A Design Example of a Wood Cantilever Diaphragm

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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.
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.
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-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:

- 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

- **Exterior shear walls**
  - A
  - B
  - C
  - D
- **If rectangular diaphragm**
- **Seismic Loads**
- **Support**
- **Rigid or spring Support**

**Unit with Exterior Wall**

- **Full support** (SW rigid)
- **Partial support** (Decreasing SW stiffness)
- **No support**

**Condition A**
- Flexible diaphragm

**Condition B**
- Loads shift

**Condition C**

**Podium**

- Full cantilever, no exterior wall support
- No significant exterior wall support. Conserv. to design as cantilever
- Most load goes to corridor walls. Check Diaph./SW stiffness, use RDA to design diaphragm
- Can be idealized as flexible diaphragm

Consider SW multi-story effects

Traditional SW vs. MBA SW

FTAO?
Review Stiffness at Offsets
Longitudinal Loads - Shear Wall A.R.=1.5:1

Bending increases at start of offset
Base Line=0

51.13 k
46.4 k
43.21 k

A.R. 1.5:1 wall
A.R. 2:1 wall
A.R. 3.5:1 wall

4.47
7.66
80.48

0.674
8.95
8.95

414.2'k
526.55'k
692.1'k

Full cantilever
Ext. SW
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!

Study Assumptions:
- Flexible Diaphragm.
- No torsion

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

Fixed support

- V=3.81k, k=40
- V=4.15k, k=40.71

(3) 10’ ext. walls
ΣLsw=30’, A.R.=1:1
- V=4.35k, k=41.06
- V=4.53k, k=41.36
- V=4.82k, k=41.8

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

Rigid support

(3) 8’ ext. walls
ΣLsw=24’
A.R.=1.25:1
- V=5.3k, k=42.43
- V=5.42k, k=42.58
- V=5.63k, k=42.81

(3) 6’ ext. walls
ΣLsw=18’
A.R.=1.67:1
- V=6.39k, k=43.56
- V=6.45k, k=43.61
- V=6.55k, k=43.7

(3) 4’ ext. walls
ΣLsw=12’
A.R.=2.5:1
- V=1.21k, k=8.2, %=21
- V=1.15k, k=7.74, %=19
- V=1.05k, k=6.98, %=17

Extending Diaph. walls

10d nails

Corridor Walls

Prelim conclusion (This example only):
- If walls near 44% or if k ≤ 20 consider open-font
- Magic 20’ SW
Minimum Design Check Considerations (You make the judgement call)

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
- No support
- 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.
A matter of Stiffness

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

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

Flexible structures are susceptible to damage from wind or seismic forces

Can require engineering judgement
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
(Not enough shear walls across front)
Possible Soft Story

An Engineered Structure?

Possible Soft Story

No shear walls
Open-Front Diaphragms
Relevant 2015 SDPWS Sections

4.2.5.2 Open Front Structures:

New definitions added:
- Open front structures
- Notation for $L'$ and $W'$ for cantilever Diaphragms

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

Similar to MS-MF structures
SDPWS 4.2.5.2 Open Front Structures: (Figure 4A)

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

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

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

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

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

5. The diaphragm length, L’, (normal to the open side) does not exceed **35 feet**.
   (2008 SDPWS: L’\(_{\text{max}}\)=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

Disclaimers:
The following information is an open-front diaphragm example which is subject to further revisions and validation. The information provided is project specific, and is for informational purposes only. It is not intended to serve as recommendations or as the only method of analysis available.
Open Front Structures **Code Checks:**

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

1. Verify aspect ratios of diaphragms and shear walls  
   SDPWS 4.2.5.2 (2), 4.3.4
2. Verify diaphragm length, L’  
   SDPWS 4.2.5.2(4)
3. Check stiffness of diaphragm and shear walls  
   ASCE 7 12.3.1, SDPWS 4.2.5.2 (3)
4. Check diaphragm flexibility  
   ASCE 7 12.3, SDPWS 4.2.5.2 (3)
5. Check drift at edges  
   ASCE 7 12.8.6, SDPWS 4.2.5.2 (3)
6. Check for torsional irregularity  
   ASCE 7 12.3.2.1 and 12.3.3, 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
7. Assume or verify redundancy  
   ASCE 7 12.3.4

For resistance to **Wind** loads, **recommend** complying with **all** of the following requirements:

- Items 1 thru 4 as noted above.
- Item 5 noted above for drift check– P-Delta and non-structural components
Example Plan

Page 5
Typical Exterior Wall Sections
Floor or roof sheathing

Blocking or continuous rim joist

Continuous rim joist, beam, special truss or double top plate can be used as strut / collector or chord.

Splice at all joints in boundary element

Trusses, top chord bearing with blocking between (shown)

Bm./Strut

Bn

Strut

Opening

Header

Platform framing

Opening

Column

Semi-balloon framing

SW

SW

Typical Exterior Wall Elevations at Grid Lines A and B
Platform Framing at Corridor

Semi-balloon Framing at Corridor

Typical Wall Sections at Corridor Walls
Let’s Take a 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
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:
\[ A_x = 1.0 \]
\[ \rho = 1.0 \]

\( A_x \): Amplification of accidental torsion if torsional irregularity exists
\( \rho \): Redundancy

Regular Plans
Questionable Plans-Unsymmetrical Plan Layouts

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

Page 62, 63
Questionable Plans-Corridor Walls One Side Only

Rotation transfer shears

Shear panels, blocking or framing members over entire wall line to transfer diaphragm shears down into the shear walls

Assume: \( A_x > 1.0 \)
\( \rho = 1.3 \)

Inherent Torsion + Amplified Accidental Torsion

Non-shear wall

\( W' = 40' \)

\( L' = 35' \)

Page 64, 65

\( L = 76' \)
Questionable Plans-Complex Plans-horizontal offsets

Chord fixity at corridor walls

Assume:
\[ A_x > 1.0 \]
\[ \rho = 1.3 \]

Chord splice location

Walls have no stiffness

Chord splice location

Chord splice location

Chord fixity at corridor walls

Additional units as occurs

Page 65, 66
Assume:
\[ A_x > 1.0 \]
\[ \rho = 1.3 \]
Questionable Plans-Core Structures

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

- Light framed
- CLT
**Typical Spreadsheet**

### Rigid Diaphragm Analysis

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### Longitudinal Analysis

#### Transverse Loading

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#### Shear Walls LC7

**LC7=0.726D+14**

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#### Shear Walls LC6

**LC6=1.347D+20**

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#### Diaphragm Deflection (STR)

**Rt. Cantilever**

**Lt. Cantilever**

---

Use this load combination for defining Nominal Stiffness values, \( K_{eff} \). Then use those \( K_{eff} \) values for all other analyses.

