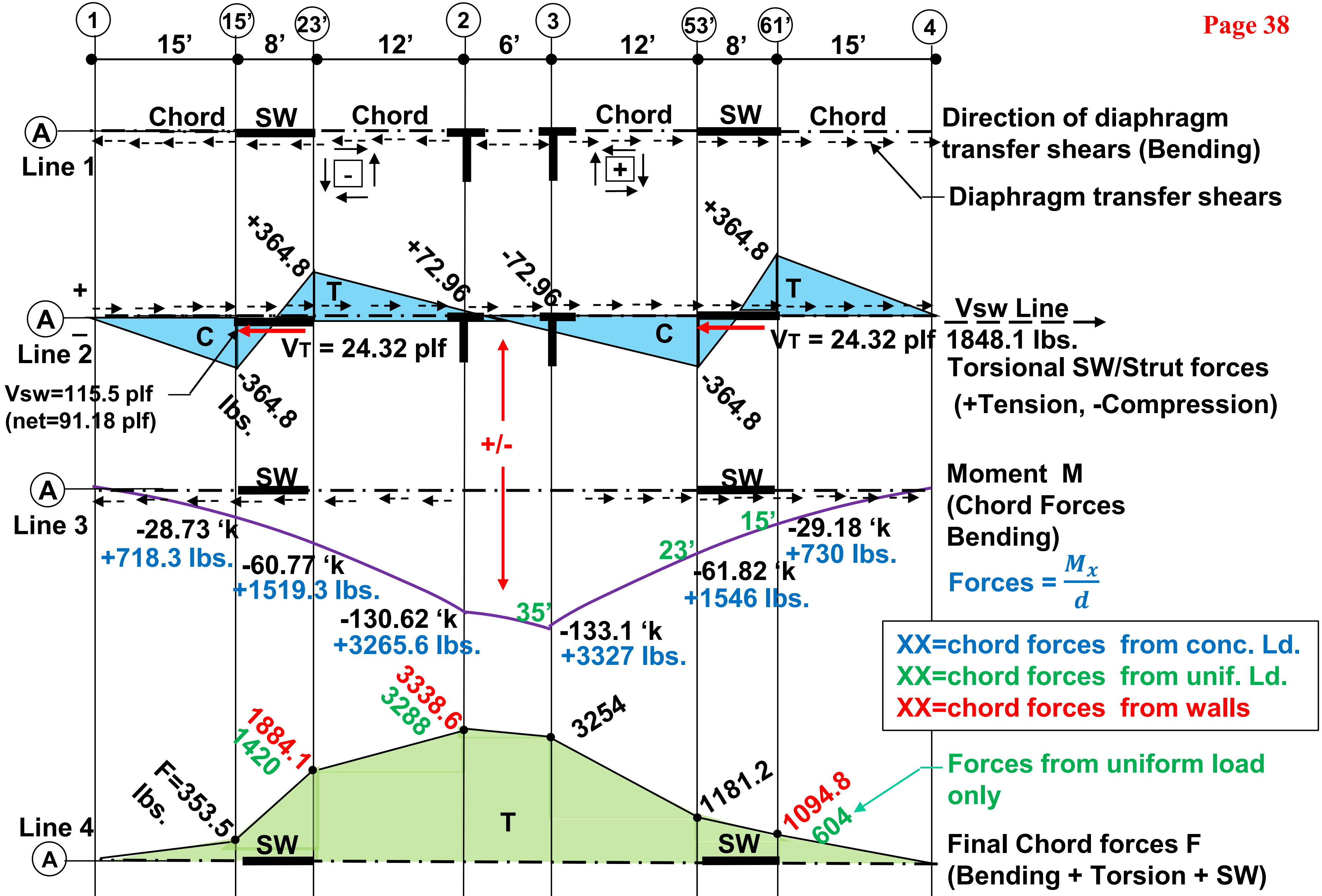
The Visual Shear Transfer Method. How to visually show the distribution of shears through the diaphragm



Determine Maximum Chord Force

(Answer questions 5 and 6)

Page 36, 37



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.

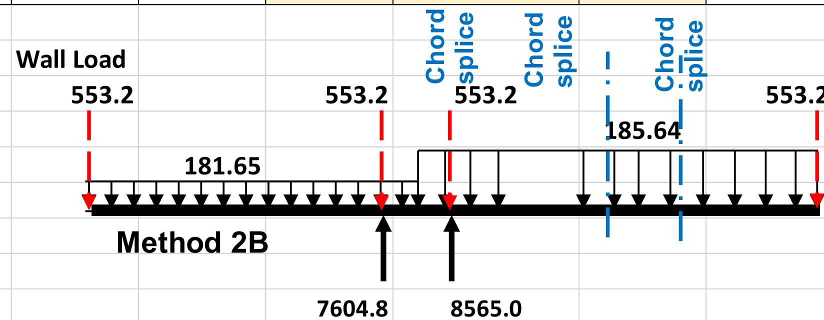
Diaphragm Chords

Diaphragm Deflection (ASD)

	Splice Forces (Lbs.)			Σδ_slip	v unif.	v conc.	Ga	L'	W'	δDiaph Uni	δDiaph con	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										

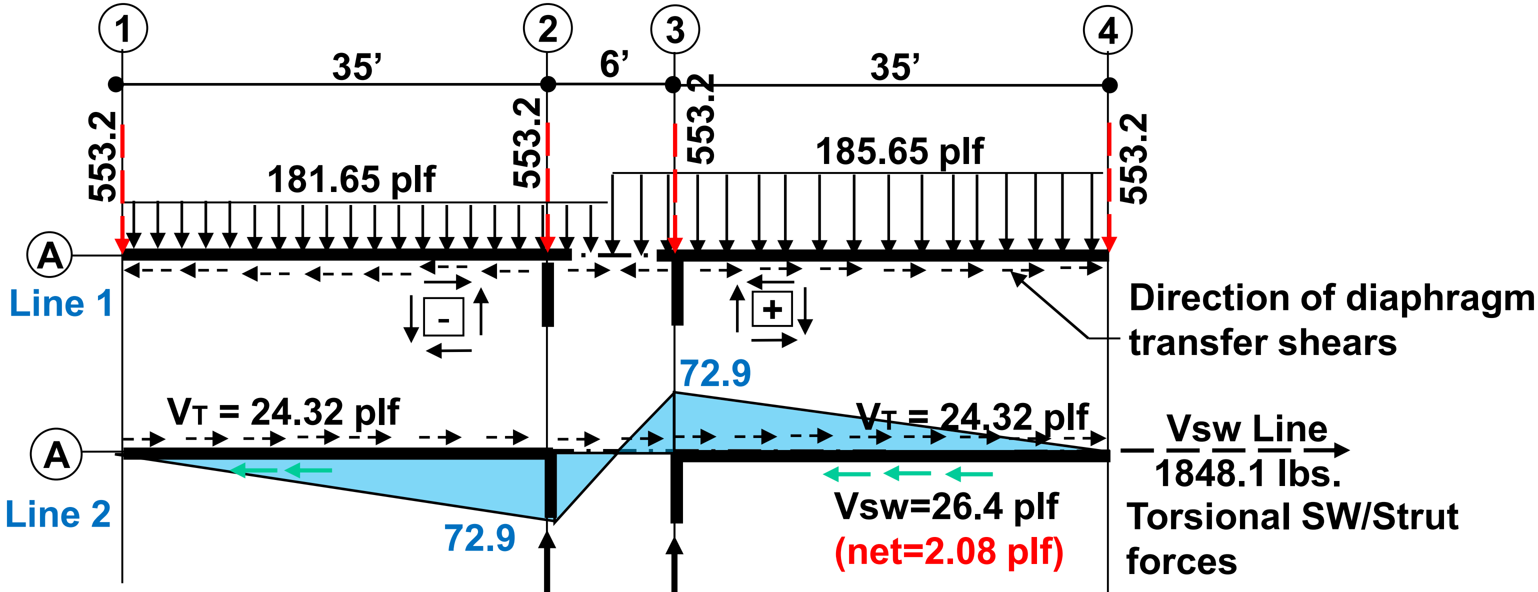
Maximum chord force = 3338.5 lbs.

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.

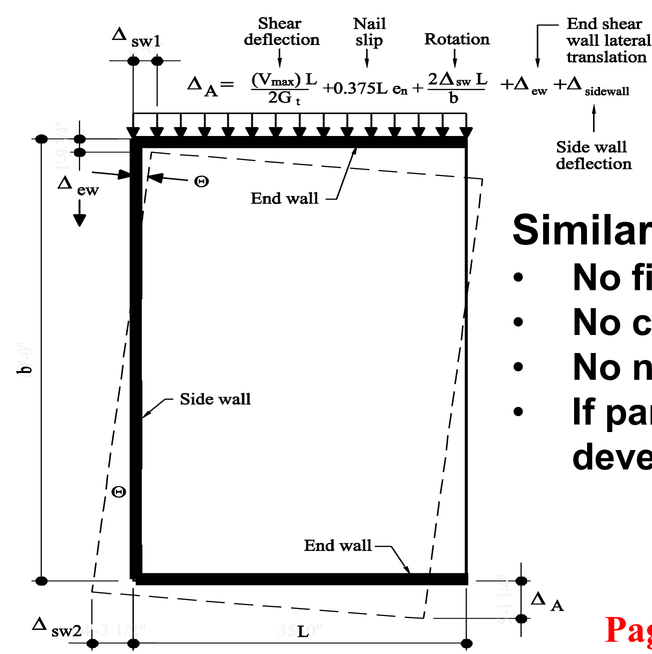
$f_t = \frac{F_{chord}}{(1)2x6}$, **Number of nails =** $\frac{F_{chord}}{226}$ **, where 226 lbs. is adjusted lateral design value, Z' (ASD), for 16d nails (face nailed).**

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

Check for Effects of Full Length Shear Walls on Chord Forces



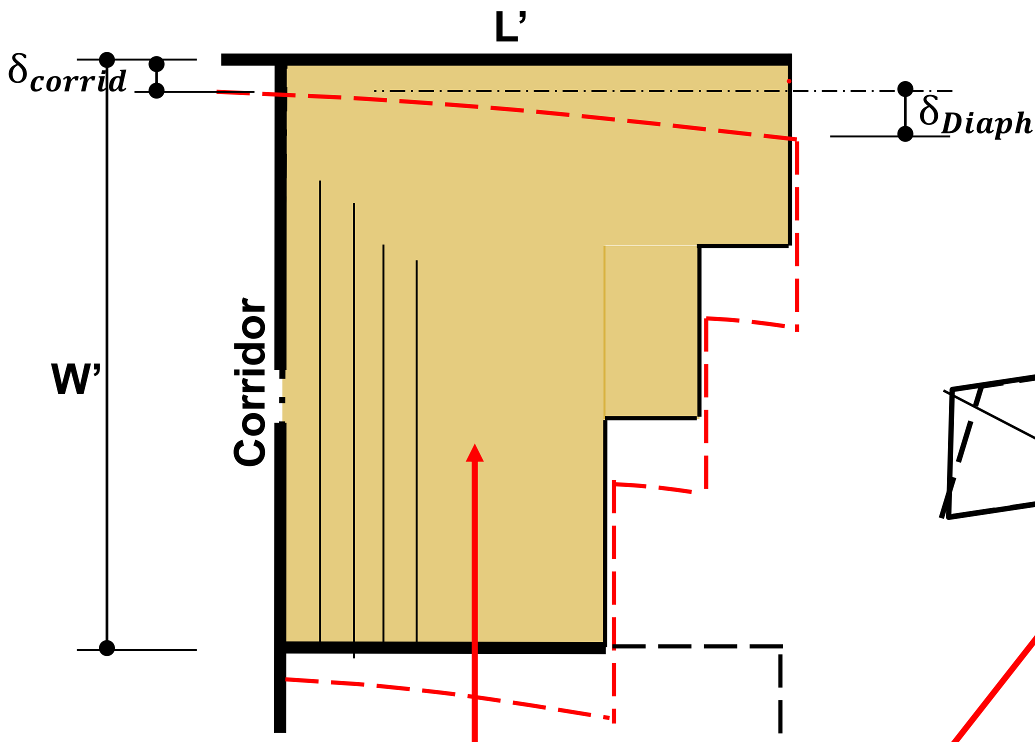
Uniform torsional shears vs. Shear Wall Shears

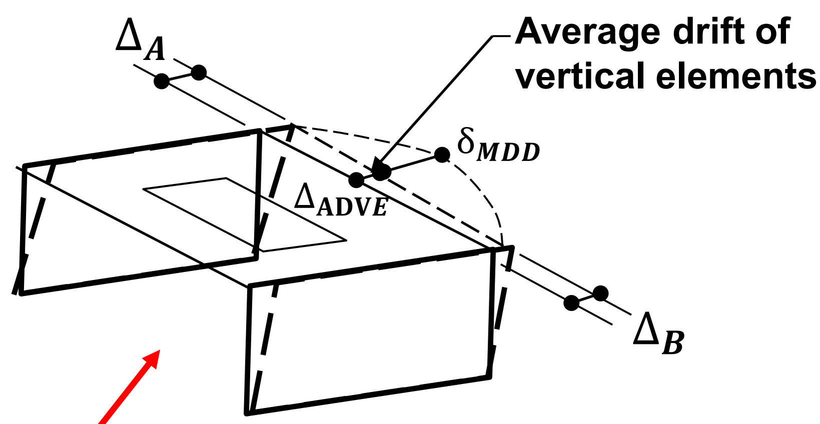


Similar to APA Example

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

Diaphragm Flexibility, ρ=1.0, Ax=1.25



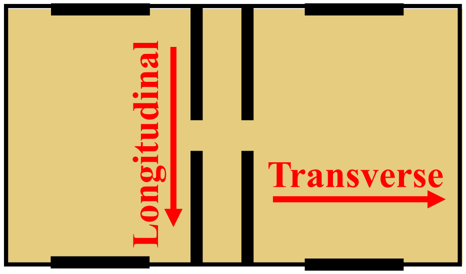


ASCE 7-16 Figure 12.3-1

How does this relate to this?

Analysis Flow

Longitudinal Design

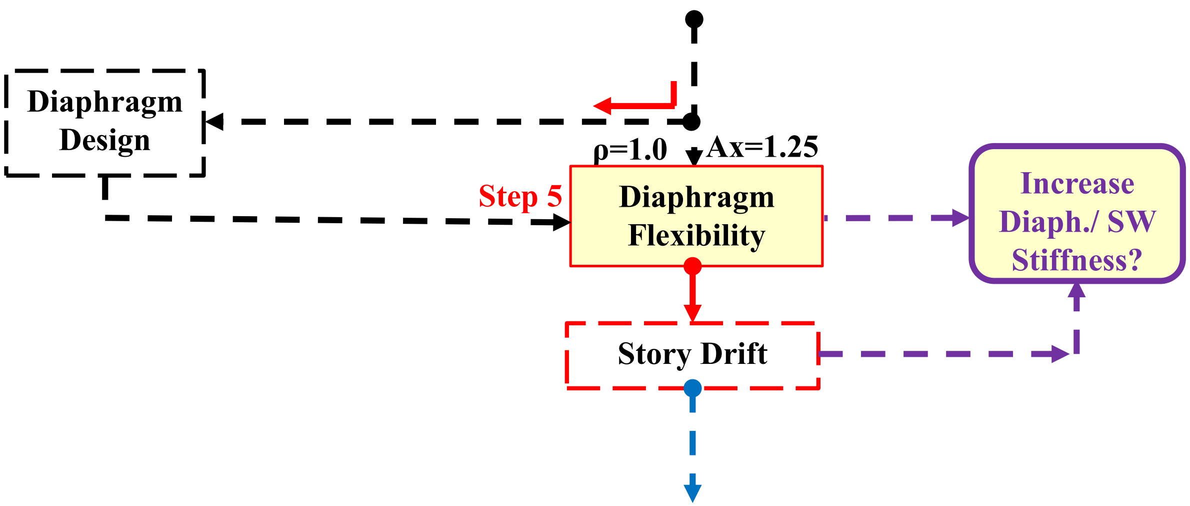


Example Plan

Legend

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

ASD Design | STR Design

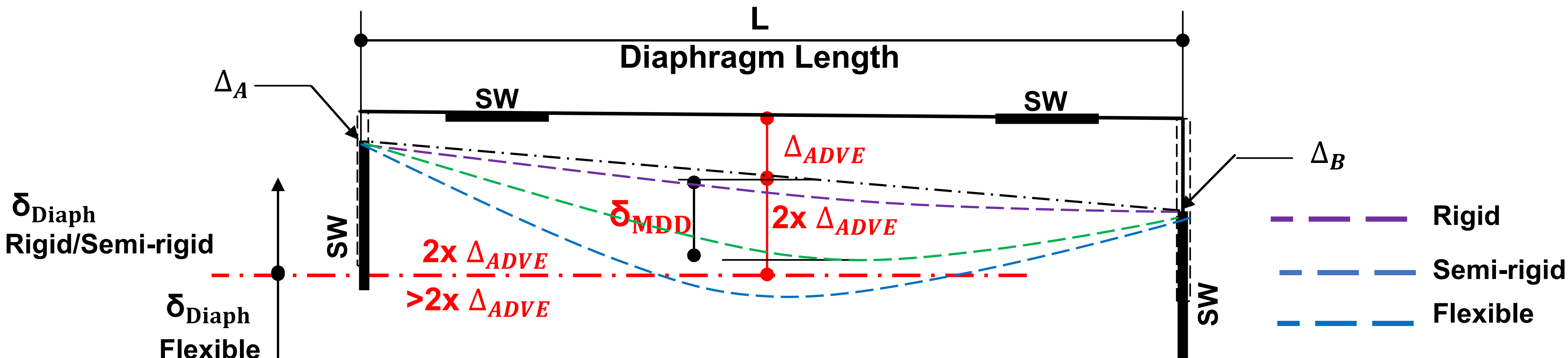


Check Diaphragm Flexibility

Seismic- ρ=1.0, Ax=1.25

Diaphragm Flexibility

- **ASCE 7-16**
  - **12.3.1.1 Flexible Diaphragm Condition.**
    - **Untopped steel decking or wood structural panels**
    - **Permitted to be <u>idealized as flexible</u> 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 <u>idealized as rigid.</u>**
  - **12.3.1.3 Calculated Flexible Diaphragm Condition.**
    - **Diaphragms not satisfying the conditions of Sections 12.3.1.1or 12.3.1.2**
    - **Permitted to be <u>idealized as flexible</u> provided: $\delta_{MDD} > 2\Delta_{ADVE}$.**
- **2018 IBC Section 1604.4:**
  - **A diaphragm is <u>rigid</u> when $\delta_{MDD} \leq 2\Delta_{ADVE}$.**
- **2015 SDPWS 4.2.5 Horizontal Distribution of Shear**
  - **<u>Idealize as rigid</u> when <u>computed</u> $\delta_{MDD} \leq 2\Delta_{ADVE}$**