**Expected Dead + Seismic** 

\( K_{eff} = \frac{\delta_{sw}}{F_{K}} \)

**Calculation of Stiffness**

\( V \) equal to revised wall force based on HD STR (design) capacity

Use the following formulas:

\[ \delta_{sw} = \frac{F_{w}}{K_{eff}} \]

\[ K = \frac{F}{\delta_{sw}} \]

**Nominal wall stiffness**

**Calculate nominal stiffness by 3-term or 4-term deflection equation.**

\[ K = \frac{F}{\delta_{sw}} \]

\[ \delta_{sw} = \frac{F_{w}}{K_{eff}} \]

---

*Note: All values are approximate and should be verified with actual calculations.*
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, $\rho=1.3$

Force Distribution to Shear Walls

Seismic- $\rho=1.3$, $Ax=1.25$
Basic Project Information

- Structure-Occupancy B, Office, Construction Type VB-Light framing:
  - Wall height=10’-Single story
  - L=76’, total length
  - W’=40’, width/depth
  - L’=35’, cantilever length (max.)
  - 6’ corridor width

- Roof DL (seismic)= 35.0 psf including wall/ partitions

- Wall DL = 13.0 psf (in-plane)

- Roof snow load = 25 psf > required roof LL=20 psf

- Roof (lateral)= roof + wall H/2 plus parapet
Lateral Load Calculations - Seismic

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

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

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

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

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

\[ C_s = \frac{S_{DS}}{R} = 0.167 \] short period controls 12.8-2

Basic lateral force MSFRS

\[ V = C_sW = 0.167(35)(76)(40) = 17769 \text{ lbs. STR} \]
\[ 17769(0.7) = 12438 \text{ lbs. ASD} \]

Rigid Diaphragm Analysis- \( \rho=1.3, \text{Ax}=1.25 \)
Initial wall stiffness will be based on wall length.

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

RDA Equations

\[ T = V(e)(Ax)(\rho) \text{ ft. lbs.} \]
\[ F_T = T \frac{kd}{\sum kd^2_x + kd^2_y} \]
\[ F_{sw} = F_V + F_T \]
\[ F_V = F_x \frac{k}{\sum k} \]
\[ J = \sum kd^2_x + kd^2_y \]
Preliminary Shear Wall Design
Analysis Flow

Longitudinal Design

Translation
Rotation
Displacements
affected by wall
stiffness

Diaphragm
deflection

Example Plan
- Drift
- Torsional Irreg.

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

Page 12
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 ≤ 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.
  \( E = 1,700,000 \text{ psi, wall studs @ 16” o.c.} \)
  \( \text{EA} = 42,075,000 \text{ lbs. at grid line A,B} = (3)2x6 \text{ D.F., KD, studs @16” o.c. boundary elem.} \)
  \( \text{EA} = 28,050,000 \text{ lbs. at grid line 2,3} = (2)2x6 \text{ D.F., KD, studs @16” o.c. boundary elem.} \)

- Calculate wall deflection
• Shear Wall Deflection-calculated using:

Traditional 4 term deflection equation

\[ \delta_{SW} = \frac{8v h^3}{E A b} + \frac{v h}{G_v t_v} + 0.75 h e_n + \frac{h \Delta_a}{b_{\text{eff}}} \]  

C4.3.2-1

SDPWS combines

Bending | Nail slip | Rod elongation (Wall rotation)

Shear

SDPWS 3 term deflection equation

\[ \delta_{SW} = \frac{8v h^3}{E A b} + \frac{v h}{1000G_a} + \frac{h \Delta_a}{b_{\text{eff}}} \]  

4.3-1 Alt.

Bending | Vertical elongation

Apparent shear stiffness

• Nail slip
• Panel shear deformation

Where

\( v = \) wall unit shear (plf)
\( h = \) wall height (ft.)
\( b_{\text{eff}} = \) Wall rotation width (ft.)
\( b = \) Wall width (ft.)
\( G_a = \) apparent shear stiffness (k/in.)
\( \Delta_a = \) Sum of vertical displacements at anchorage and boundary members (in.)

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 = \( \frac{T(\text{Allow.Displ})}{\text{ASD Capacity}} \)
  - 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 = \( (0.0025)(D)(\text{Starting MC} - \text{End MC}) \)

  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.

$F_{c\perp0.02} = 0.73 F'_{c\perp} = 0.73(625) = 456.3$ psi

$F_{c\perp0.04} = F'_{c\perp} = 625$ psi

When $f_{c\perp} \leq F_{c\perp0.02}$

$\Delta_{crush} = 0.02 \left( \frac{f_{c\perp}}{F_{c\perp0.02}} \right)$

When $F_{c\perp0.02} \leq f_{c\perp} \leq F_{c\perp0.04}$

$\Delta_{crush} = 0.04 - 0.02 \left( \frac{1 - \frac{f_{c\perp}}{F_{c\perp0.04}}}{0.27} \right)$

When $f_{c\perp} > F_{c\perp0.04}$

$\Delta_{crush} = 0.04 \left( \frac{f_{c\perp}}{F_{c\perp0.04}} \right)^3$

If $f_{c\perp} = \left( \frac{c}{A_{chord}} \right) < 456.3$ psi, Crushing $= 0.02 \left( \frac{f_{c\perp}}{456.3} \right)(1.75)$
Shear Wall Rotation
Proposed nomenclature of next edition of SDPWS

Current term = \( \frac{h\Delta_a}{b} \)

Wall rotation:
- Hold down slip/elongation
- Sole plate shrinkage
- Sole plate crushing

\( SW_{rot} = \frac{h\Delta_a}{b_{eff}} \) or \( SW_{rot} = \frac{h\Delta_{a,eff}}{b} \)

Where
- \( h \)=wall height (ft.)
- \( b_{eff} \)=Wall rotation arm (ft.)
- \( b \)=Wall width (ft.)

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

\( \Delta_a \)=Sum of vertical displacements at tension edge of wall

\( \Delta_a = 0.25'' \)

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

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

\( SW_{rot} = \frac{10(0.267)}{8} = 0.333'' \)
Load Combinations (ASD):

\[
\begin{align*}
\text{LC8} &= 1.152D + 0.7\rho Q_E \\
\text{LC9} &= 1.114D + 0.525\rho Q_E + 0.75S \\
\text{LC10} &= 0.448D + 0.7\rho Q_E
\end{align*}
\]

Full dead loads shown, 1.0D
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.**

<table>
<thead>
<tr>
<th>SW Line</th>
<th>Ky k/in</th>
<th>Kx k/in</th>
<th>Dx Ft.</th>
<th>Dy Ft.</th>
<th>Kd</th>
<th>Kd²</th>
<th>Fv Lbs.</th>
<th>Ft Lbs.</th>
<th>Total Lbs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>------</td>
<td>16</td>
<td>------</td>
<td>20</td>
<td>320</td>
<td>6400</td>
<td>0</td>
<td>1842.4</td>
<td>1842.4</td>
</tr>
<tr>
<td>B</td>
<td>------</td>
<td>16</td>
<td>------</td>
<td>20</td>
<td>320</td>
<td>6400</td>
<td>0</td>
<td>-1842.4</td>
<td>-1842.4</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>------</td>
<td>3</td>
<td>------</td>
<td>90</td>
<td>270</td>
<td>8084.9</td>
<td>-518.2</td>
<td>7566.7</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>------</td>
<td>3</td>
<td>------</td>
<td>90</td>
<td>270</td>
<td>8084.9</td>
<td>518.2</td>
<td>8603.1</td>
</tr>
</tbody>
</table>

\[ \Sigma Ky=60 \quad \Sigma Kx=32 \]

\[ J=16169.8 \]

**Transverse Direction, e=2.5’, T = 40424.5 ft. lbs.**

<table>
<thead>
<tr>
<th>SW Line</th>
<th>Ky k/in</th>
<th>Kx k/in</th>
<th>Dx Ft.</th>
<th>Dy Ft.</th>
<th>Kd</th>
<th>Kd²</th>
<th>Fv Lbs.</th>
<th>Ft Lbs.</th>
<th>Total Lbs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>------</td>
<td>16</td>
<td>------</td>
<td>20</td>
<td>320</td>
<td>6400</td>
<td>8084.9</td>
<td>969.7</td>
<td>9054.6</td>
</tr>
<tr>
<td>B</td>
<td>------</td>
<td>16</td>
<td>------</td>
<td>20</td>
<td>320</td>
<td>6400</td>
<td>8084.9</td>
<td>-969.7</td>
<td>7115.2</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>------</td>
<td>3</td>
<td>------</td>
<td>90</td>
<td>270</td>
<td>0</td>
<td>-272.7</td>
<td>-272.7</td>
</tr>
<tr>
<td>3</td>
<td>30</td>
<td>------</td>
<td>3</td>
<td>------</td>
<td>90</td>
<td>270</td>
<td>0</td>
<td>272.7</td>
<td>272.7</td>
</tr>
</tbody>
</table>

\[ \Sigma Ky=60 \quad \Sigma Kx=32 \]

\[ J=16169.8 \]
Preliminary Shear Wall Design - Distribution based on wall lengths

Adding Gravity Loads to Shear Walls

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

**ASD Load Combination:** \( \text{LC10} = 0.448D + 0.7\rho Q \)
\( \rho = 1.3, A_x = 1.25 \)

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

**Shear Walls Along Grid Lines 2 and 3**
**Longitudinal Loading**
### Shear Wall Capacity-Wood Based Panels

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

<table>
<thead>
<tr>
<th>Sheathing Material</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Fastener Penetration In Framing Member or Blocking (in.)</th>
<th>Fastener Type &amp; Size (common or Galvanized box)</th>
<th>Panel Edge Fastener Spacing (in.)</th>
<th>Panel Edge Fastener Spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Structural Panels-Sheathing</td>
<td>15/32</td>
<td>1-3/8</td>
<td>8d</td>
<td>602</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td>1-1/2</td>
<td>10d</td>
<td>680</td>
<td>19</td>
</tr>
</tbody>
</table>

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

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

**SW_{2,3}:** Use 15/32” OSB w/ 10d@4” o.c., vs= (920)/2 = 460 plf, Ga=30

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

**ASD, \( \Delta a = 0.114” @ \) capacity

**STR, \( \Delta a = 0.154” @ \) 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 affect the distribution of loads to the shear walls which will affect the shear wall deflections. This can lead to a different set of stiffness values that may not be consistent.

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

**Sources of non-linearities:**

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

Since deflection is “non-linear”.... the stiffness can vary with the loading, even when using 3-term deflection equation.
LATERAL Load for Shear Wall Deflection & Stiffness Calculations

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

![Example 3-Term vs 4-Term Shear Wall Deflection](image)

- Lower stiffness from HD flexibility after uplift
- Net uplift
- Lightly Loaded Walls have most non-linearity

Method allows having only one set of nominal stiffness values.
Objective:
Use a single rational vertical and lateral load combination to calculate deflections and **Nominal** shear wall stiffness.

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

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

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

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

Proposing:
1. Stiffness calculated using 3-term eq. and LC 1.0D+Qe, with $\rho$=1.0 and $Ax$=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) \( \rho=1.0, A_x=1.0 \)

Load Combination: \( 1.0D + Q_E \)

<table>
<thead>
<tr>
<th>Grid Line</th>
<th>( G_a )</th>
<th>( V ) on wall</th>
<th>( v )</th>
<th>( T )</th>
<th>( C )</th>
<th>( \Delta_a )</th>
<th>( F_c )</th>
<th>Crush.</th>
<th>Shrink</th>
<th>( \delta_B )</th>
<th>( \delta_S )</th>
<th>( \delta_{Rot} )</th>
<th>( \delta_{SW} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>37</td>
<td>7308.0</td>
<td>913.5</td>
<td>6391</td>
<td>13770</td>
<td>0.154</td>
<td>556.36</td>
<td>0.056</td>
<td>0.019</td>
<td>0.022</td>
<td>0.247</td>
<td>0.313</td>
<td>0.581</td>
</tr>
<tr>
<td>B</td>
<td>37</td>
<td>7308.0</td>
<td>913.5</td>
<td>6391</td>
<td>13770</td>
<td>0.154</td>
<td>556.36</td>
<td>0.056</td>
<td>0.019</td>
<td>0.022</td>
<td>0.247</td>
<td>0.313</td>
<td>0.581</td>
</tr>
</tbody>
</table>

Calculate Stiffness of Walls on A & B using LRFD Capacity

Calculate Stiffness of Walls on 2 & 3 using LRFD Loading

| No | \( V \) on wall | \( v \) | \( T \) | \( C \) | \( \Delta_a \) | \( F_c \) | Crush. | Shrink | \( \delta_B \) | \( \delta_S \) | \( \delta_{Rot} \) | \( \delta_{SW} \) |
|----|----------------|------|------|------|-----------|--------|--------|--------|--------|--------|--------|--------|--------|
| 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

1836 D 158.3 D 158.3 D

Wall Capacity Grid A and B

Shear wall Grid A and B

Transverse Loading
Nominal Strength

Shear wall Grid 2 and 3

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), \( \Delta_a=0.154'' \)

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

<table>
<thead>
<tr>
<th>SW Line</th>
<th>Ky k/in</th>
<th>Kx k/in</th>
<th>Dx Ft.</th>
<th>Dy Ft.</th>
<th>Kd</th>
<th>Kd²</th>
<th>Fv Lbs.</th>
<th>Ft Lbs.</th>
<th>Total Lbs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-------</td>
<td>25.14</td>
<td>-------</td>
<td>20</td>
<td>502.8</td>
<td>10056</td>
<td>0</td>
<td>1848.1</td>
<td>1848.1</td>
</tr>
<tr>
<td>B</td>
<td>-------</td>
<td>25.14</td>
<td>-------</td>
<td>20</td>
<td>502.8</td>
<td>10056</td>
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<td>-1848.1</td>
<td>-1848.1</td>
</tr>
<tr>
<td>2</td>
<td>43.54</td>
<td>-------</td>
<td>3</td>
<td>-------</td>
<td>130.62</td>
<td>391.86</td>
<td>8084.9</td>
<td>-480.1</td>
<td>7604.8</td>
</tr>
<tr>
<td>3</td>
<td>43.54</td>
<td>-------</td>
<td>3</td>
<td>-------</td>
<td>130.62</td>
<td>391.86</td>
<td>8084.9</td>
<td>480.1</td>
<td>8565.0</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>SW</th>
<th>Ky k/in</th>
<th>Kx k/in</th>
<th>Dx Ft.</th>
<th>Dy Ft.</th>
<th>Kd</th>
<th>Kd²</th>
<th>Fv Lbs.</th>
<th>Ft Lbs.</th>
<th>Total Lbs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>-------</td>
<td>25.14</td>
<td>-------</td>
<td>20</td>
<td>502.8</td>
<td>10056</td>
<td>8084.9</td>
<td>972.7</td>
<td>9057.6</td>
</tr>
<tr>
<td>B</td>
<td>-------</td>
<td>25.14</td>
<td>-------</td>
<td>20</td>
<td>502.8</td>
<td>10056</td>
<td>8084.9</td>
<td>-972.7</td>
<td>7112.2</td>
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<tr>
<td>2</td>
<td>43.54</td>
<td>-------</td>
<td>3</td>
<td>-------</td>
<td>130.62</td>
<td>391.86</td>
<td>0</td>
<td>252.7</td>
<td>252.7</td>
</tr>
<tr>
<td>3</td>
<td>43.54</td>
<td>-------</td>
<td>3</td>
<td>-------</td>
<td>130.62</td>
<td>391.86</td>
<td>0</td>
<td>-252.7</td>
<td>-252.7</td>
</tr>
</tbody>
</table>