(a) ASCE 7-16 Figure 12.3-1

Simple Span Diaphragm

Determination of Cantilever Diaphragm Flexibility (Question 3):

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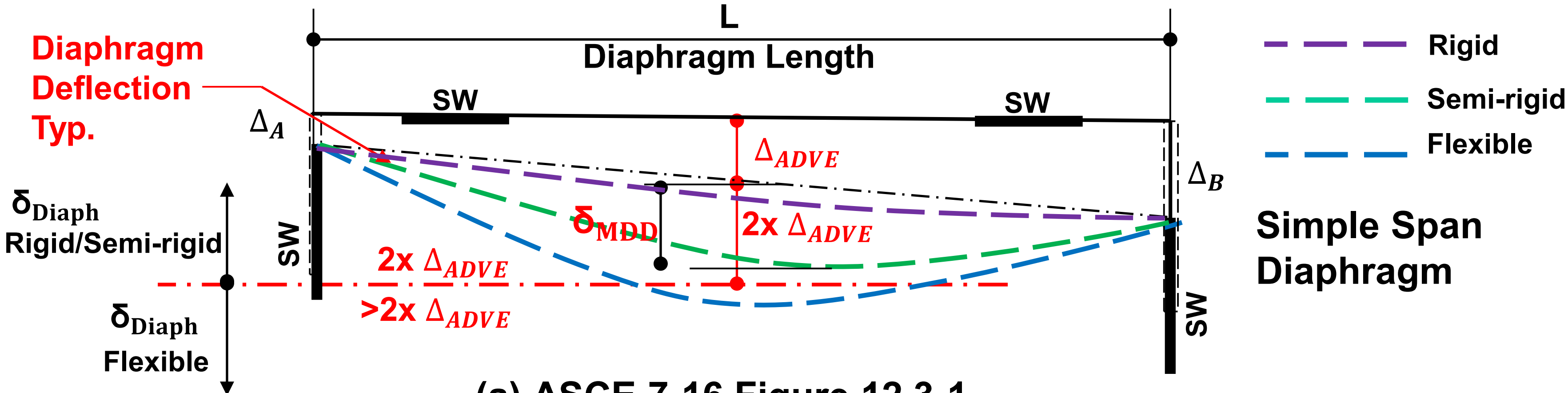
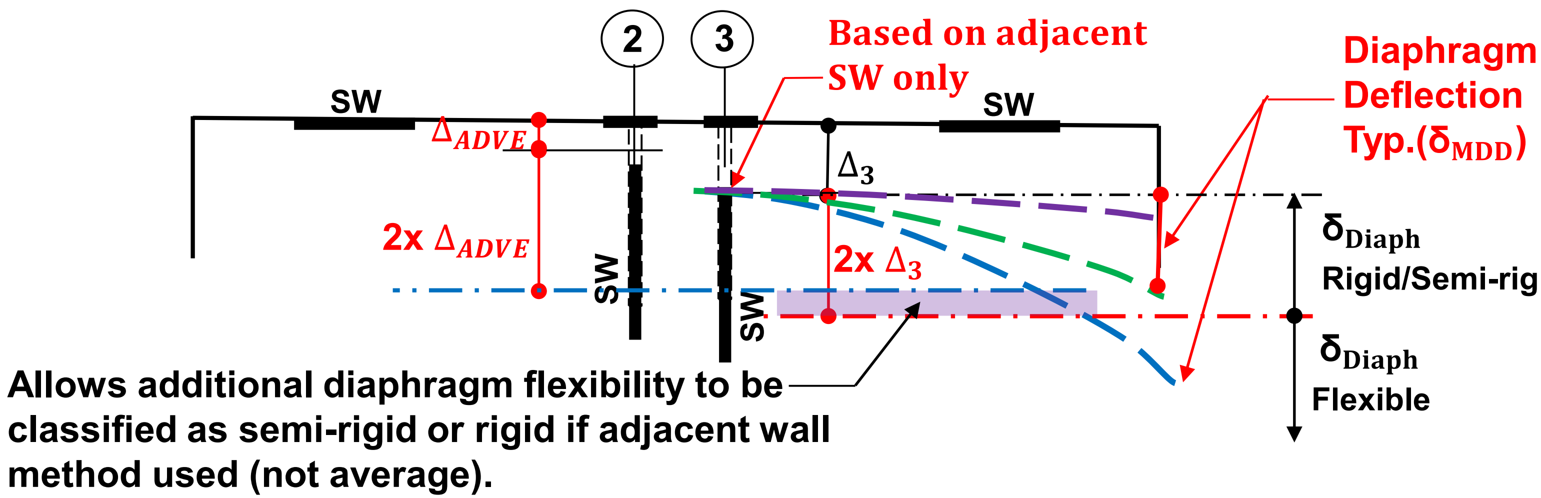
To What Degree, Rigid or Semi-rigid?



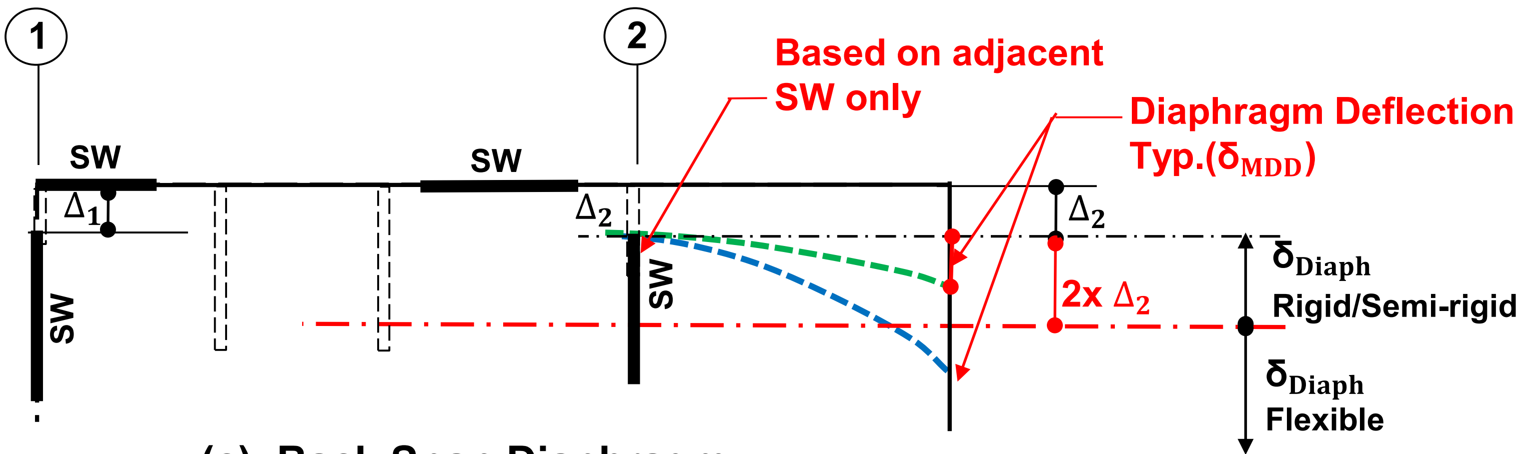
(a) ASCE 7-16 Figure 12.3-1



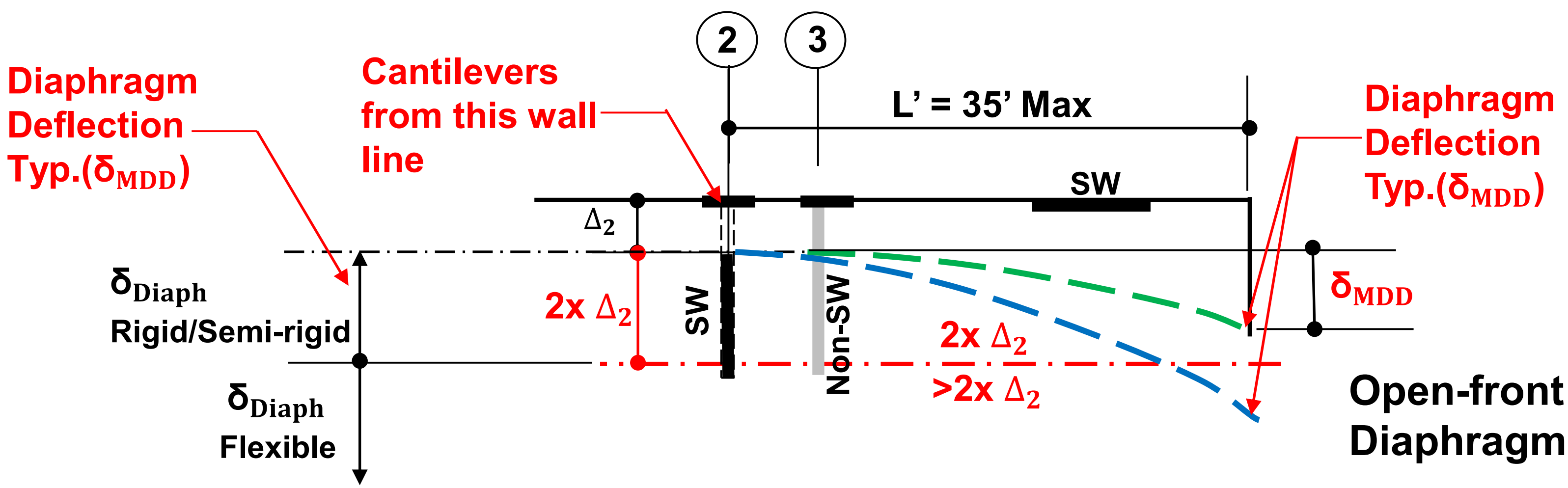
(b) Corridor Walls Only

Preferred Method – Simplifies Check

Can require engineering judgement



(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:

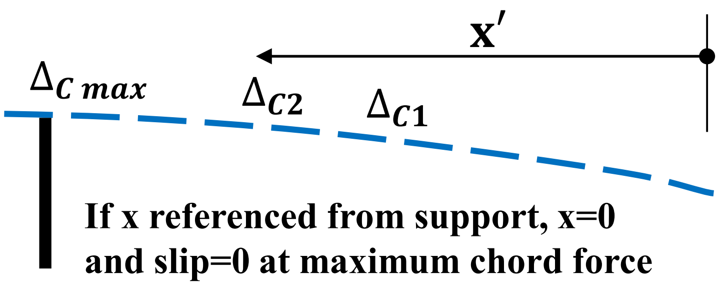
$$\delta_{Diaph\ Conc} = \frac{8vL'^3}{EAW'} + \frac{vL'}{Gvtv} + 0.75\ L'\ e_n + \frac{\Sigma x'\Delta_C}{W'}$$

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.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	kd²	Fv		FT	Fv+FT
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 Walls
lines A & B

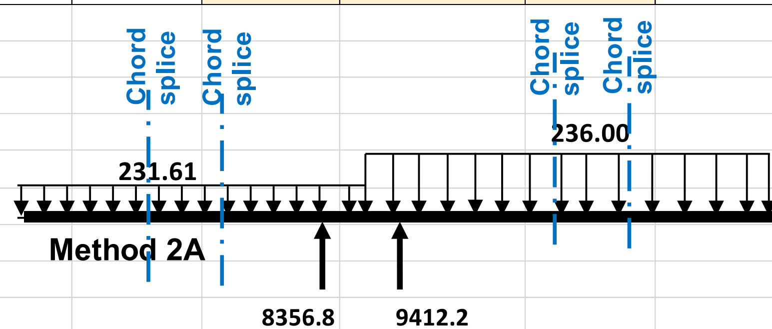
Diaphragm Deflection (STR)

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”

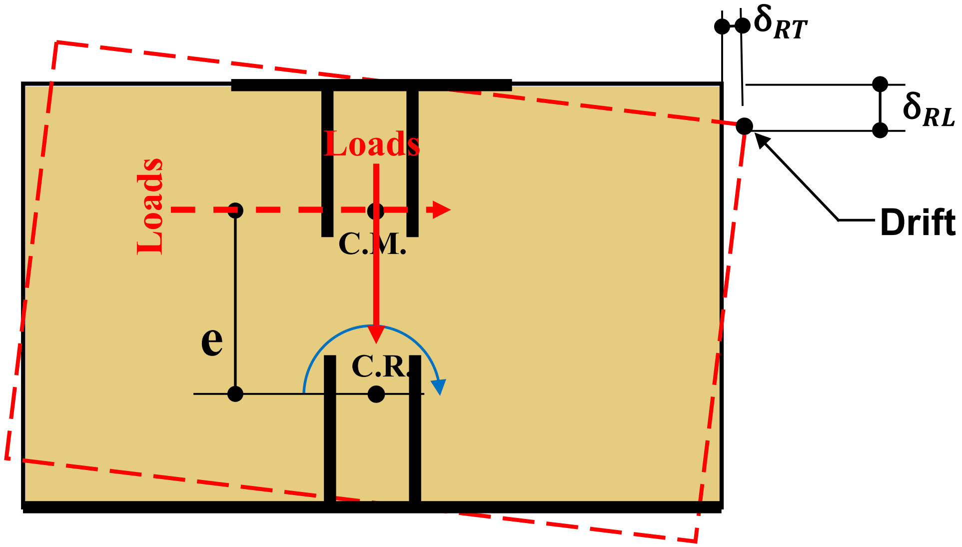
$$2 \text{ x } \Delta_3 = 0.432''$$

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

Diaphragm Flexibility – Wind

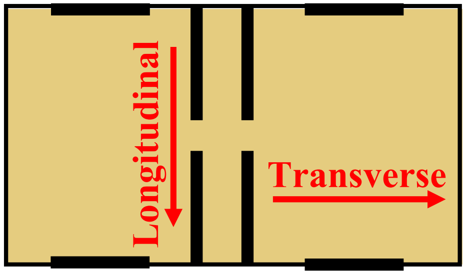
- **ASCE 7-16, Chapter 27, Section 27.5.4-DIAPHRAGM FLEXIBILITY-requires that the structural analysis shall consider the stiffness of diaphragms and vertical elements of the main wind force resisting system (MWFRS).**
- **Section 26.2 - Definitions, DIAPHRAGM, diaphragms constructed of WSP are permitted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

Story Drift, ρ=1.0, Ax=1.25



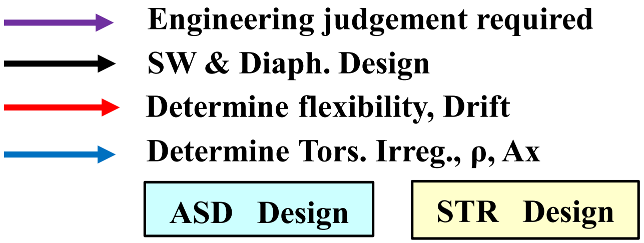
Analysis Flow

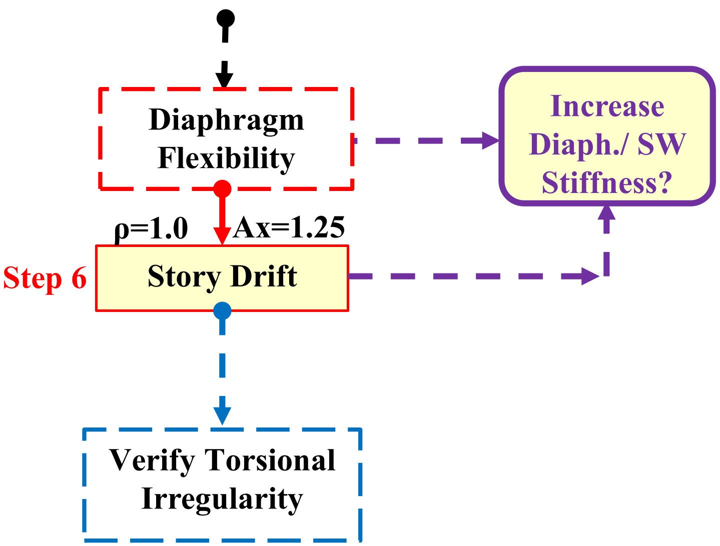
Longitudinal Design



Example Plan

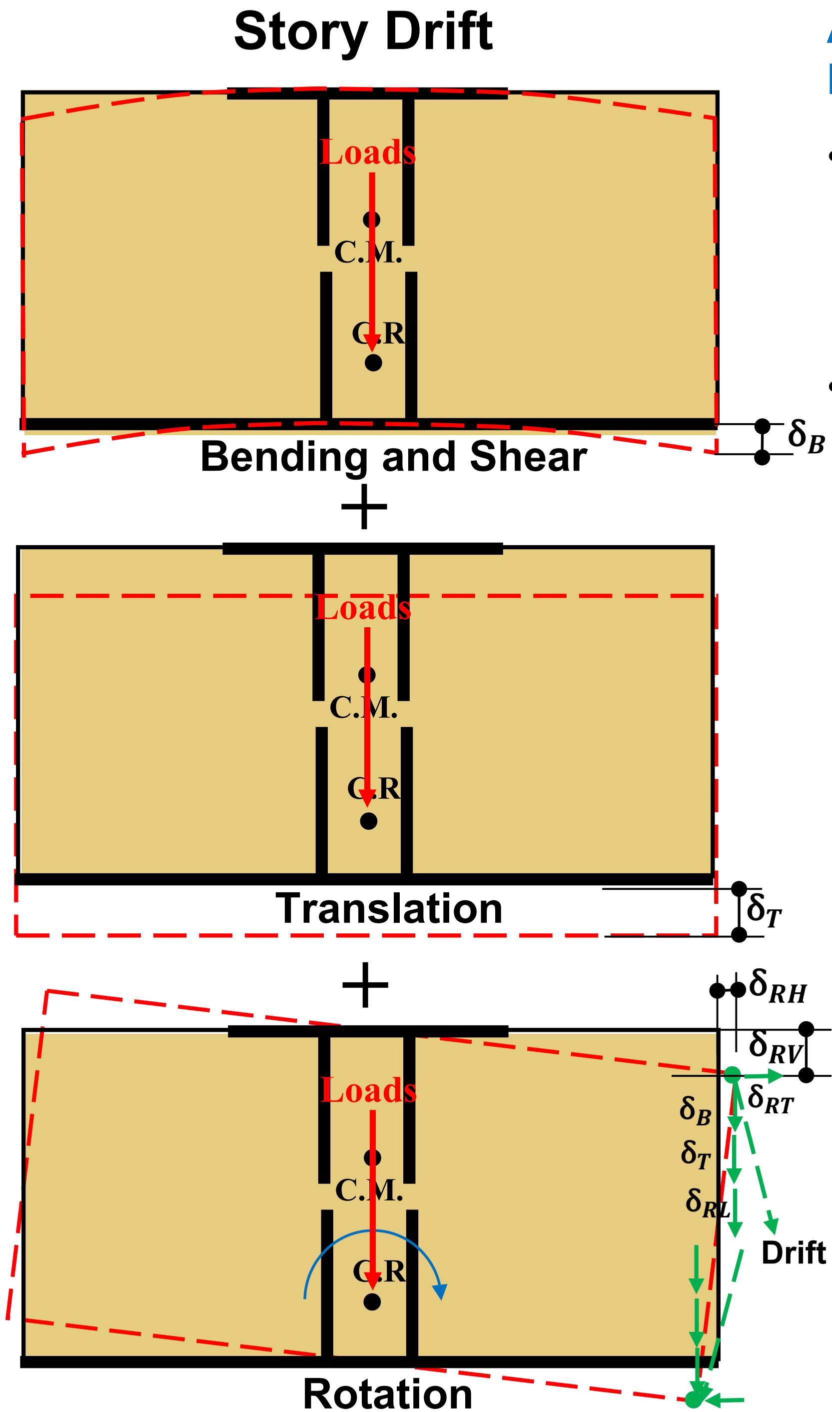
Legend





Check Story Drift

Seismic- ρ=1.0, Ax=1.25



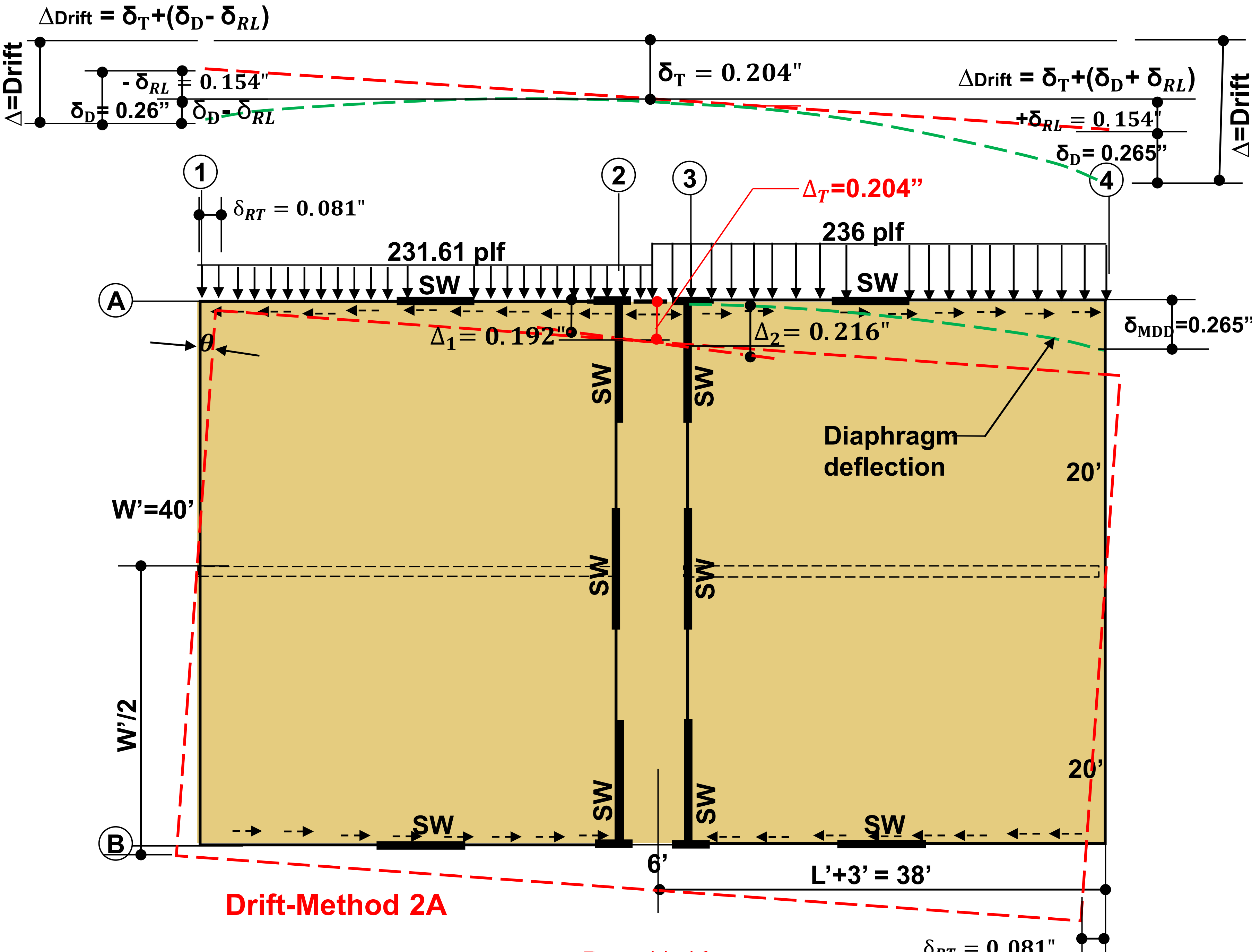
ASCE 7-16 Section 12.8.6-Story Drift Determination Regular structures:

- **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 .**

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



Drift-Method 2A

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Drift-Method 2A 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

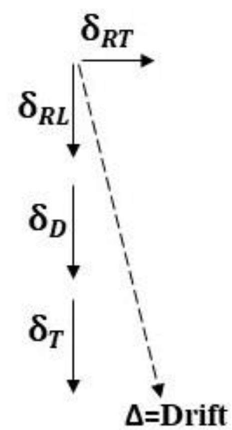
δ_A = 2.031 k / 25.14 k/in = 0.081 in,

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

Δ_{Diaph}= 0.265”

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

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



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

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

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

δ_{RT} = Transverse component of rotation

δ_{RL} = Longitudinal component of rotation

δ_D=Diaphragm displacement

δ_T = Translational displacement

Cd = 4, Ie = 1

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

Table 12.12-1 Allowable Story Drift, Δ_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 exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

- 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.
- $0.025h_{sx}$ 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.

$0.025h_{sx} = 0.025(10)(12) = 3.0" > 2.51"$ ∴ drift O.K.

$0.02h_{sx} = 0.02(10)(12) = 2.4" < 2.51"$ ∴ 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).
- 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

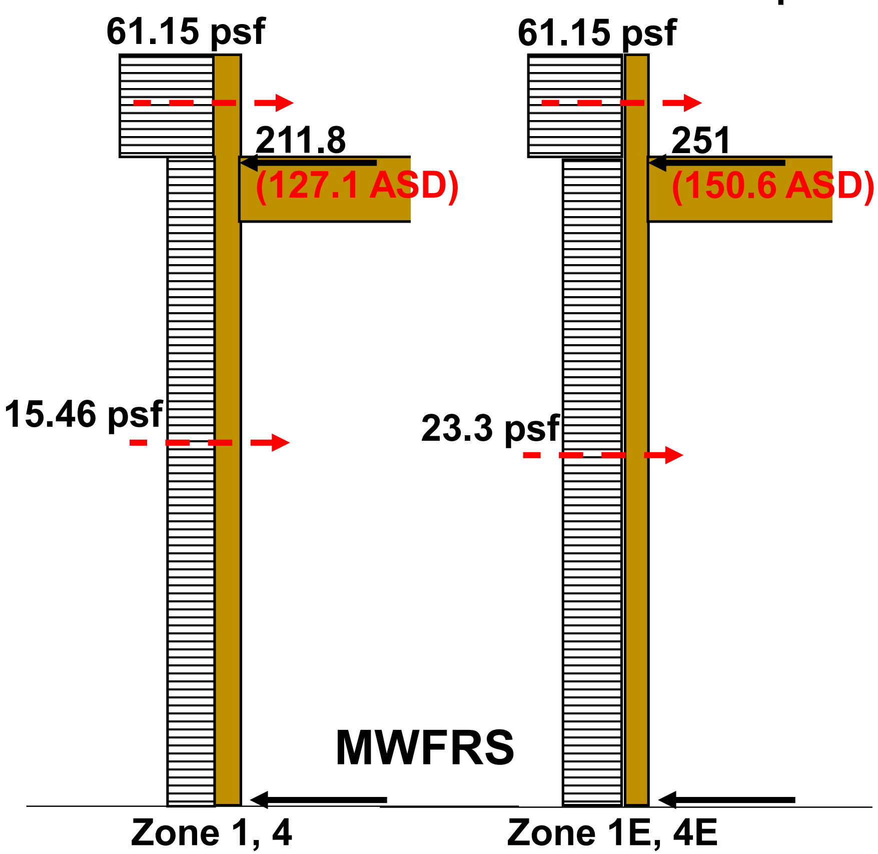
Simplified Procedure Chapter 28, Part 1 Low-rise Buildings, Enclosed

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

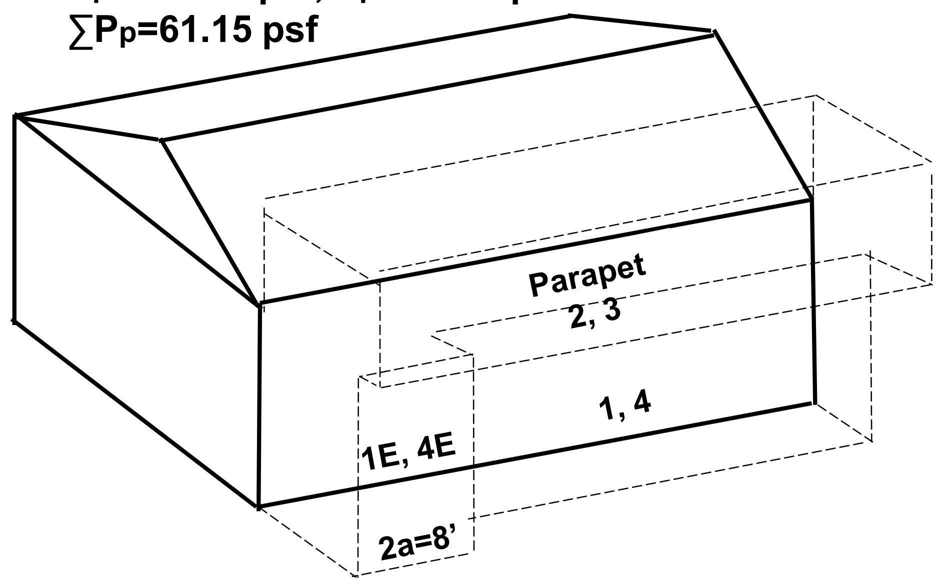
K_z=0.85 @ 12' Top of parapet

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		F_T	Fv+F_T
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

δ_{SW}				
0.112	Rho=	1	2a=	8
0.114	Ax=	1	Net=	23.5
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	kd²	Fv		F_T	Fv+F_T
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 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)

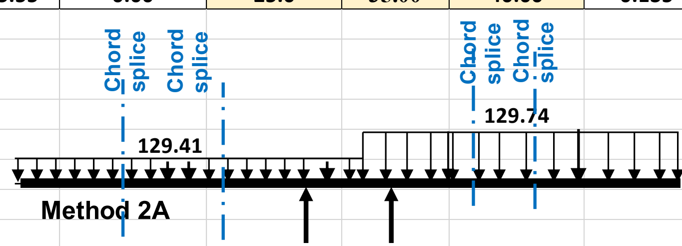
$$\delta_{Diaph\,Unif} = \frac{3vL'^3}{EAW'} + \frac{0.5v_{max}L'}{1000G_a} + \frac{\Sigma\Delta_c X_c}{W'}$$