ΣKy=87.08  ΣKx=50.28  J=20895.72

**Nominal stiffness values used**

---

Page 26
ASD Load Combination: **LC10 0.448D + 0.7\(\rho\)Q\(_E\)**
\(\rho=1.3, A_x=1.25\)

![Shear wall Grid A and B](image1)

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

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

\[ T = 4579.2 \text{ lbs.} \approx 4565 \text{ lbs. allowed, 0.3\% over} \]

\[ \therefore \text{hold down o.k. –check later} \]

![Shear wall Grid 3](image2)

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

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

\[ T = 2557.1 \text{ lbs.} < 4565 \text{ lbs. allowed} \]

\[ \therefore \text{hold down o.k.} \]
Let’s Take a 15 Minute Break

Mass Timber Project
Part 3-Design Example (cont.):

- Diaphragm design
- Maximum diaphragm chord force
- Diaphragm flexibility
- Story drift
Diaphragm Design Forces: MSFRS or $F_{px}$
Example Plan

Analysis Flow
Longitudinal Design

Step 4
Diaphragm Design
ρ=1.0, Fpx, or ρ=1.3, Ax=1.25
(i.e. Diaph. or MSFRS Forces)

Diaphragm construction based on max. demand (Sht’g. / nailing)

Chord splice loc’s./slip

Max. diaphragm chord forces

Step 5
Diaphragm Flexibility

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

Page 28
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:

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

where

- $F_{px}$ = the diaphragm design force at level $x$
- $F_i$ = the design force applied to level $i$
- $w_i$ = the weight tributary to level $i$
- $w_{px}$ = the weight tributary to the diaphragm at level $x$

The force shall not be less than $F_{px} = 0.2S_{DS}lew_{px}$  \hspace{1cm} (12.10-2)

The force need not exceed $F_{px} = 0.4S_{DS}lew_{px}$  \hspace{1cm} (12.10-3)

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

For a single story structure $F_x = F_{px} = \frac{S_{DS}I_e}{R} w_{px}$
Method 1

Method 2A

Method 2B

Torsional Distribution - Not mandatory

(Question 4) \( \rho = 1.3, A_x = 1.25 \)

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

Method 2A will be used for all other checks
Using method 2B- $\rho = 1.3$, $A_x = 1.25$:

$F_T = \text{Torsion forces only at corridor walls, gridlines 2 and 3}$

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

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

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

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

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

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

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

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

Right cantilever

$W_1 = 181.65 \text{ plf}$

$W_2 = 185.65 \text{ plf}$

$W_T = 553.2 \text{ plf}$

Calculate Loads to Diaphragm ASD
Direction of diaphragm transfer shears (Bending)

Values (Tension)

Forces = \( \frac{M_x}{d} \)
### Table 4.2A  Nominal Unit Shear Capacities for Wood-Framed Diaphragms

<table>
<thead>
<tr>
<th>Sheathing Grade</th>
<th>Common Nail Size</th>
<th>Minimum Fastener Penetration In Framing Member or Blocking (in.)</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal width Of nailed face At adjoining Panel edges and boundaries (in.)</th>
<th>A Seismic Nail spacing (in.) at boundaries (all cases), at continuous panel edges parallel to load (cases 3 &amp; 4), and at all panel edges (cases 5 &amp; 6).</th>
<th>B Wind Panel Edge Fastener Spacing (in.)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6 4 2 ⅓ 2</td>
<td>6 4 2 ½ 2</td>
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<tr>
<td>Sheathing and Single floor</td>
<td>8d</td>
<td>1-3/8</td>
<td>7/16</td>
<td>3</td>
<td>570 11 9 760 7 1140 10 8 1290 17 12 800 1065 1595 1805</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15/32</td>
<td>2</td>
<td>540 13 9.5 720 7.5 6.5 1060 11 8.5 1200 19 13 755 1010 1485 1680</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>600 10 8.5 800 6 5.5 1200 9 7.5 1350 15 11 840 1120 1680 1890</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>15/32</td>
<td>2</td>
<td>580 25 15 770 15 11 1150 21 14 1310 33 18 810 1080 1610 1835</td>
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<td></td>
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<td>650 21 14 860 12 9.5 1300 17 12 1470 28 16 910 1205 1820 2060</td>
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<td></td>
<td>3</td>
<td>720 17 12 960 10 8 1440 14 11 1640 24 15 1010 1345 2015 2295</td>
<td></td>
</tr>
</tbody>
</table>

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

**Unit torsional shear = 24.32 plf**

\[ V_{\text{Max diaph}} = 176.3 + 24.3 = 200.6 \text{ plf}. \]

\[ 200.6 \text{ plf} < vs = 0.5(580) = 290 \text{ plf. o.k.} \]

**Ga = 25, blocked**
Determine Maximum Chord Force
(Answer questions 5 and 6)
1. By inspection, the walls along the chord line affect the chord forces by a small amount, 364.8 lbs.
2. Calculations show that the conc. wall force at end of cantilever increase the chord force by +21% at the 15’ splice diminishing to +9% increase at 23’, and +1% at the support. Walls had a larger effect.
Diaphragm Chords

<table>
<thead>
<tr>
<th>Splice Forces (lbs.)</th>
<th>( \Delta_{slip} )</th>
<th>( v \text{ unif. plf} )</th>
<th>( v \text{ conc. plf} )</th>
<th>( G_a )</th>
<th>( L' )</th>
<th>( W' )</th>
<th>Diaph Uni</th>
<th>Diaph conc</th>
<th>Total ( \Delta )</th>
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<tbody>
<tr>
<td>F 15</td>
<td>1094.3</td>
<td>1180.9</td>
<td>3253.7</td>
<td>0.072</td>
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<td>13.83</td>
<td>25.0</td>
<td>35.00</td>
<td>40.00</td>
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<tr>
<td>F23</td>
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<td></td>
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</table>

Nails Req'd = 4.84 5.23 14.40
Use Nails = 8 16 24
Slip = 0.023 0.013 0.023
Wall Load = 553.2 553.2 553.2

Inclu. effects of sw's along chord line

Method 2B

Diaphragm Deflection (ASD)

<table>
<thead>
<tr>
<th></th>
<th>( \Delta_{slip} )</th>
<th>( v \text{ unif. plf} )</th>
<th>( v \text{ conc. plf} )</th>
<th>( G_a )</th>
<th>( L' )</th>
<th>( W' )</th>
<th>Diaph Uni</th>
<th>Diaph conc</th>
<th>Total ( \Delta )</th>
</tr>
</thead>
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<tr>
<td></td>
<td>353.6</td>
<td>1884.0</td>
<td>3383.5</td>
<td>0.070</td>
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<td>25.0</td>
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<td>1.56</td>
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<td>14.77</td>
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<tr>
<td></td>
<td>8</td>
<td>16</td>
<td>24</td>
<td></td>
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<td>0.008</td>
<td>0.020</td>
<td>0.024</td>
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</table>

Maximum chord force = 3383.5 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.

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

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

V_T = 24.32 plf
V_T = 24.32 plf
Vsw=26.4 plf
(net=2.08 plf)

Vsw Line 1848.1 lbs.
Torsional SW/Strut forces

Similar to APA Example
• No fixity at support
• No chord bending
• No net rotational shears
• If partial length end walls, will develop strut forces

Page 64
Diaphragm Flexibility, $\rho=1.0$, $Ax=1.25$

How does this relate to this?

ASCE 7-16 Figure 12.3-1
Analysis Flow
Longitudinal Design

Example Plan

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

Check Diaphragm Flexibility
Seismic- ρ=1.0, Ax=1.25

Page 41
Diaphragm Flexibility

- 12.3.1.1 Flexible Diaphragm Condition.
  - Untopped steel decking or wood structural panels
  - Permitted to be idealized as flexible under certain conditions.