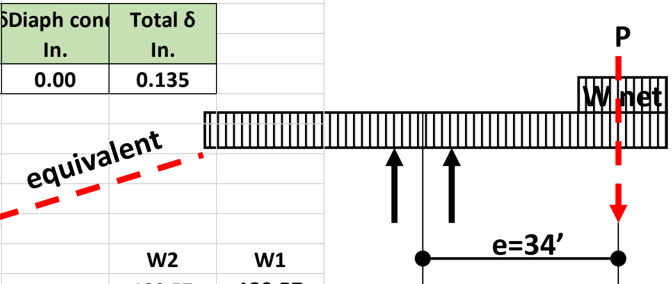
Rt. Cantilever

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





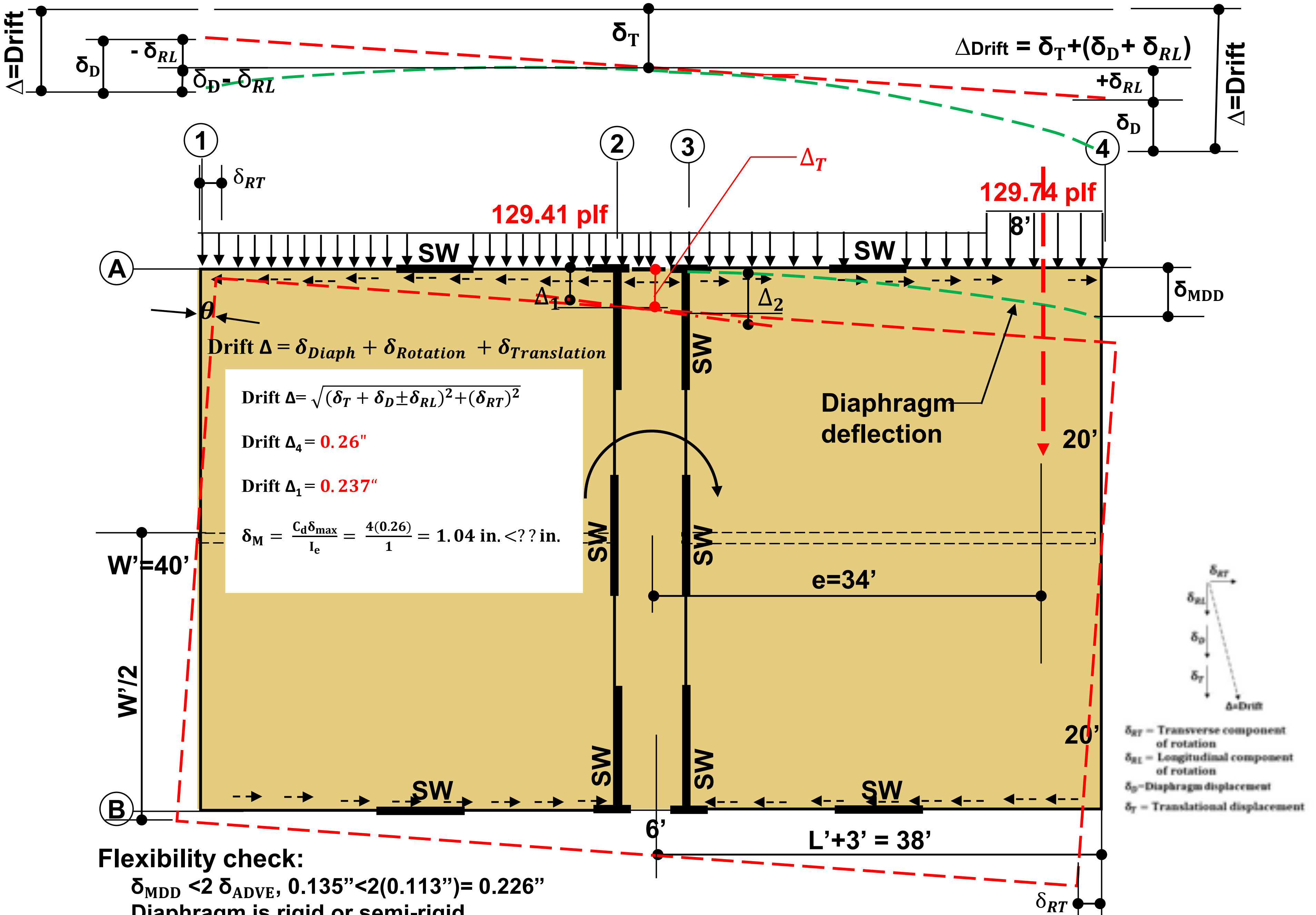
W2	W1
129.57	129.57
0.17	-0.17
129.74	129.41

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



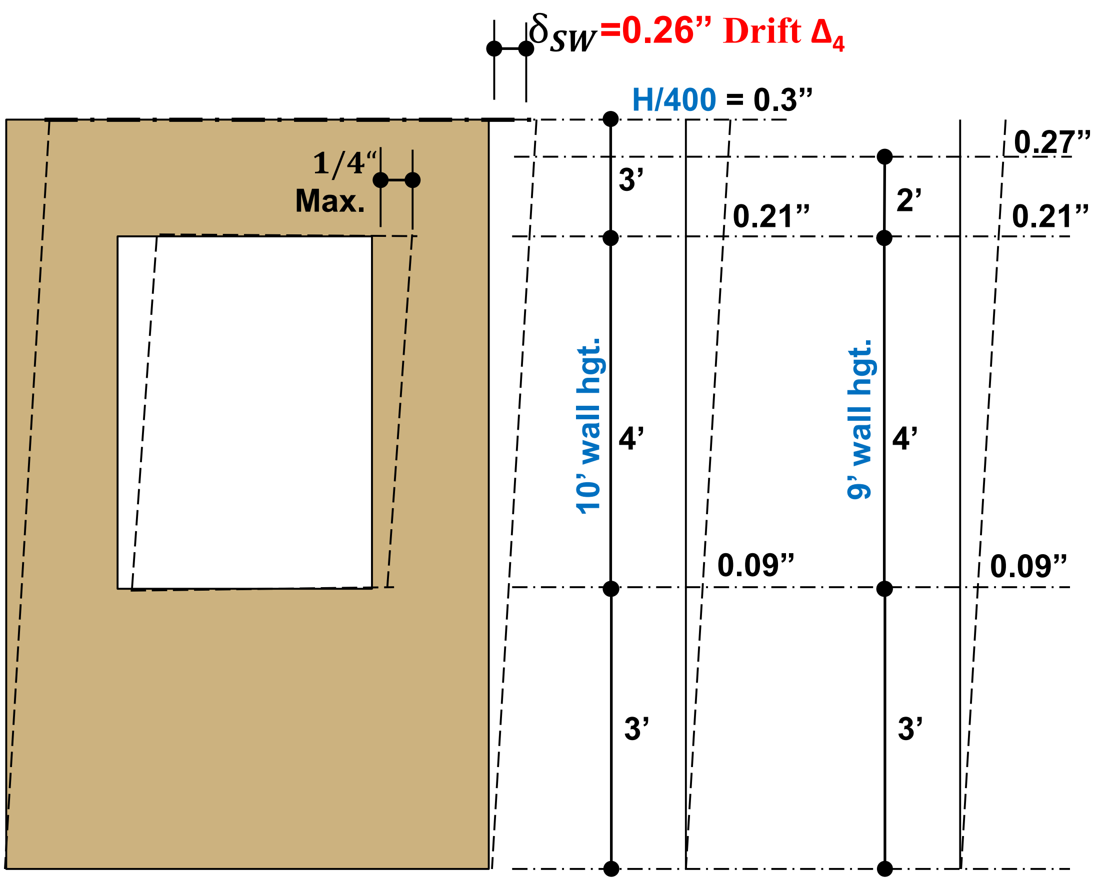
Flexibility check:

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

Diaphragm is rigid or semi-rigid

Allowable Drift Wind? H/600, H/400, 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
Drift Δ_4 =0.26”<0.3”
∴ drift OK

Maximum displacement at top of window at allow defl.=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.

Lunch

Have you had enough?

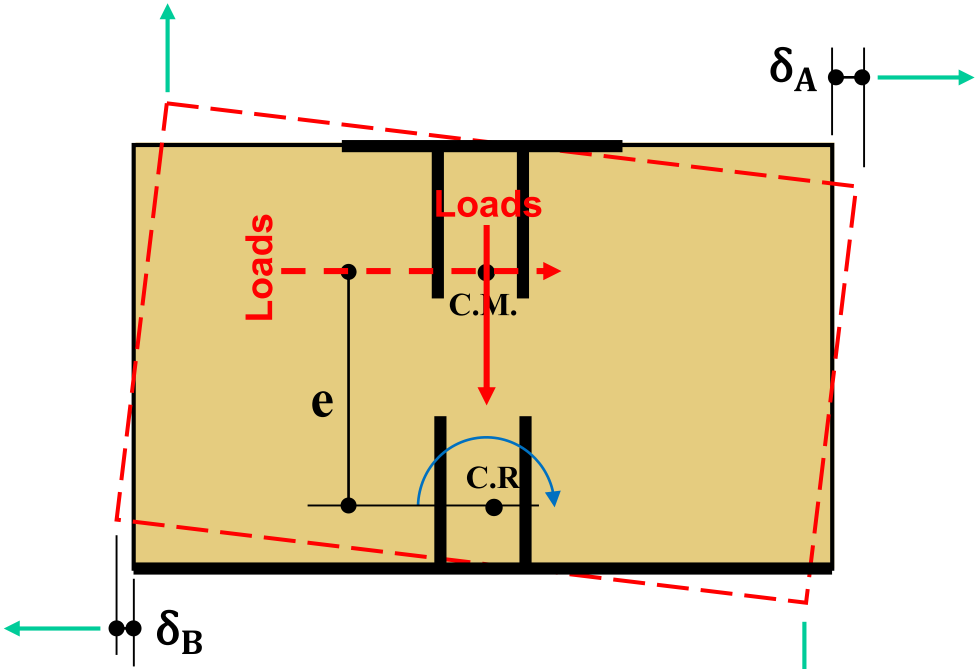
Deer in headlights

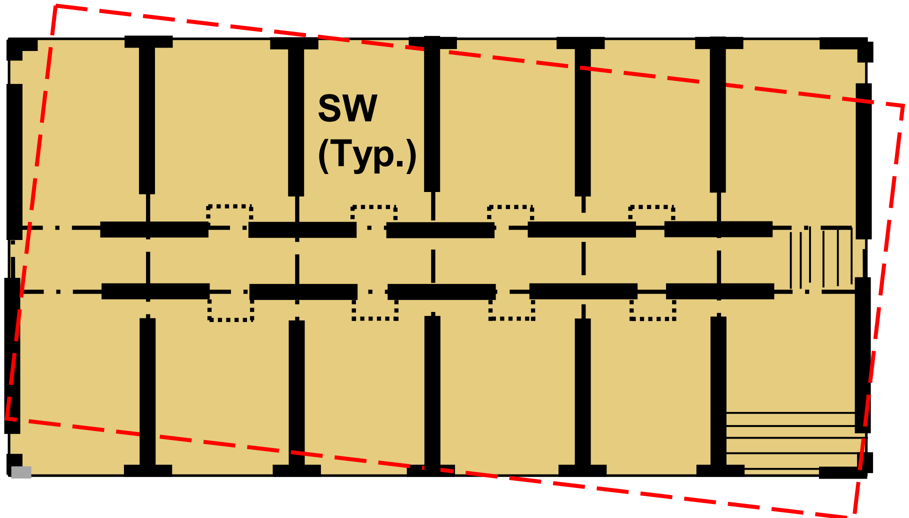
Part 4 Content

Part 4-Design Example (cont.):

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

Torsional Irregularities



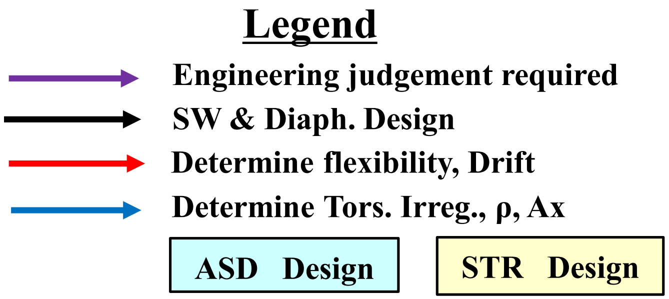


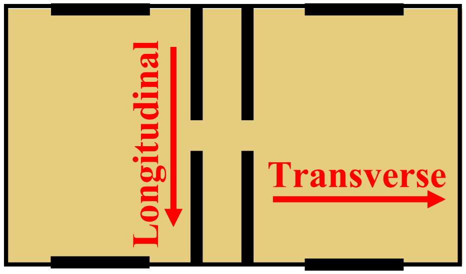
Torsional Irregularity?

Typical Floor Plan

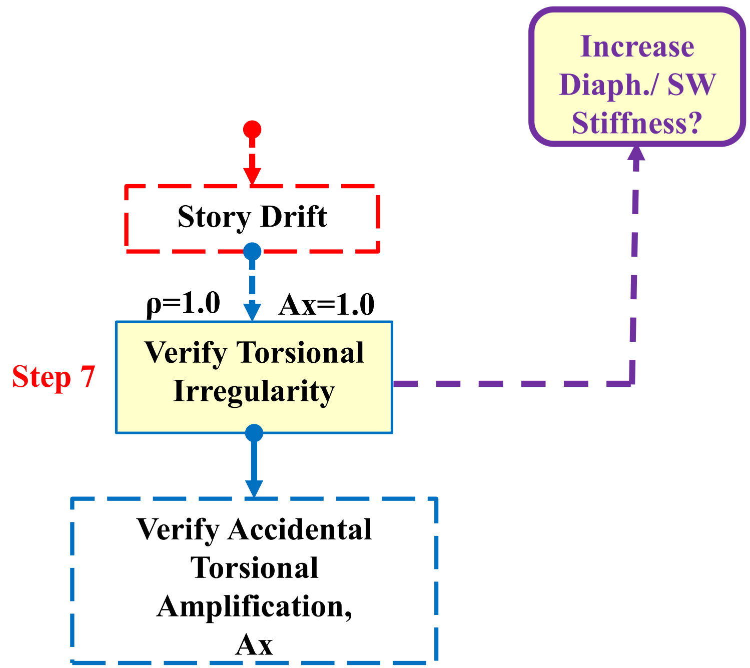
Analysis Flow

Longitudinal Design





Example Plan



Verify Torsional Irregularity

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

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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.

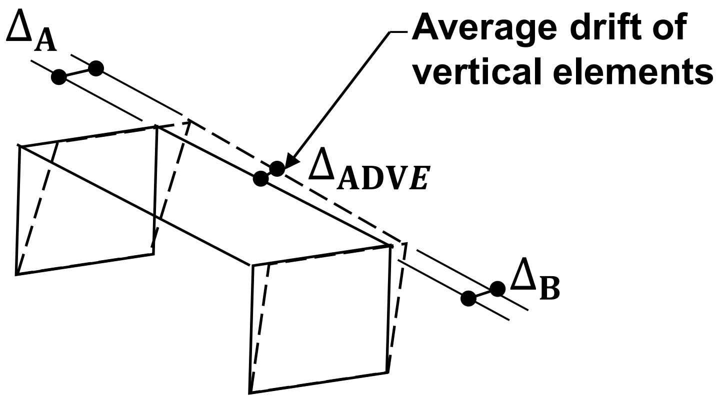
Torsional Irregularity Type 1a – seismic - Maximum story drift, Δ_{MAX}, (including accidental torsion with A_X=1.0), > 1.2x Δ_{ADVE}

- **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, Δ_{MAX} > 1.4 x Δ_{ADVE}

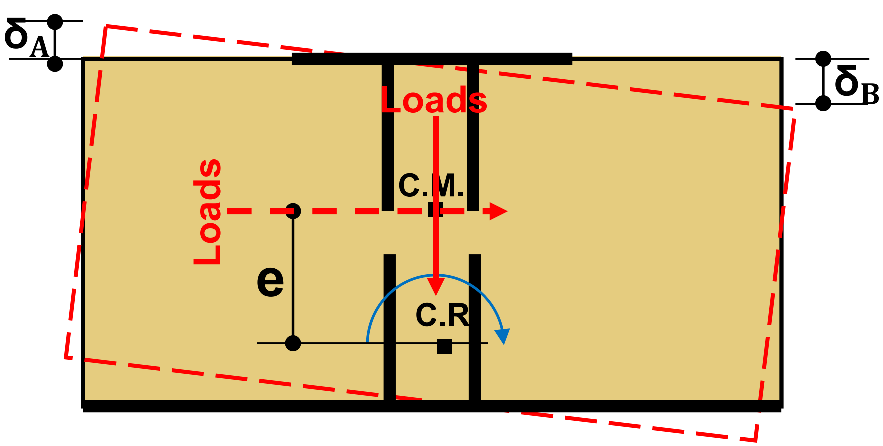
- **An extreme torsional irregularity Type 1b is allowed in structures assigned to Seismic Design Categories B, C, and D, but not in SDC E, or F.**

ASCE 7 Triggers



Δ_{MAX} >1.2x Δ_{ADVE}

ASCE 7-16 Requirements Type 1a Horizontal Irregularity



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

1a. Torsional Irregularity Δ_{MAX} >1.2x Δ_{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 Δ_{MAX} >1.4x Δ_{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. ρ=1.0, Ax=1.0

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		-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
Σ	87.09	50.27			J=	20893.23	17769			

Corridor Walls

Walls at Grid lines A & B

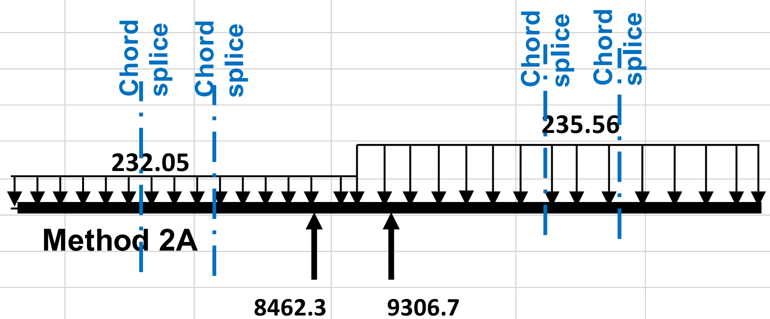
Diaphragm Deflection (STR) ρ=1.0, Ax=1.0