- 12.3.1.2 Rigid Diaphragm Condition.
  - Concrete slabs or concrete-filled metal deck (No mention of wood)
  - Span-to-depth ratios of 3 or less with no horizontal irregularities
  - Permitted to be idealized as rigid.

- 12.3.1.3 Calculated Flexible Diaphragm Condition.
  - Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2
  - Permitted to be idealized as flexible provided: \( \delta_{\text{MDD}} > 2\Delta_{\text{ADVE}} \).

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

- 2015 SDPWS 4.2.5 Horizontal Distribution of Shear
  - Idealize as rigid when \textbf{computed} \( \delta_{\text{MDD}} \leq 2\Delta_{\text{ADVE}} \)

(a) ASCE 7-16 Figure 12.3-1
Simple Span Diaphragm
Determination of Cantilever Diaphragm Flexibility

To What Degree, Rigid or Semi-rigid?

(a) ASCE 7-16 Figure 12.3-1

Diaphragm Deflection Typ.

\[ \Delta A \]

\[ \delta_{\text{Diaph}} \]
Rigid/Semi-rigid

\[ \delta_{\text{Diaph}} \]
Flexible

Simple Span Diaphragm

SW

L

Diaphragm Length

\[ \Delta B \]

\[ 2 \times \Delta_{\text{ADVE}} \]

\[ \delta_{\text{MDD}} \]

\[ >2 \times \Delta_{\text{ADVE}} \]

Based on adjacent SW only

(b) Corridor Walls Only

Preferred Method – Simplifies Check

Can require engineering judgement

Diaphragm Deflection Typ. (\(\delta_{\text{MDD}}\))

\[ \delta_{\text{Diaph}} \]
Rigid/Semi-rigid

\[ \delta_{\text{Diaph}} \]
Flexible

Allows additional diaphragm flexibility to be classified as semi-rigid or rigid if adjacent wall method used (not average).
(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:**

\[
\delta_{\text{Diaph Unif}} = \frac{3\nu L'^3}{EAW'} + \frac{0.5\nu L'}{1000G_a} + \frac{\Sigma x'\Delta_c}{W'}
\]

**Four-term equation for uniform load:**

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

**Three-term equation for point load:**

\[
\delta_{\text{Diaph Conc}} = \frac{8\nu L'^3}{EAW'} + \frac{\nu L'}{1000G_a} + \frac{\Sigma x'\Delta_c}{W'}
\]

**Four-term equation for point load:**

\[
\delta_{\text{Diaph Conc}} = \frac{8\nu L'^3}{EAW'} + \frac{\nu L'}{Gvtv} + 0.75 L' e_n + \frac{\Sigma x'\Delta_c}{W'}
\]

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

\[
\delta_{\text{Diaph Unif}} + \delta_{\text{Diaph Conc}} = \frac{3\nu L'^3}{EAW'} + \frac{0.5\nu L'}{1000G_a} + \frac{\Sigma x'\Delta_c}{W'} + \frac{8\nu L'^3}{EAW'} + \frac{\nu L'}{1000G_a} + \frac{\Sigma x'\Delta_c}{W'} + 0.376 L' e_n + \frac{\Sigma x'\Delta_c}{W'} + 0.75 L' e_n + \frac{\Sigma x'\Delta_c}{W'} + \Sigma x'\Delta_c
\]

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
- \( v_{\text{max}} \) = induced unit shear at the support from a uniform applied load, lbs/ft
- \( G_a \) = apparent diaphragm shear stiffness from nail slip and panel shear deformation, kips/in
- \( Gvtv \) = Panel rigidity through the thickness
- \( X' \) = distance from chord splice to the free edge of the diaphragm, ft
- \( \Delta_c \) = diaphragm chord splice slip, in.
- \( \delta_{\text{Diaph Unif}} \) = 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 \( v_{\text{max}} \)
- \( \delta_{\text{Diaph Conc}} \) = calculated deflection at the free edge of the diaphragm, in.

EA chords =28,050,000 lbs., 2-2x6 wall top plate.
Longitudinal Loading  e=4.75', T = 84403 ft. lbs., ρ=1.0, Ax=1.25

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<tr>
<th>Grid Line</th>
<th>kx</th>
<th>Ky</th>
<th>dx</th>
<th>dy</th>
<th>kd</th>
<th>kd²</th>
<th>Fv</th>
<th>Ft</th>
<th>Fv+Ft</th>
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<td>43.54</td>
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<td>-527.7</td>
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<tr>
<td>3</td>
<td>43.54</td>
<td>3</td>
<td>130.63</td>
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<td>8884.5</td>
<td>527.7</td>
<td>9412.2</td>
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<td>A</td>
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<tr>
<th>Diaphragm Deflection (STR)</th>
<th>Rt. Cantilever</th>
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<tr>
<td>Splice Forces (Lbs.)</td>
<td>Σδ slipped</td>
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<td>F 15</td>
<td>F23</td>
</tr>
<tr>
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<tr>
<td>EA= 28050000, (2)2x6</td>
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<tr>
<td>includes effects of sw's along chord line</td>
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Method 2A: Chord splice Chord splice Chord splice Chord splice 231.61 236.00

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<th>Diaphragm Deflection (STR)</th>
<th>Lft. Cantilever</th>
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<td>16</td>
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<td>0.005</td>
<td>0.021</td>
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</table>

Flexibility and Drift
Page 43
Diaphragm Deflection - Method 2A, $p=1.0$, $A_x=1.25$

$$\delta_{Diaph\ Unif} = \frac{3v_{max}L^3}{EAWr} + \frac{0.5v_{max}L'}{1000G_a} + \frac{\Sigma A_cX_c}{W'}$$

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”

$$2 \times \Delta_3 = 0.432\"$$

$$0.265\" < 0.432\" \therefore \text{Diaphragm can be idealized as Rigid}$$

Diaphragm Flexibility – Wind

- ASCE 7-16, Chapter 27, Section 27.5.4-DIAPHRAGM FLEXIBILITY-requires that the structural analysis shall consider the stiffness of diaphragms and vertical elements of the main wind force resisting system (MWFRS).

- Section 26.2 - Definitions, DIAPHRAGM, diaphragms constructed of WSP are permitted to be idealized as flexible.

- There is no drift limit requirement in the code for wind design.
Story Drift, $\rho=1.0$, $Ax=1.25$
Check Story Drift
Seismic - ρ=1.0, Ax=1.25
Page 44
**Story Drift**

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

- Story drift ($\Delta$) shall be computed as the difference of the deflections at the centers of mass 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 **horizontal irregularity Type 1a or 1b** of Table 12.3-1, the design story drift, $\Delta$, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure.