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.
	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

Iincludes effects of sw's along chord line

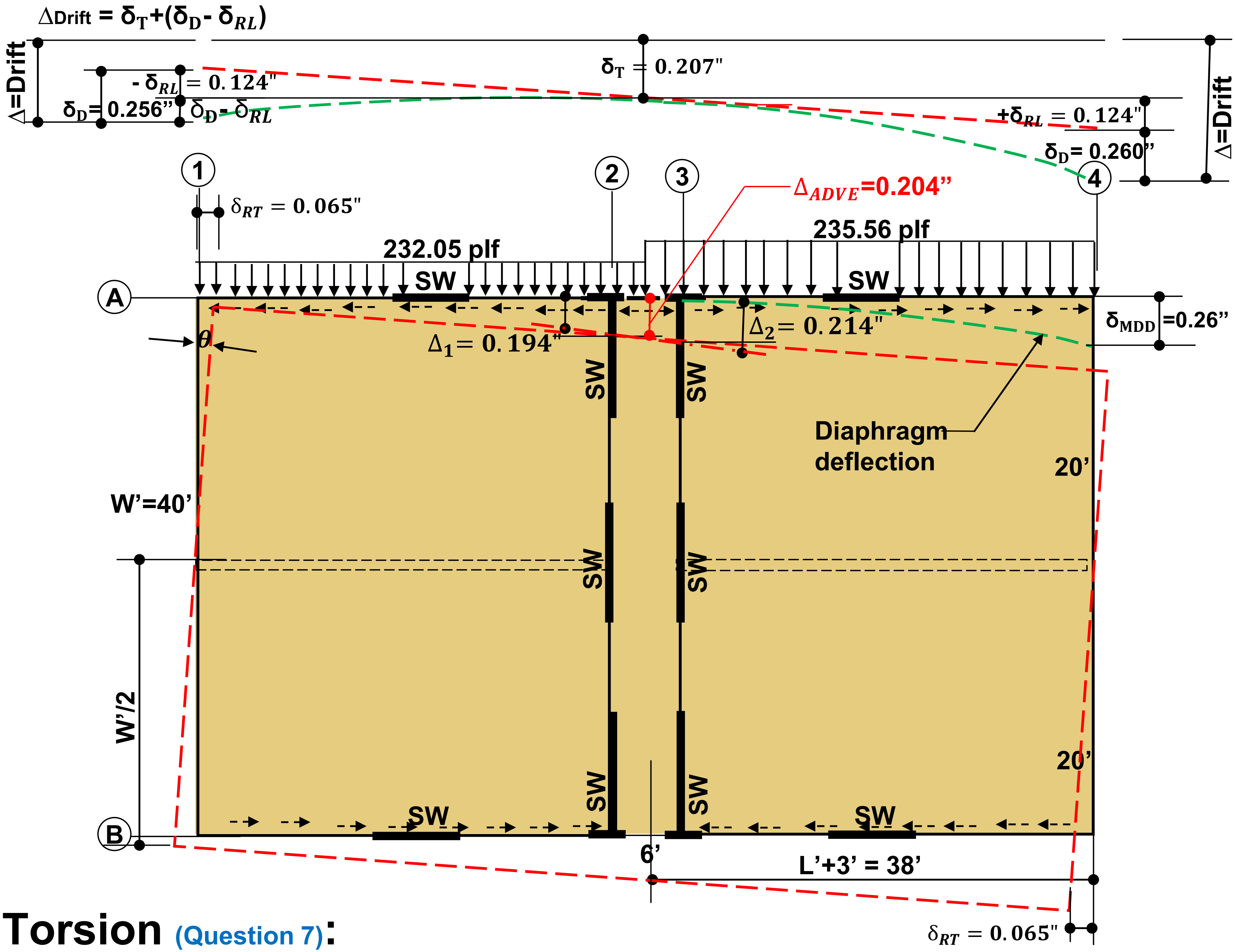


Diaphragm Deflection (STR)

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									

Torsional Irregularity Check-Method 2A



Torsion (Question 7):

Check for Torsional Irregularity Type 1a - ρ=1.0, Ax=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" =** δ_{RT} **Transverse displacement at Lines A and B from rigid diaphragm rotation**

$\delta_{RL} = \frac{2\delta_{SWA,B}(L'+3')}{W'}$ **=0.124" Vertical component of rotation**

Diaphragm deflections:

$\delta_{D,1}$**=0.256"**

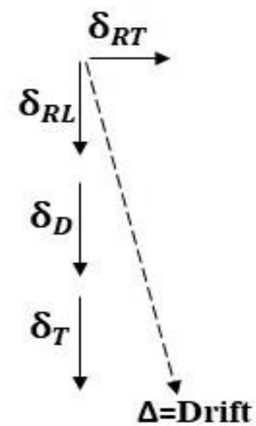
$\delta_{D,4}$**=0.260"**

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

Drift $\boldsymbol{\Delta_4} = \sqrt{(\mathbf{0.204 + 0.260 + 0.124})^2 + (\mathbf{0.065})^2} = \mathbf{0.592}''$

Drift $\boldsymbol{\Delta_1} = \sqrt{(\mathbf{0.204 + 0.256 - 0.124})^2 + (\mathbf{0.065})^2} = \mathbf{0.342}''$

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



δ_{RT} = **Transverse component of rotation**

δ_{RL} = **Longitudinal component of rotation**

δ_D=**Diaphragm displacement**

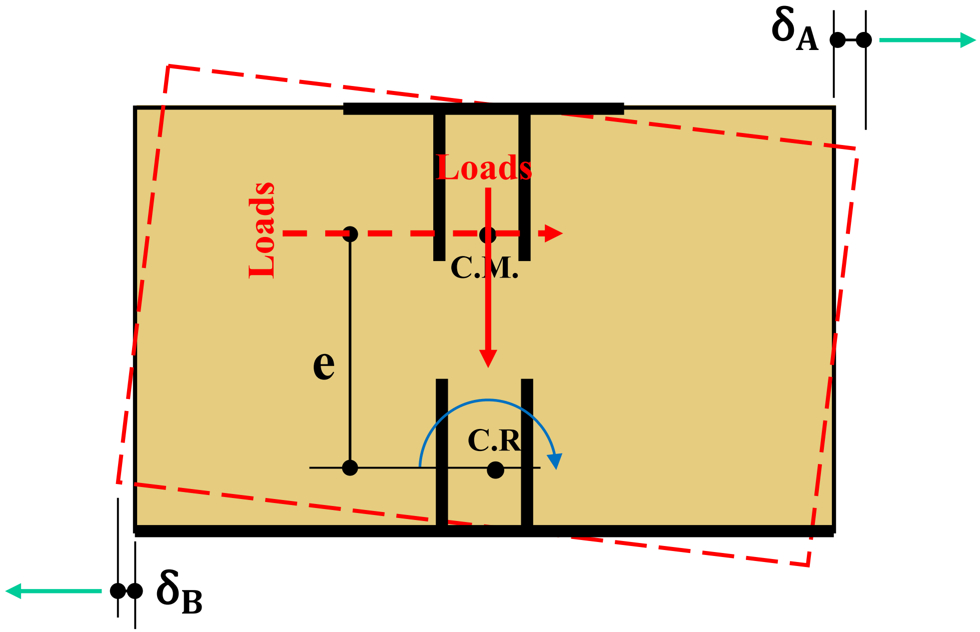
δ_T = **Translational displacement**

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

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

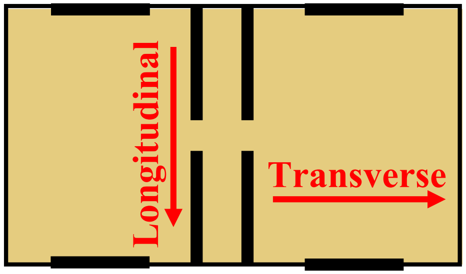
Amplification of Accidental Torsion

Seismic- ρ=1.0, Ax=1.0



Analysis Flow

Longitudinal Design



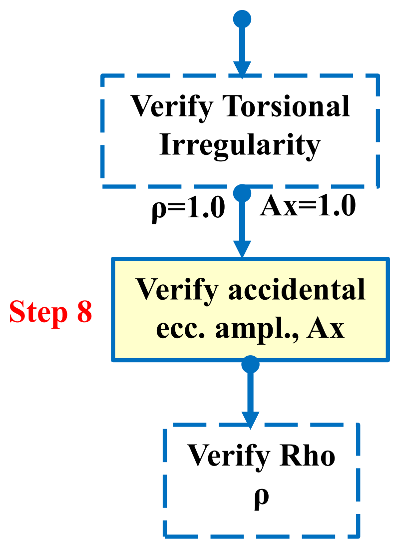
Example Plan

Legend

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

ASD Design

STR Design



Verify Amplification of Accidental Torsion, Ax

Seismic- ρ=1.0, Ax=1.0

Page 54

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 (Ax) as illustrated in Fig. 12.8-1 and determined from the following equation:

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

Where

δ_{max} =maximum displacement at level x computed assuming Ax = 1

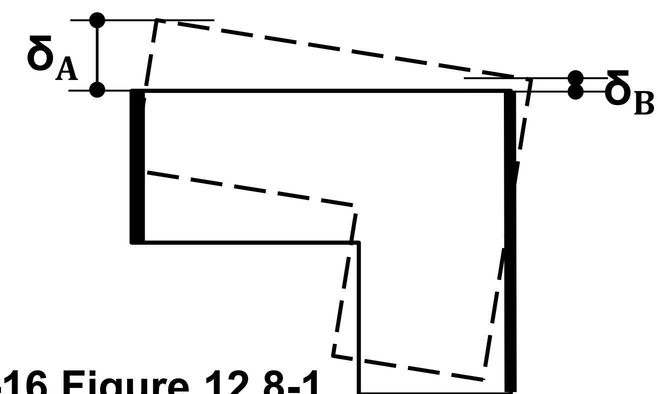
δ_{avg} =average of the displacements at the extreme points of the structure at level x computed assuming Ax = 1.

M_{ta} =accidental torsional moment

From torsion section:

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

∴ Can recalculate if desired.

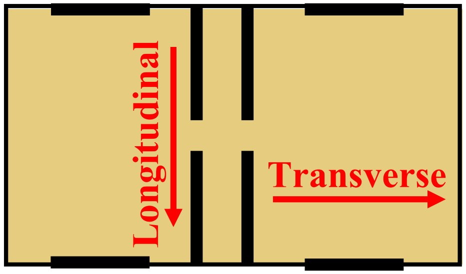


ASCE 7-16 Figure 12.8-1
Amplification of accidental torsion

ASCE 7-10 (1st printing) 12.8.4.1 Inherent Torsion **Exception below is not in 3rd 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



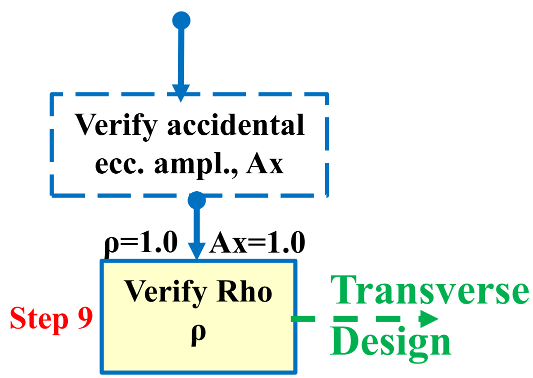
Example Plan

Legend

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

ASD Design

STR Design



Verify Redundancy, ρ

Seismic- ρ=1.0, Ax=1.0

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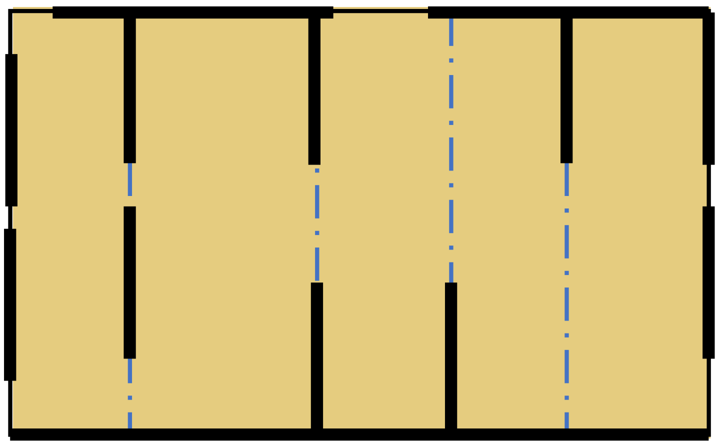
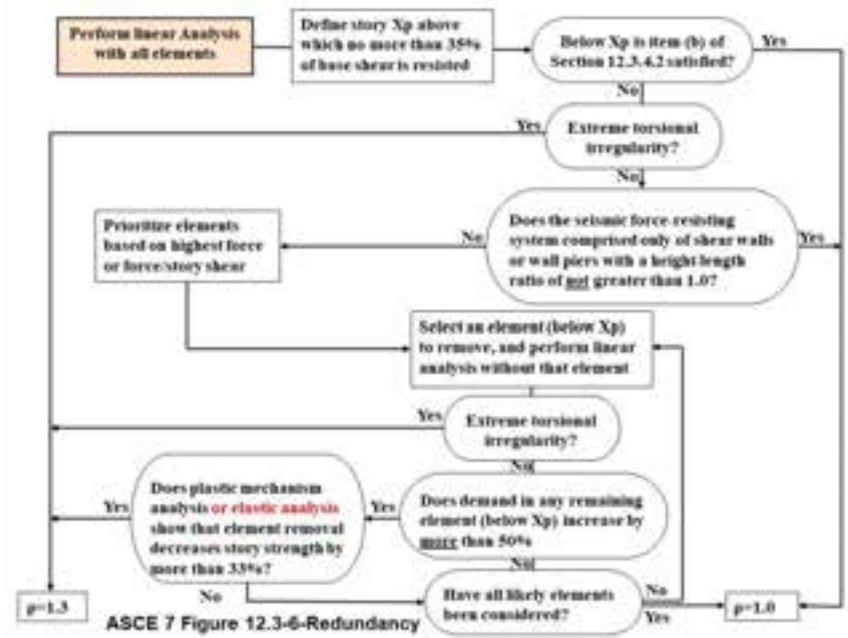
Redundancy

Seismic- ρ=1.0, Ax=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 ρ is 1.0. The value of ρ 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.**

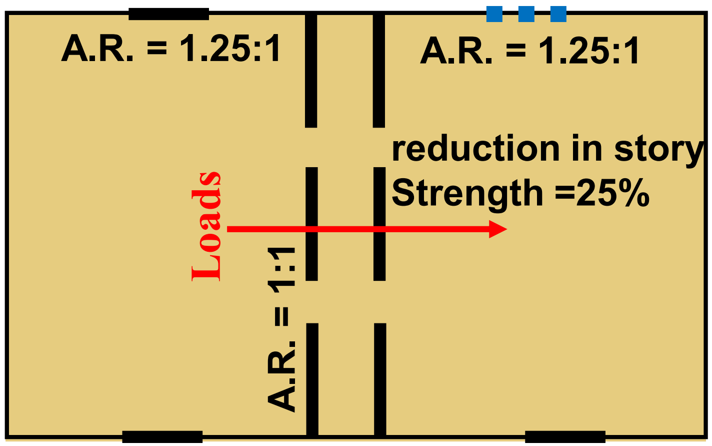
7. **Diaphragm loads, Fpx, determined using Eq. 12.10-1, including min. & max. values.**

12.3.4.2 Redundancy Factor, ρ, for Seismic Design Categories D through F.

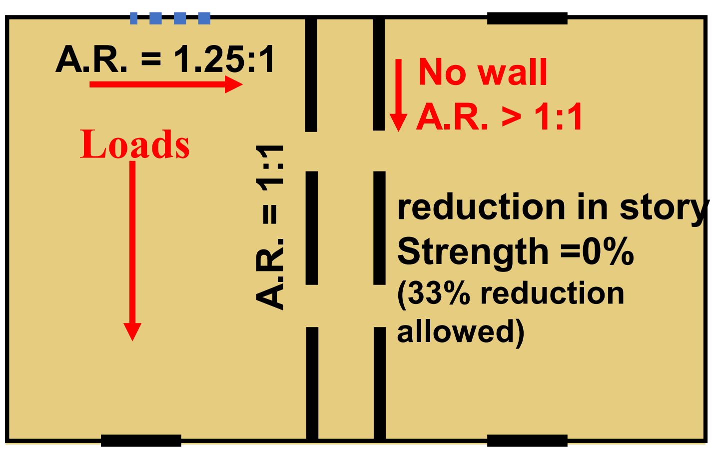
- **For structures assigned to Seismic Design Category D and having extreme torsional irregularity as defined in Table 12.3-1, Type 1b, ρ shall equal 1.3.**

- **For other structures assigned to Seismic Design Category D and for structures assigned to Seismic Design Categories E or F, ρ shall equal 1.3 unless one of the following two conditions (a. or b.) is met, whereby ρ is permitted to be taken as 1.0.**

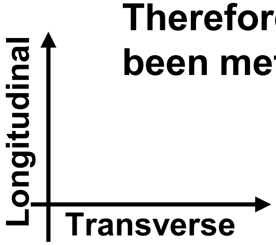
Let's check condition b. first



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



Therefore condition "a" has been met and ρ=1.0.



b. Structures that are <u>regular in plan</u> at all levels ρ=1.0 provided:

- SFRS consist of at least two bays of <u>perimeter</u> SFRS framing on <u>each side of the structure in each orthogonal direction</u> at each story resisting more than 35% of the base shear.
- The number of bays for a shear wall = Lsw / hsx, or 2Lsw / hsx, for light-frame construction.

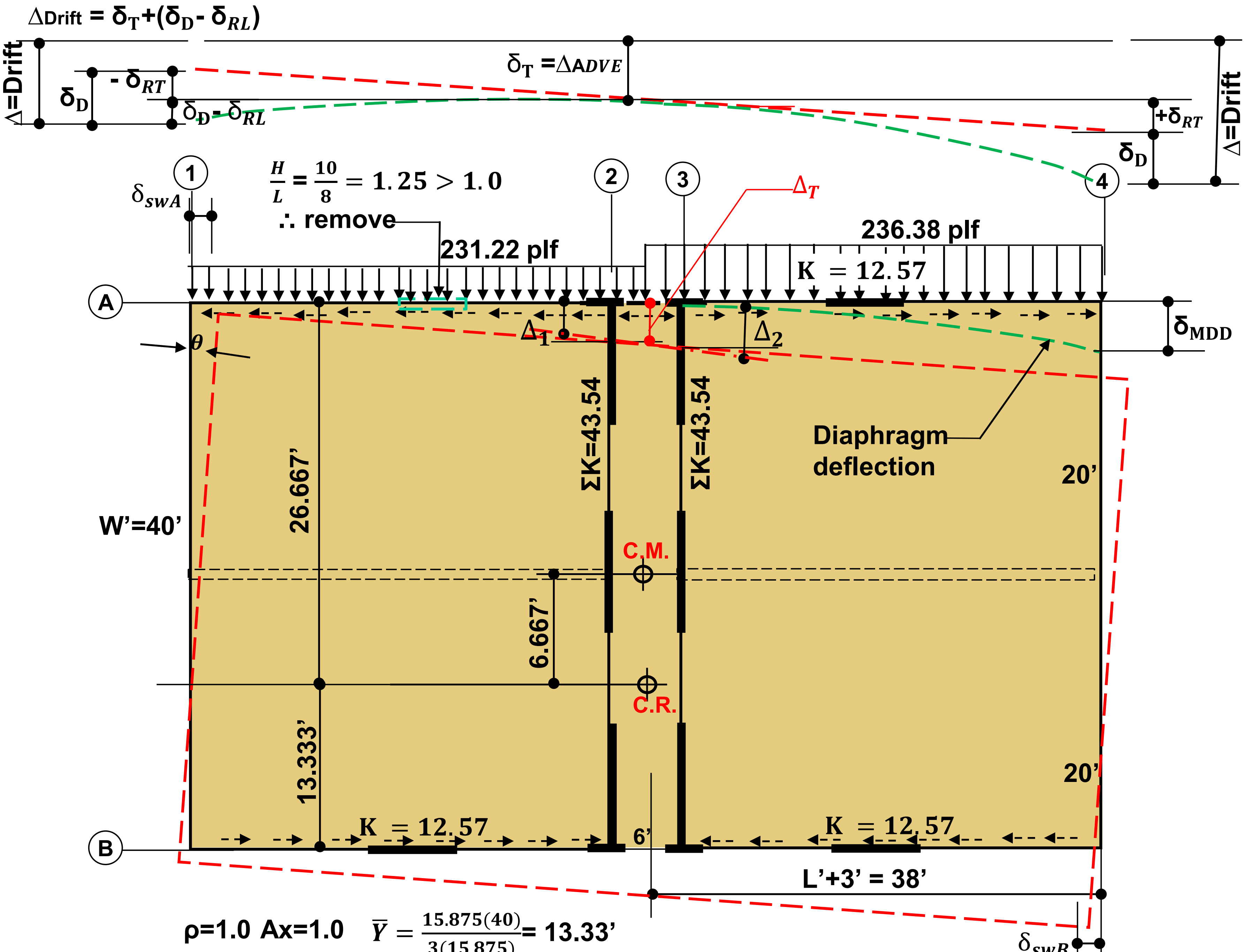
Although the plan is regular, in the longitudinal direction, there are no SFRS walls at all exterior wall lines. Therefore, the structure does not comply with condition "b", and condition "a" must be met.

Condition a.

Each story resisting more than 35% of the base shear in the direction of interest shall comply with Table 12.3-3.

<u>Table 12.3-3.</u>

- Removing one wall segment with A.R. > 1:1 will not result in reduction in story strength > 33% limit.
- Removing 1 wall <u>within</u> any story will not result in extreme torsional irregularity, Type 1b.



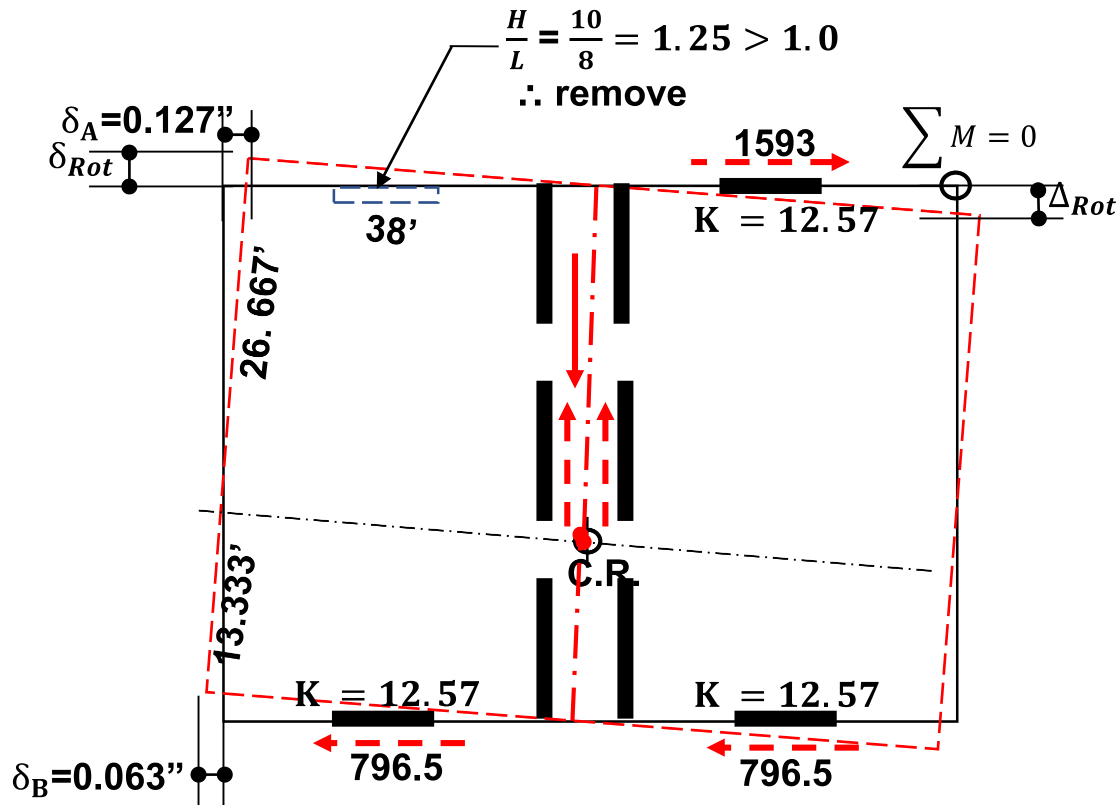
ΔDrift = δT+(δD- δRL)
Δ=Drift
δD
- δRT
δD - δRL
δT =ΔADVE
+δRT
δD
Δ=Drift
1
2
3
4
δswA
H/L = 10/8 = 1.25 > 1.0
∴ remove
231.22 plf
236.38 plf
ΔT
K = 12.57
A
δMDD
Δ1
Δ2
θ
ΣK=43.54
ΣK=43.54
Diaphragm deflection
20'
W'=40'
26.667'
C.M.
6.667'
C.R.
13.333'
20'
B
K = 12.57
K = 12.57
6'
L'+3' = 38'
δswB
ρ=1.0 Ax=1.0
Ȳ = 15.875(40) / 3(15.875) = 13.33'

Redundancy Study

Spreadsheet results

- δ_A= 0.127”
- δ_B= 0.063”
- δ_2= 0.190”
- δ_3= 0.218”
- Δ_{Diaph} L= 0.256”
- Δ_{Diaph} R= 0.260”

	Total
F_A	1595
F_B	1595
F_2	8263
F_3	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”, ∴ Horizontal torsional irregularity Type 1b does not exist in this direction and ρ = 1.0

Shear wall Deflection

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

Shear wall Nominal Stiffness

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

Struts and Collectors-Seismic

Struts / collectors and their connections shall be designed in accordance with ASCE 7-16 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**

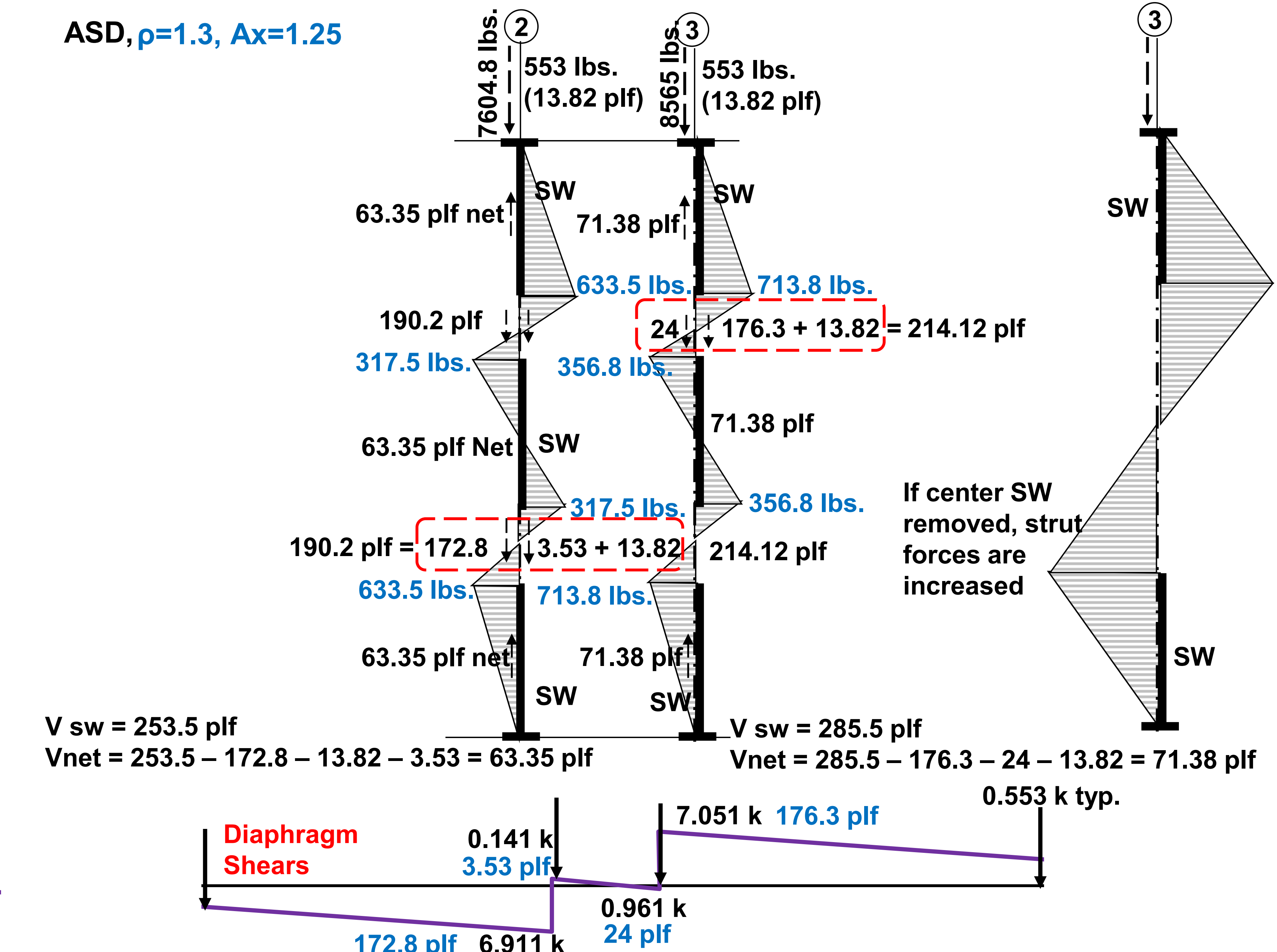
$F_{px\,min} = 0.2 S_{DS} I_e w_{px}$ **- Lower bound seismic diaphragm design forces determined by Eq. 12.10-2 (Fpx_{min}) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength)-do not require the over-strength factor.**

and

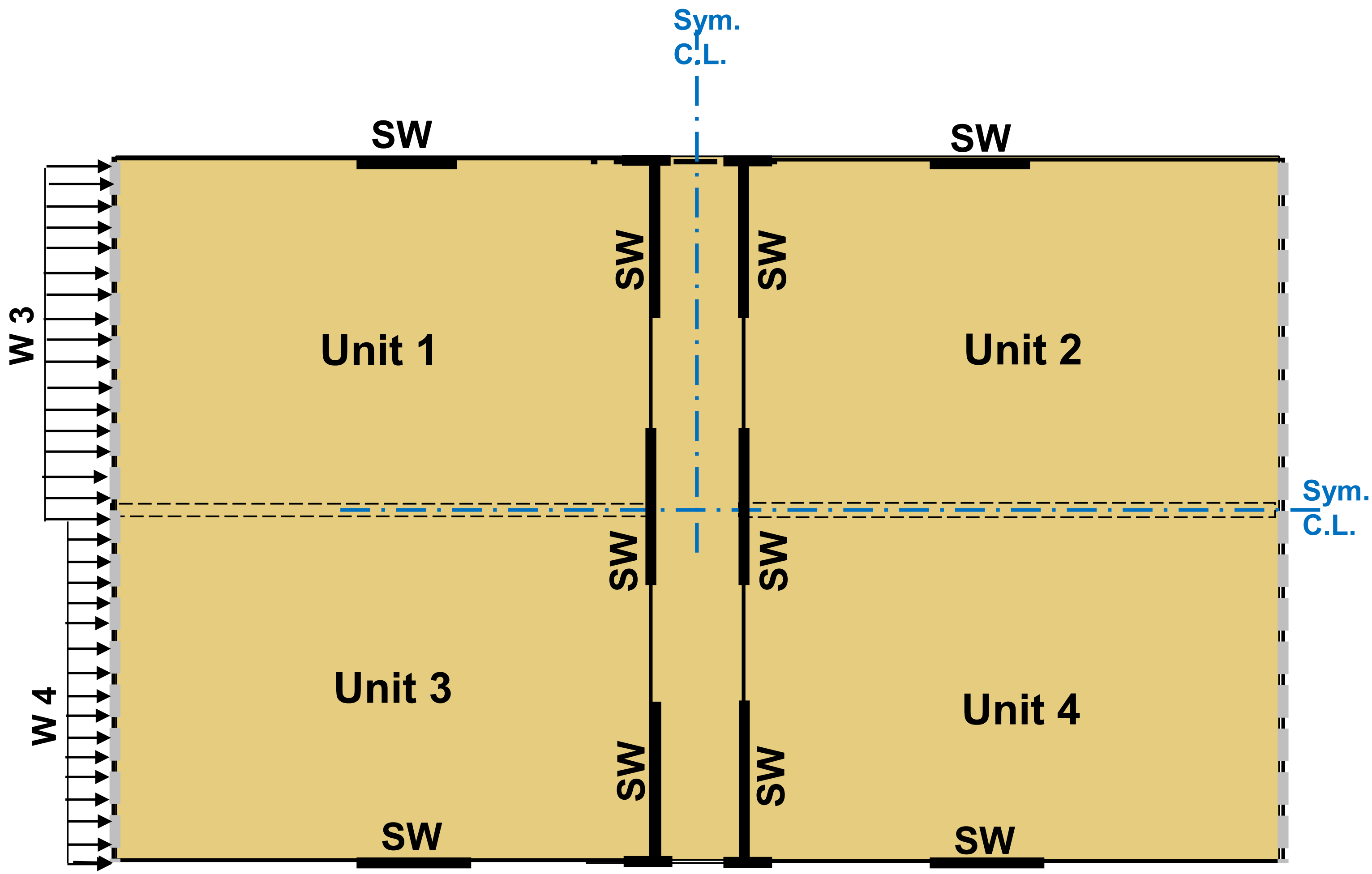
$F_{px\,max} = 0.4 S_{DS} I_e w_{px}$**- Upper bound seismic diaphragm design forces determined by Eq. 12.10-2 (Fpx_{max}) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength)-do not require the over-strength factor.**