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

- Maximum story drift at each edge of the structure $\leq$ 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 $C_d$.

\[
\delta_x = \frac{C_d \delta_{xe}}{I_e} \quad (12.8-15)
\]
\[ \Delta \text{Drift} = \delta_T + (\delta_D - \delta_{RL}) \]

\[ \delta_T = 0.204'' \]

\[ \Delta \text{Drift} = \delta_T + (\delta_D + \delta_{RL}) + \delta_{RL} = 0.154'' \]

\[ \delta_D = 0.265'' \]

\[ \Delta_1 = 0.192'' \]

\[ \Delta_2 = 0.216'' \]

\[ \delta_{RT} = 0.081'' \]

Diaphragm deflection

Drift-Method 2A

Page 44, 46
Drift-Method 2A  \( \rho=1.0, \text{ Ax}=1.25 \)

Drift \( \Delta = \delta_{Diaph} + \delta_{Rotation} + \delta_{Translation} \)

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

\( \delta_3 = 9.412 \text{ k} / 43.54 \text{ k/in} = 0.216 \text{ in} \),

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

\( \delta_B = -2.031 \text{ k} / 25.14 \text{ k/in} = -0.081 \text{ in} \),

\( \Delta_{Diaph} = 0.265'' \)

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

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

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

\( Cd = 4, \text{ le} = 1 \)

\( \delta_M = \frac{Cd \delta_{max}}{I_e} = \frac{4(0.628)}{1} = 2.51'' \)
Table 12.12-1 Allowable Story Drift, $\Delta_a$

<table>
<thead>
<tr>
<th>Structure</th>
<th>Risk Category</th>
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<tbody>
<tr>
<td>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.</td>
<td>I or II: 0.025hsx, III: 0.020hsx, IV: 0.015hsx</td>
</tr>
<tr>
<td>Masonry cantilever shear wall structures</td>
<td>I or II: 0.010hsx, III: 0.010hsx, IV: 0.010hsx</td>
</tr>
<tr>
<td>Other masonry shear wall structures</td>
<td>I or II: 0.007hsx, III: 0.007hsx, IV: 0.007hsx</td>
</tr>
<tr>
<td>All other structures</td>
<td>I or II: 0.020hsx, III: 0.015hsx, IV: 0.010hsx</td>
</tr>
</tbody>
</table>

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

$0.025hsx = 0.025(10)(12) = 3.0” > 2.51” \therefore \text{drift O.K.}$

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

<table>
<thead>
<tr>
<th>Sheathing Grade</th>
<th>Common nail Size</th>
<th>Minimum Fastener Penetration In Framing Member or Blocking (in.)</th>
<th>Minimum Nominal Panel Thickness (in.)</th>
<th>Minimum Nominal width Of nailed face At adjoining Panel edges and boundaries (in.)</th>
<th>A Seismic Nail spacing (in.) at boundaries (all cases), at continuous panel edges parallel to load (cases 3 &amp; 4), and at all panel edges (cases 5 &amp; 6).</th>
<th>B Wind Panel Edge Fastener Spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OSB PLY</td>
<td>OSB PLY</td>
<td>OSB PLY</td>
<td>OSB PLY</td>
<td>OSB PLY</td>
<td>OSB PLY</td>
<td>OSB PLY</td>
</tr>
<tr>
<td>Sheathing and Single floor</td>
<td>8d 1-3/8</td>
<td>1-3/8</td>
<td>7/16</td>
<td>3</td>
<td>570 11 9</td>
<td>760 7 6</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>2</td>
<td>540 13 9.5</td>
<td>720 7.5 6.5</td>
<td>1060 11 8.5</td>
<td>1200 19 13</td>
</tr>
<tr>
<td></td>
<td>10d 1-1/2</td>
<td>15/32</td>
<td>2</td>
<td>580 25 15</td>
<td>770 15 11</td>
<td>1150 21 14</td>
</tr>
<tr>
<td></td>
<td>19/32</td>
<td>2</td>
<td>640 21 14</td>
<td>850 13 9.5</td>
<td>1280 18 12</td>
<td>1460 28 17</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>720 17 12</td>
<td>960 10 8</td>
<td>1440 14 11</td>
<td>1640 24 15</td>
<td>1010 1345 2015 2295</td>
</tr>
</tbody>
</table>
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 ∆a) to lessen rod elongation and wall rotation.
   - Increase the number of boundary studs (decrease bearing perpendicular to grain stresses, crushing).
   - Add additional interior shear walls to decrease forces on other shear walls.

d. **Calculation Method:** A final option which may increase the calculated system stiffness and reduce the deflections is to use the four-term deflection equation for the shear wall and diaphragm deflections to avoid introducing an artificial bias in the results by selectively combining three-term and four-term deflection calculations.
Solution for 2% drift issue:  

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.

\[
\delta_{\text{Diaph Unif}} = \frac{3vL'^3}{EAW'} + \frac{0.5vL'}{Gv_{tv}} + 0.376 L' e_n + \frac{\Sigma x \Delta_c}{W'}
\]

Where:

\[
e_n = \left(\frac{v_n}{769}\right)^{3.276} = \left(\frac{116.6}{769}\right)^{3.276} = 0.002 \text{ in}
\]

SDPWS Table C4.2.2D

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

\[
Gv_{tv} = 35000 \text{ lb/in depth, 4-ply}
\]

SDPWS Table C4.2.2A

\[
v = 233.2 \text{ plf}
\]

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

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

\[
\delta_M = \frac{C_d \delta_{\text{max}}}{L_e} = \frac{4(0.608)}{1} = 2.434 \text{ in.} \approx 2.4 \text{ in.}
\]

Close enough to comply with the 2% drift limitation. Drift can also be improved if \( \rho \) or \( Ax \) 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

Exposure C

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

$28.3.1$ Design wind pressure

$K_d=0.85$ Wind directionality factor 26.8
$GC_{pi}=+/-0.18 \left( \frac{2}{z} \right)$ Internal pressure coeff. 26.13
$K_z=2.01 \left( \frac{15}{z} \right)$ 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

<table>
<thead>
<tr>
<th>Surface</th>
<th>1</th>
<th>4</th>
<th>1E</th>
<th>4E</th>
</tr>
</thead>
<tbody>
<tr>
<td>GC_{pi}</td>
<td>0.4</td>
<td>-0.29</td>
<td>0.61</td>
<td>-0.43</td>
</tr>
<tr>
<td>$P$ (psf)</td>
<td>8.96</td>
<td>6.5</td>
<td>13.66</td>
<td>9.63</td>
</tr>
</tbody>
</table>

Parapet

$P_p=Q_p(GC_{pn})$

$K_z=0.85 @ 12'$ Top of parapet 28.3-2
$Q_p=24.46$ psf
$GC_{pn \; WW}=1.5, \; GC_{pn \; LW}=-1.0$

$P_{pw}=36.69$ psf, $P_{pl}=24.46$ psf
$\sum P_p=61.15$ psf

$2a=8'$
Rigid Diaphragm Analysis (ASD)

Wind \( V_{ulh} = 115 \) MPH

### Longitudinal Loading

<table>
<thead>
<tr>
<th>Grid Line</th>
<th>( kx )</th>
<th>( Ky )</th>
<th>( dx )</th>
<th>( dy )</th>
<th>( kd )</th>
<th>( kd^2 )</th>
<th>( Fv )</th>
<th>( Ft )</th>
<th>( Fv+Ft )</th>
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<td>130.63</td>
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<td>4923.8</td>
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<td>3</td>
<td>130.63</td>
<td>391.89</td>
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<td>153.8</td>
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<tr>
<td>B</td>
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<td>502.74</td>
<td>10054.72756</td>
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<td>-153.8</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( \Sigma )</td>
<td>87.09</td>
<td>50.27</td>
<td>J = 20893.23102</td>
<td>9847.6</td>
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<td></td>
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</table>

### Transverse Loading

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<th>( Ky )</th>
<th>( dx )</th>
<th>( dy )</th>
<th>( kd )</th>
<th>( kd^2 )</th>
<th>( Fv )</th>
<th>( Ft )</th>
<th>( Fv+Ft )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>43.54</td>
<td>3</td>
<td>130.63</td>
<td>391.89</td>
<td>4923.8</td>
<td>18.8</td>
<td>18.8</td>
<td></td>
<td></td>
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<tr>
<td>3</td>
<td>43.54</td>
<td>3</td>
<td>130.63</td>
<td>391.89</td>
<td>4923.8</td>
<td>-18.8</td>
<td>-18.8</td>
<td></td>
<td></td>
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<tr>
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<td>4851.4</td>
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<tr>
<td>( \Sigma )</td>
<td>87.09</td>
<td>50.27</td>
<td>J = 20893.23102</td>
<td>9847.6</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

Use this load combination for defining Nominal Stiffness values, \( Kf \). Then use those \( Kf \) values for all other analyses.

Expected Dead + Seismic D+Q (other terms if "expected" gravity loads as per AS-Dp=1.0, Ax=1.0)

<table>
<thead>
<tr>
<th>Grid Line</th>
<th>( SW )</th>
<th>( Ga )</th>
<th>( \gamma_{h} )</th>
<th>( V_{on wall} )</th>
<th>( v )</th>
<th>( T )</th>
<th>( C )</th>
<th>( \Delta_{a} )</th>
<th>( F_{r} )</th>
<th>( \phi ) Crush.</th>
<th>Shrink</th>
<th>( \delta_{B} )</th>
<th>( \delta_{S} )</th>
<th>( \delta_{Rot} )</th>
<th>( \delta_{SW} )</th>
<th>( K ) (k/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>37</td>
<td>1.0</td>
<td>1.0</td>
<td>913.5</td>
<td>6390.8</td>
<td>13770</td>
<td>0.154</td>
<td>556.36</td>
<td>0.056</td>
<td>0.019</td>
<td>0.022</td>
<td>0.247</td>
<td>0.313</td>
<td>0.581</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>37</td>
<td>1.0</td>
<td>1.0</td>
<td>913.5</td>
<td>6390.8</td>
<td>13770</td>
<td>0.154</td>
<td>556.36</td>
<td>0.056</td>
<td>0.019</td>
<td>0.022</td>
<td>0.247</td>
<td>0.313</td>
<td>0.581</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Total**

- **Shear wall \( p=1.3 \), \( Ax=1.25 \)**
- **Torsion, \( Ax=1.0 \)**
- **Flex/Drift \( p=1.0 \), \( Ax=1.25 \)**
- **Redundancy \( p=1.0 \), \( Ax=1.0 \)**

V equal to revised wall force based on HD STR (design) capacity 625 Max. Add stud 43.54

### Diaphragm Deflection (STR)

#### Rt. Cantilever

<table>
<thead>
<tr>
<th>Splice Forces (lbs.)</th>
<th>( \Sigma_{slip} )</th>
<th>( v ) unif.</th>
<th>( v ) conc.</th>
<th>( Ga ) k/in.</th>
<th>( L' ) Ft.</th>
<th>( W' ) Ft.</th>
<th>( \delta_{Diaph} ) Unf. In.</th>
<th>Diaph conv. In.</th>
<th>Total ( \delta ) In.</th>
</tr>
</thead>
<tbody>
<tr>
<td>F15</td>
<td>F23</td>
<td>F35</td>
<td>395.2</td>
<td>827.5</td>
<td>1980.6</td>
<td>0.041</td>
<td>115.55</td>
<td>25.0</td>
<td>15.00</td>
</tr>
<tr>
<td>1.75</td>
<td>3.66</td>
<td>8.76</td>
<td>0.00</td>
<td>25.0</td>
<td>35.00</td>
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<tr>
<td>8</td>
<td>16</td>
<td>24</td>
<td>0.008</td>
<td>0.009</td>
<td>0.014</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

**EA = 280500000 \times (2)\times 6**

includes effects of sw’s along chord line

**Method 2A**

<table>
<thead>
<tr>
<th>Diaphragm Deflection (STR)</th>
<th>333.6</th>
<th>886.1</th>
<th>1987.6</th>
<th>0.041</th>
<th>115.26</th>
<th>0.00</th>
<th>25.0</th>
<th>35.00</th>
<th>40.00</th>
<th>0.135</th>
<th>0.00</th>
<th>0.135</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.48</td>
<td>3.92</td>
<td>8.79</td>
<td>0.007</td>
<td>0.009</td>
<td>0.014</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**E = 34’**
Wind Design (ASD) Drift-Similar to Method 2A  0.6D+0.6W

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

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

Drift $\Delta_T = 0.26''$

Drift $\Delta_1 = 0.237''$

$\delta_M = \frac{C_d \delta_{max}}{e} = \frac{4(0.26)}{1} = 1.04$ in. < ?? in.

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 $\Delta_4 = 0.26” < 0.3”$
∴ 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
Part 4-Design Example (cont.):

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

Diagram showing torsional irregularities with labels for C.M., C.R., δ_A, and δ_B.

Typical Floor Plan

Torsional Irregularity?
Verify Torsional Irregularity

Seismic- $\rho=1.0$, $Ax=1.0$
**Torsional Irregularities** $\rho = 1.0$ and $A_x = 1.0$

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

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

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

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

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

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

- An extreme torsional irregularity Type 1b is allowed in structures assigned to Seismic Design Categories B, C, and D, but not in SDC E, or F.
ASCE 7-16: Table 12.3-1 Horizontal Structural Irregularity Requirement References

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

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

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

- 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

<table>
<thead>
<tr>
<th>Grid Line</th>
<th>kx</th>
<th>Ky</th>
<th>dx</th>
<th>dy</th>
<th>kd</th>
<th>kd²</th>
<th>Fv</th>
<th>Ft</th>
<th>Fv+Ft</th>
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<td>-1624.7</td>
<td>-1624.7</td>
<td></td>
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<td>20</td>
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<td>10054.73</td>
<td>1624.7</td>
<td>1624.7</td>
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### Diaphragm Deflection (STR) ρ=1.0, Ax=1.0

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<tr>
<th>Splice Forces (Lbs.)</th>
<th>Σδ_slip</th>
<th>v unif.</th>
<th>v conc.</th>
<th>Ga</th>
<th>L'</th>
<th>W'</th>
<th>δDiaph Unif</th>
<th>δDiaph conc</th>
<th>Total δ</th>
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<td>3542.8</td>
<td>0.075</td>
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<td>35.00</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Nails Req'd= 4.35 5.47 15.68
Use Nails = 8 16 24
Slip= 0.021 0.013 0.025

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

### Torsional Irregularity Check-Method 2A

<table>
<thead>
<tr>
<th>Diaphragm Deflection (STR)</th>
<th>Lft. Cantilever</th>
</tr>
</thead>
<tbody>
<tr>
<td>332.0</td>
<td>1855.1</td>
</tr>
<tr>
<td>0.073</td>
<td>224.42</td>
</tr>
</tbody>
</table>

| 1.47                      | 8.21            | 16.01   |
| 8                         | 16              | 24      |
| 0.007                     | 0.020           | 0.026   |

Walls at Grid Corridor lines A & B

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Torsion (Question 7):
Check for Torsional Irregularity Type 1a - $\rho=1.0, \Delta x=1.0$

SDPWS 4.2.5.2 (2):

- A.R. $\leq 1:1$ if torsional irregularity - one-story structure
- A.R. $= 0.67:1$ - multi-story structure

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

$\Delta_2 = 0.194''$, $\Delta_3 = 0.214''$

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

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

$\delta_{RL} = \frac{2\delta_{SWAB}(L' + 3')}{W'} = 0.124''$ Vertical component of rotation

Diaphragm deflections:

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

$\delta_{D,4} = 0.260''$
Drift $\Delta = \sqrt{(\delta_T + \delta_D \pm \delta_{RL})^2 + (\delta_{RT})^2}$

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

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

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

$0.592 > 1.2(0.467) = 0.56''$, \(\therefore\) Horizontal torsional irregularity Type 1a does exist in this direction.