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}).**

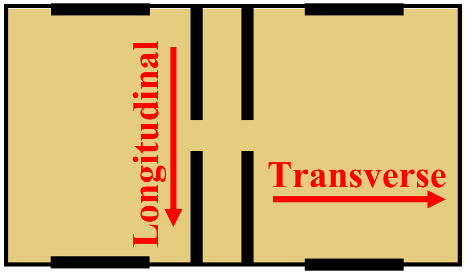

ASD, ρ=1.3, Ax=1.25
2
3
3
7604.8 lbs.
553 lbs.
(13.82 plf)
8565 lbs.
553 lbs.
(13.82 plf)
SW
SW
SW
63.35 plf net
71.38 plf
633.5 lbs.
713.8 lbs.
190.2 plf
24
176.3 + 13.82
= 214.12 plf
317.5 lbs.
356.8 lbs.
71.38 plf
63.35 plf Net
SW
317.5 lbs.
356.8 lbs.
If center SW removed, strut forces are increased
190.2 plf =
172.8
3.53 + 13.82
214.12 plf
633.5 lbs.
713.8 lbs.
63.35 plf net
71.38 plf
SW
SW
SW
V sw = 253.5 plf
Vnet = 253.5 – 172.8 – 13.82 – 3.53 = 63.35 plf
V sw = 285.5 plf
Vnet = 285.5 – 176.3 – 24 – 13.82 = 71.38 plf
0.553 k typ.
7.051 k
176.3 plf
Diaphragm Shears
0.141 k
3.53 plf
0.961 k
24 plf
172.8 plf
6.911 k

Design Example- Transverse Direction



Analysis Flow

Longitudinal Design



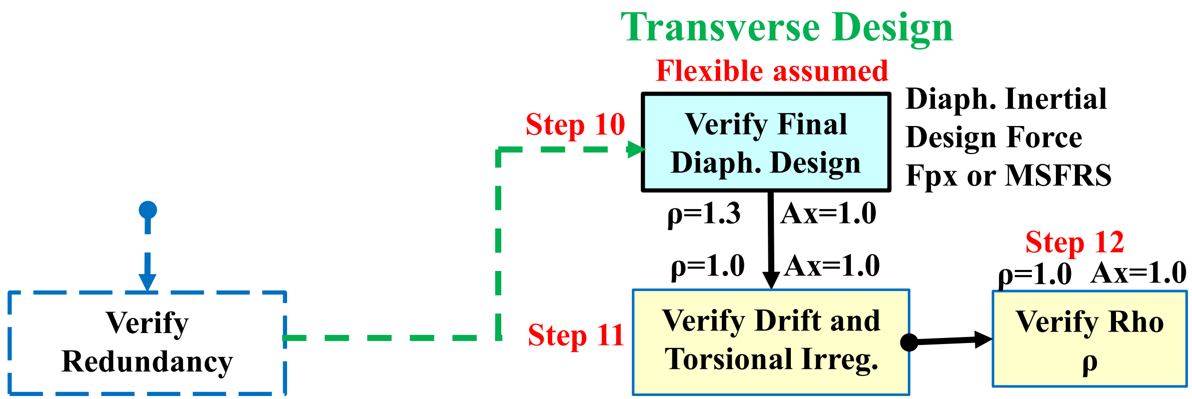
Example Plan

Legend

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

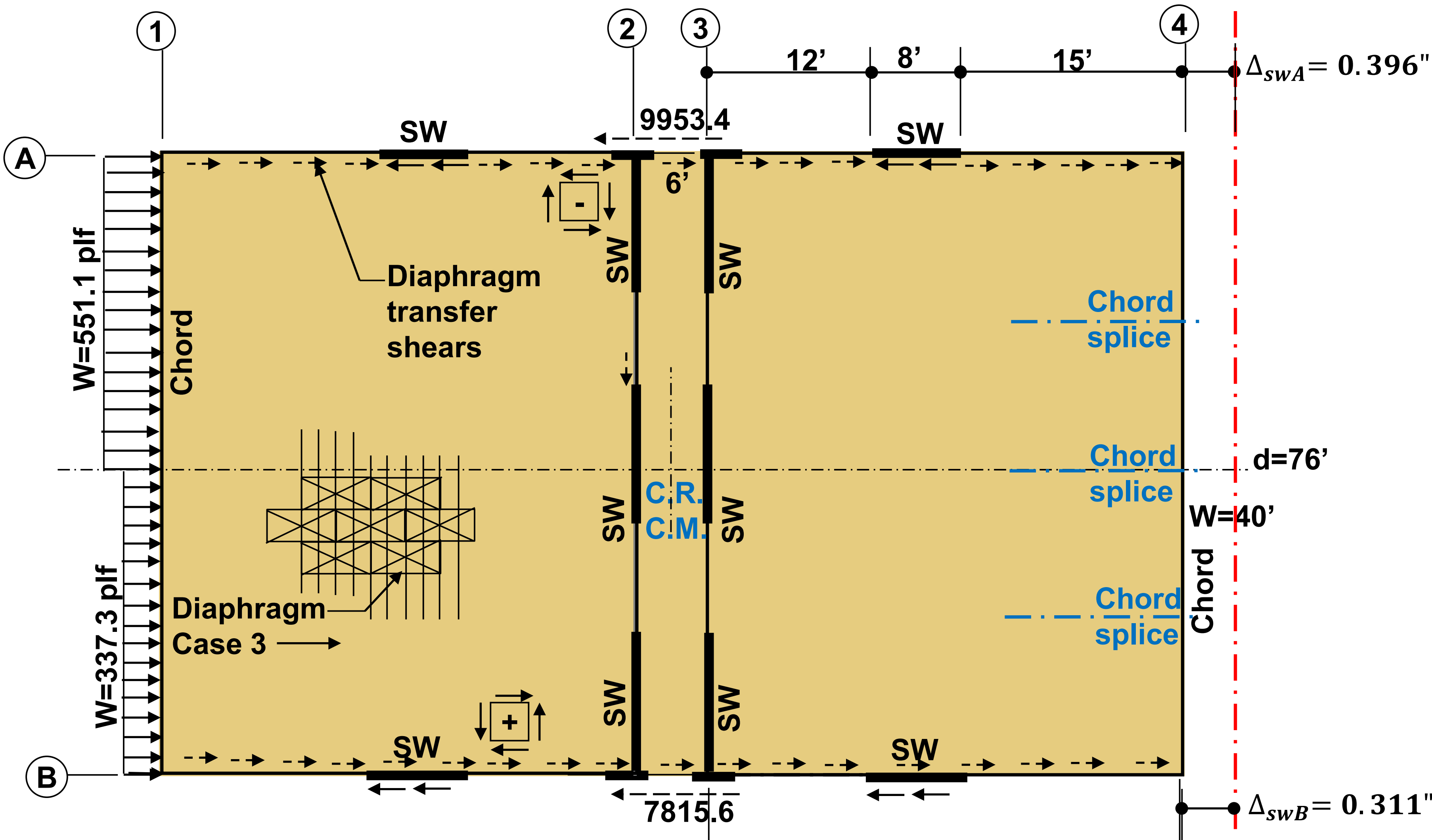
Transverse Design

Seismic- ρ=1.3, Ax=1.0



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 ≤ 1 ½" 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



Drift

ρ=1.0, Ax=1.25

Torsional and Redundancy Check

ρ=1.0, Ax=1.0

Page 60, 61

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

W= 17769/76=444.1 plf (ASD)

V_A=9057.6 lbs.

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

From spreadsheet (STR)

$\delta_{Diaph} = 0.066''$

$\Delta_{SW\ A} =$ 0.396", $\Delta_{SW\ B} =$ 0.311", $2x\Delta_{Average} = 0.707''$

0.066" < 0.707" ∴ Rigid diaphragm, as initially assumed.

Check Story Drift

$\rho = 1.0$ and $A_x = 1.25$

$C_d = 4$, $I_e = 1$

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

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

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

Check for Torsional Irregularity ρ=1.0, Ax=1.0

Rigid diaphragm, ρ =1.0 and Ax = 1.0 as required by ASCE 7 Table 12.3-1

From spreadsheet

δ_{SWA}=0.387"

δ_{SWB}=0.319"

$\Delta_{Average} = \frac{0.387+0.319}{2} = 0.353"$ **From spreadsheet**

0.387 < 1.2(0.353) = 0.424", ∴ No torsional irregularity exists in this direction, as assumed.

Redundancy Check ρ=1.0, Ax=1.0

Table 12.3-3 Requirements

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

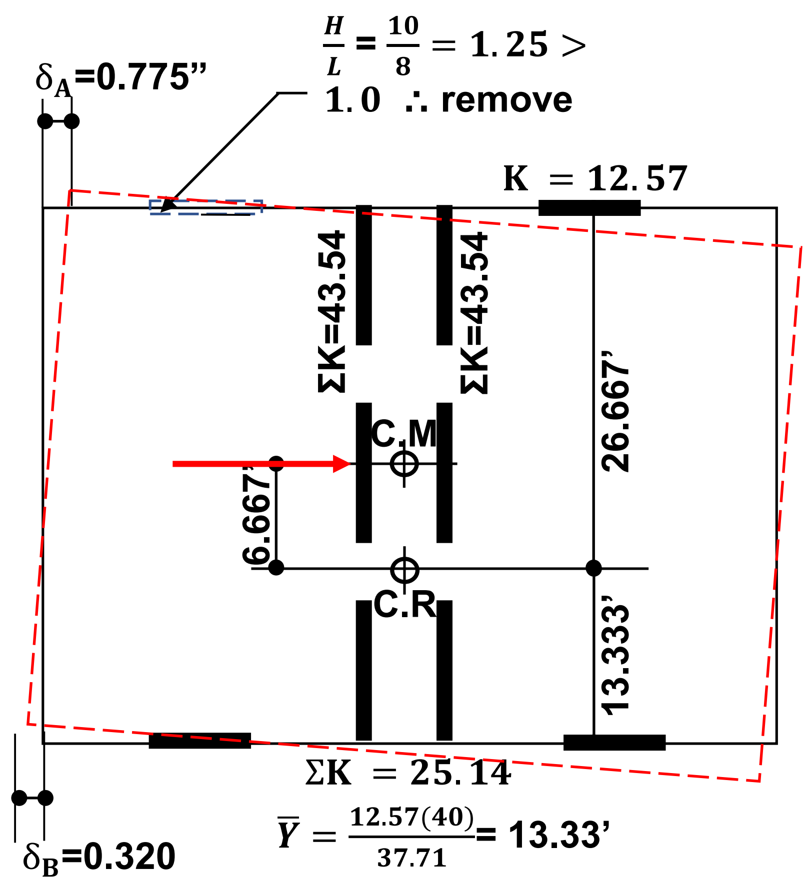
- δ_A= 0.775”

- δ_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” ∴ Type 1b ∴ ρ=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, Ax=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, ρ=1.3 Incorrect:
  - ρ = 1.0 Longitudinal
  - ρ = 1.3 Transverse

Other Design Requirements:

- Drift < allowable

Multi-Story, Stiffness Issues

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

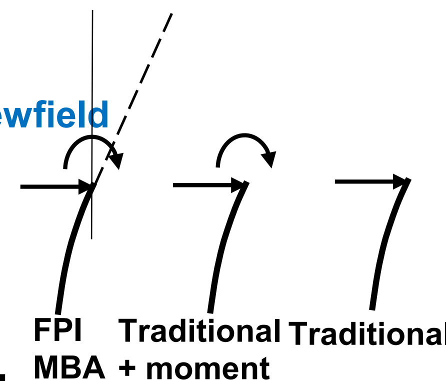
- **FPInnovations-Website NEW**
  "Seismic Analysis of Wood-Frame Buildings on Concrete Podium", Newfield

- **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



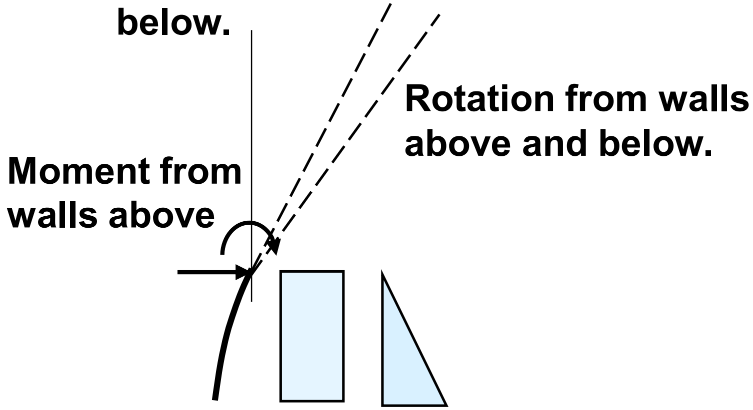
New Research and Analytical methods-Tall Shear Walls

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

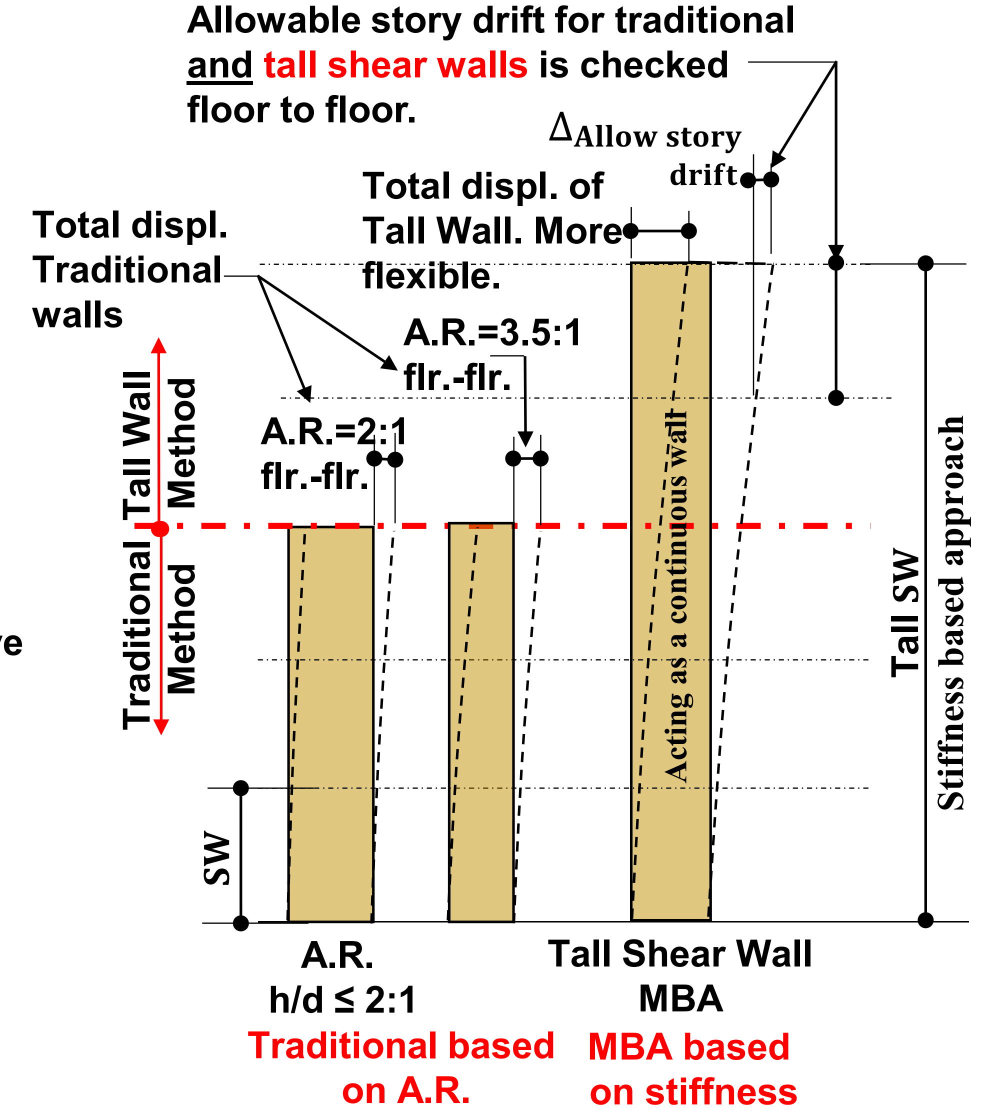
Testing shows that the traditional deflection equation is less accurate for walls with aspect ratios higher than 2:1.
(Dolan)

- 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.

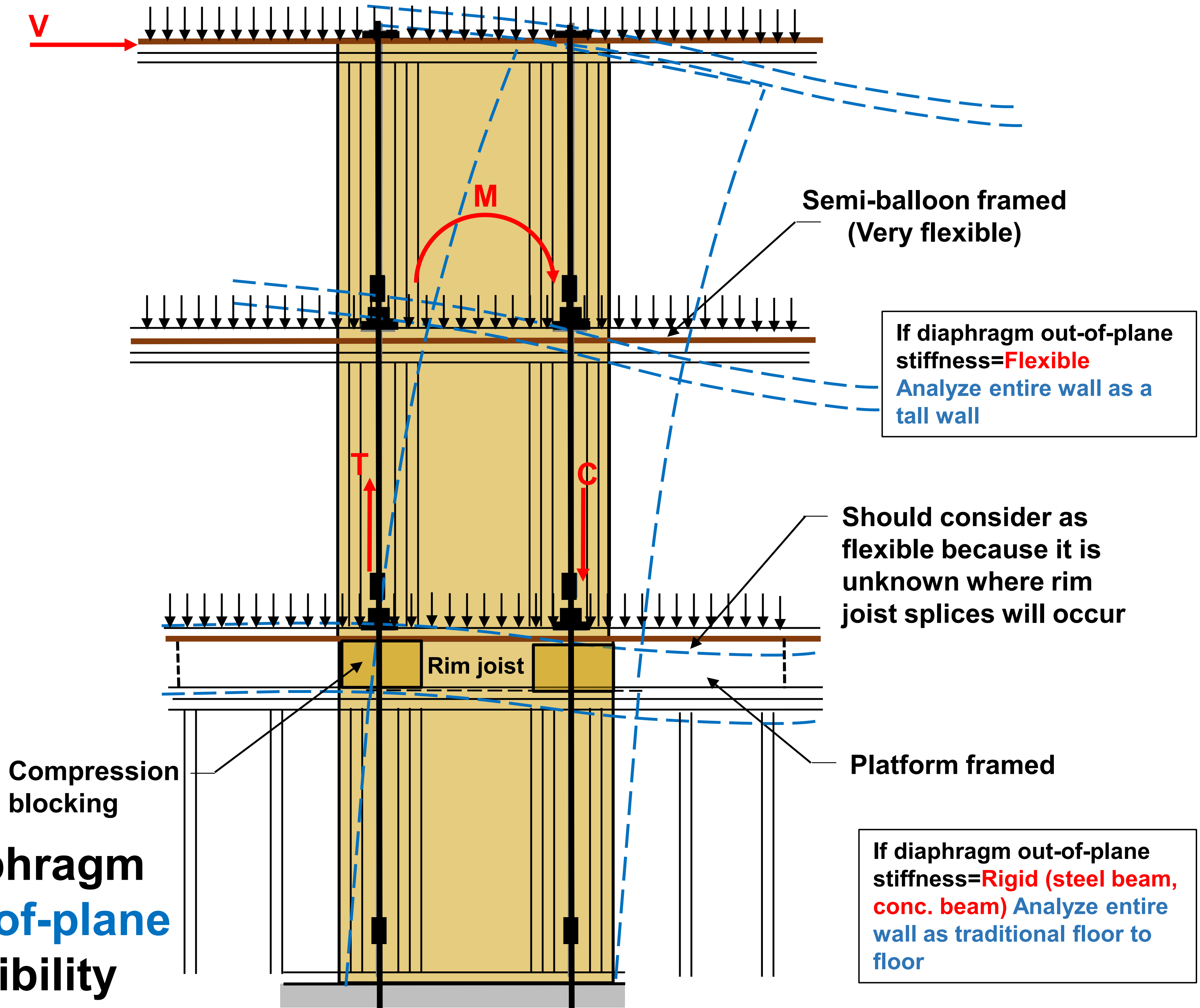


$$\frac{\sum M_i H_i^2}{2(EI)_i} + \frac{\sum V_i(H^3)}{3(EI)_i}$$



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

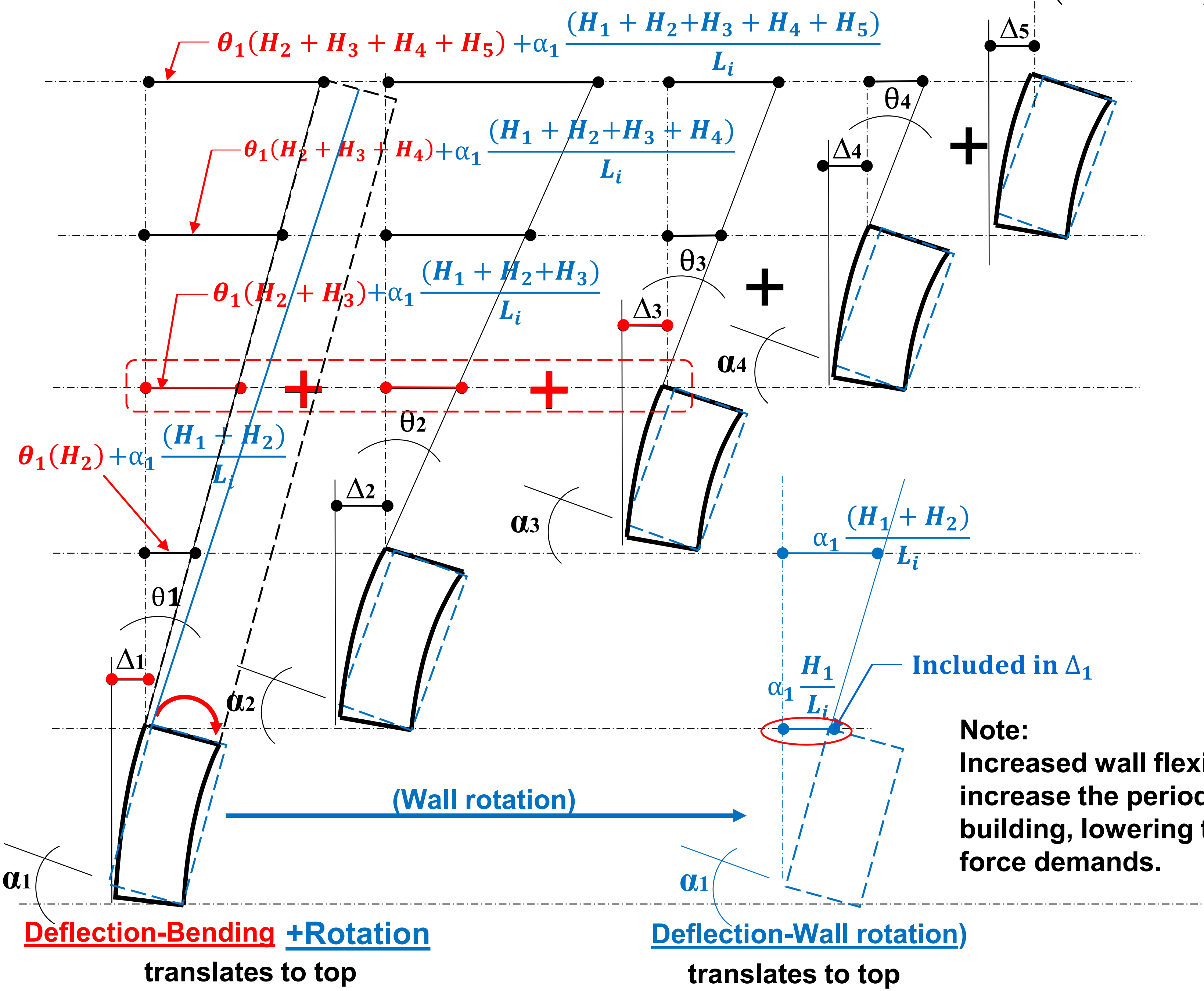
Not in example



Diaphragm out-of-plane Flexibility

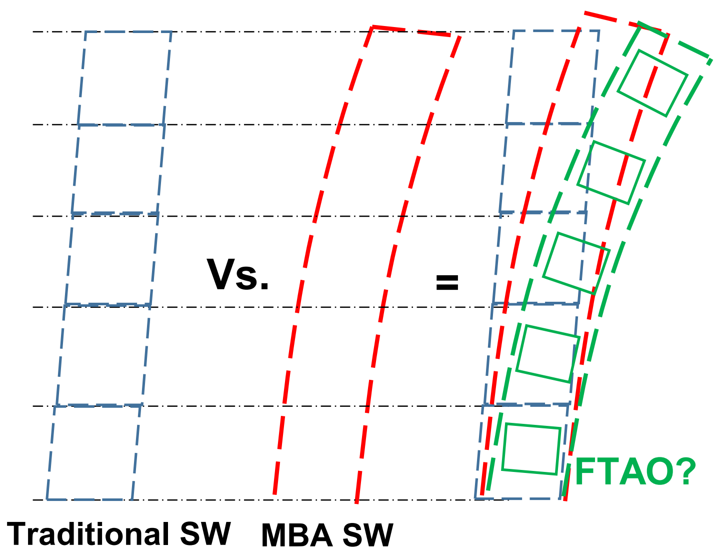
Tall Wall Deflection

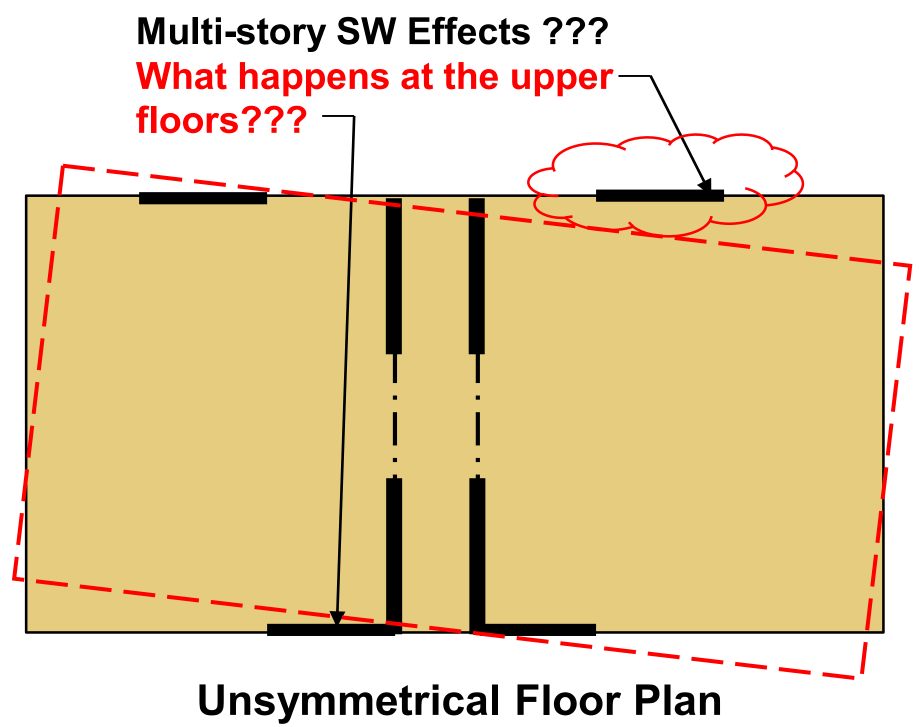
$$\Delta_i = \frac{\sum M_i H_i^2}{2(EI)_i} + \frac{\sum V_i (H^3)}{3(EI)_i} + \frac{V_i H_i}{G_{v,i} t_{v,i}} + 0.75 H_i e_{n,i} + \frac{H_i}{L_i} d_{a,i} + H_i \sum_{j=1}^{i-1} \left(\frac{M_j H_j}{(EI)_j} + \frac{V_j H_j^2}{2(EI)_j} \right) + H_i \sum_{j=1}^{i-1} \frac{d_{a,j}}{L_j}$$



Note:
Increased wall flexibility can increase the period of the building, lowering the seismic force demands.

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





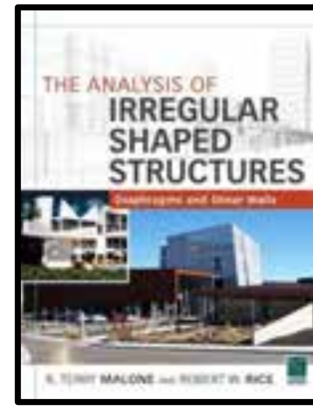
Question of the day:

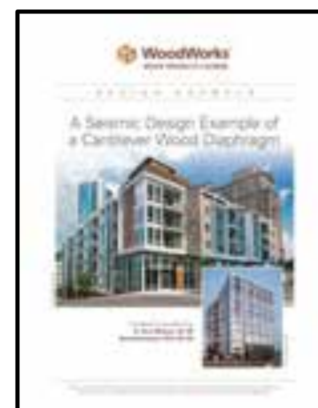
Reference Materials

- **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

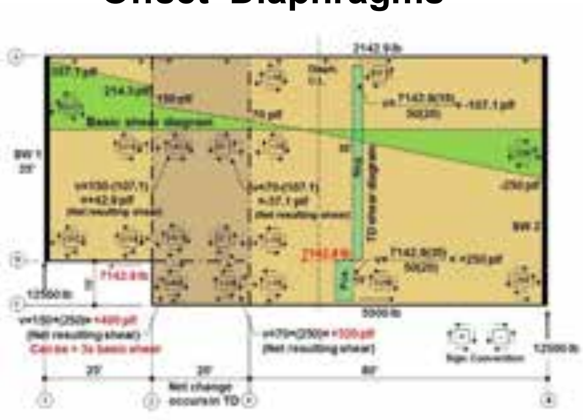
- **Woodworks-Guidelines for the Seismic Design of an Open-Front Wood Diaphragm (paper).** **Complete Example**





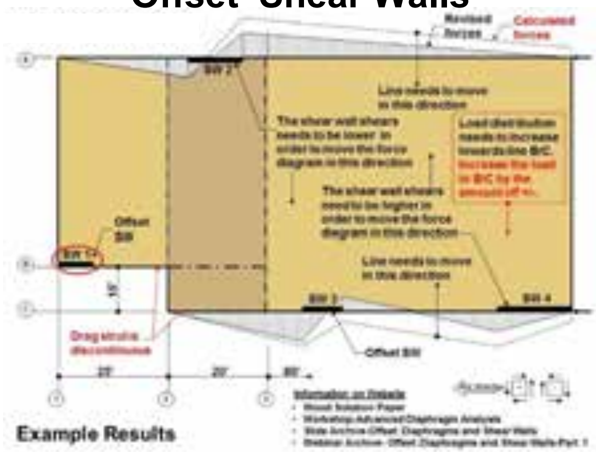
Method of Analysis and Webinar References

Offset Diaphragms



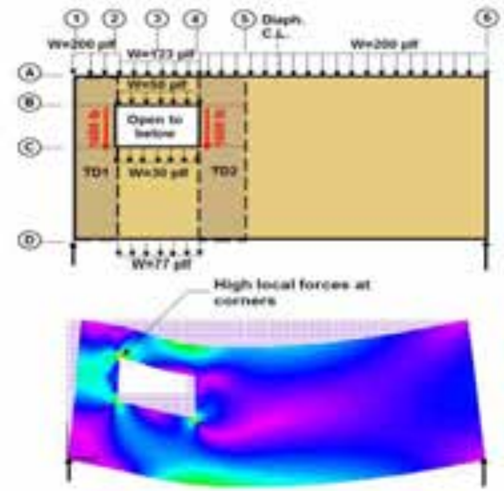
https://vimeo.com/woodproductscouncil/review/114574994/b64da97f09

Offset Shear Walls



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

Diaphragms Openings



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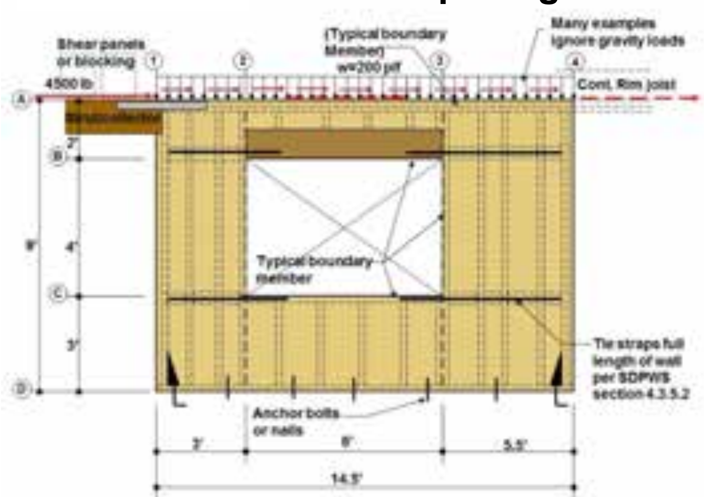
Shear Walls with Openings



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

Mid-rise Design Considerations

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

<u>Information on Website:</u> Presentation Slide Archives, Workshops, White papers, research reports

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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Thank You