$0.592 < 1.4(0.467) = 0.654''$, \(\therefore\) Horizontal torsional irregularity Type 1b does not exist in this direction.
Amplification of Accidental Torsion

Seismic - $\rho=1.0$, $Ax=1.0$
Verify Amplification of Accidental Torsion, Ax

Seismic- ρ=1.0, Ax=1.0
ASCE 7-16 12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M\text{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

\[ A_x = \left( \frac{\delta_{\text{max}}}{1.2\delta_{\text{avg}}} \right)^2 \]

Where

\[ \delta_{\text{max}} = \text{maximum displacement at level x computed assuming } A_x = 1 \]
\[ \delta_{\text{avg}} = \text{average of the displacements at the extreme points of the structure at level x computed assuming } A_x = 1. \]

\[ M\text{ta} = \text{accidental torsional moment} \]

From torsion section:

\[ A_x = \left( \frac{\delta_{\text{max}}}{1.2\delta_{\text{avg}}} \right)^2 = \left( \frac{0.592}{1.2(0.467)} \right)^2 = 1.116 < 1.25 \text{ assumed.} \]

\[ \therefore \text{Can recalculate if desired.} \]

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

- The application of rho relates directly to increasing the capacity of the walls only, or adding more walls.
- The rho factor has an effect of reducing R, for less redundant structures which increases the seismic demand.
- Shear wall systems have been included in Table 12.3-3 so that either an adequate number of walls are included, or a proper redundancy factor has been applied.
12.3.4.1 Conditions Where Value of \( \rho \) is 1.0. The value of \( \rho \) is permitted to equal 1.0 for the following:

2. Drift calculation and P-delta effects.

5. Design of collector elements, splices, and their connections for which the seismic load effects including over-strength factor of section 12.4.3 are used.

6. Design of members or connections where seismic load effects including over-strength factor of section 12.4.3 are required for design.

7. Diaphragm loads, \( F_{px} \), determined using Eq. 12.10-1, including min. & max. values.

12.3.4.2 Redundancy Factor, \( \rho \), for Seismic Design Categories D through F.

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

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

Let’s check condition b. first
b. Structures that are regular in plan at all levels p=1.0 provided:

- SFRS consist of at least two bays of perimeter SFRS framing on each side of the structure in each orthogonal direction at each story resisting more than 35% of the base shear.
- The number of bays for a shear wall = 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**.

**Table 12.3-3.**
- Removing one wall segment with A.R. > 1:1 will not result in reduction in story strength > 33% limit.
- Removing 1 wall within any story will not result in extreme torsional irregularity, Type 1b.
\[ \Delta \text{Drift} = \delta_T + (\delta_D - \delta_{RL}) \]

\[ \delta_T = \Delta ADVE \]

\[ \frac{H}{L} = \frac{10}{8} = 1.25 > 1.0 \]

\[ \therefore \text{remove} \]

231.22 plf

236.38 plf

\[ \text{Diaphragm deflection} \]

\[ \rho = 1.0 \quad Ax = 1.0 \quad \bar{Y} = \frac{15.875(40)}{3(15.875)} = 13.33' \]

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Redundancy Study

Spreadsheet results

<table>
<thead>
<tr>
<th></th>
<th>FA</th>
<th>FB</th>
<th>F2</th>
<th>F3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total</td>
<td>1595</td>
<td>1595</td>
<td>8263</td>
<td>9506</td>
</tr>
</tbody>
</table>

Check

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

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

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

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

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

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

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 \textit{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:

\textbf{Shall be the maximum of:}

\begin{align*}
\Omega_o F_x & \quad \text{- Forces determined by ELF Section 12.8 or Modal Response Spectrum Analysis procedure 12.9} \\
\Omega_o F_{px} & \quad \text{- Forces determined by Diaphragm Design Forces (Fpx), Eq. 12.10-1 or} \\
F_{px}\text{min} & = 0.2S_{DS}I_{e w_{px}} \quad \text{- Lower bound seismic diaphragm design forces determined by Eq. 12.10-2 (Fpx_{min}) using the Seismic Load Combinations of section 12.4.2.3 (w/o over-strength) - do not require the over-strength factor.} \\
F_{px}\text{max} & = 0.4S_{DS}I_{e w_{px}} \quad \text{- 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.}
\end{align*}

\textbf{Exception:}

1. In structures (or portions of structures) \textit{braced entirely by light framed shear walls}, collector elements and their connections, including connections to vertical elements need only be designed to resist forces using the standard seismic force load combinations of Section 12.4.2.3 with forces determined in accordance with Section 12.10.1.1 (Diaphragm inertial Design Forces, \(F_{px}\)).
ASD, $\rho=1.3$, $A_x=1.25$

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

If center SW removed, strut forces are increased

Diaphragm Shears

0.141 k 3.53 plf
7.051 k 176.3 plf

0.961 k 24 plf

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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
\( p = 1.0, \ Ax = 1.25 \)

Torsional and Redundancy Check
\( p = 1.0, \ Ax = 1.0 \)
Diaphragm Flexibility, Resulting numbers: \( \rho=1.0, \ A_x=1.25 \)

\[ W= \frac{17769}{76}=444.1 \text{ plf (ASD)} \]

\[ V_A=9057.6 \text{ lbs.} \]

\[ V_{\text{max Diaph}} = \frac{9057.6}{76} = 119.2 \text{ plf} < 464 \text{ plf} : \text{ O.K} \]

From spreadsheet (STR)

\( \delta_{\text{Diaph}} = 0.066" \)

\[ \Delta_{SW\ A} = 0.396", \ A_{SW\ B} = 0.311", \ 2x\Delta_{\text{Average}} = 0.707" \]

\( 0.066" < 0.707" : \text{ Rigid diaphragm, as initially assumed.} \)

Check Story Drift

\( \rho =1.0 \text{ and } A_x = 1.25 \)

\[ C_d = 4, \ I_e = 1 \]

\[ \delta_{SW\ A} = 0.396 \text{ in} \quad \text{from spreadsheet} \]

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

\( 0.020 \ h_{sx} = 0.020(10)(12) = 2.4 \text{ in} > 1.58 \text{ in}, : \text{ Drift OK} \)
Check for Torsional Irregularity \( \rho=1.0, A_x=1.0 \)

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

From spreadsheet

\( \delta_{SWA} = 0.387'' \)

\( \delta_{SWB} = 0.319'' \)

\[ \Delta_{Average} = \frac{0.387 + 0.319}{2} = 0.353'' \] From spreadsheet

\( 0.387 < 1.2(0.353) = 0.424'' \), \( \therefore \) 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**, \( A_x = 1.121 \)

• Horizontal irregularity Type 1b does not occur in either direction. **Correct**, however, when checking redundancy, it occurs in the transverse direction by the removal of 1 wall.

• No redundancy in both directions, \( \rho = 1.3 \) **Incorrect:**
  - \( \rho = 1.0 \) Longitudinal
  - \( \rho = 1.3 \) Transverse

Other Design Requirements:

• Drift < allowable
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/](http://www.woodworks.org/education/online-seminars/)


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

• Shiotani/Hohbach Method - Woodworks Slide archive

• FPI Innovations - 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

• FPI Innovations - 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 FPI Innovations 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.

\[ \sum M_i H_i^2 + \sum V_i (H_i^3) \] 

\[ \frac{2}{(EI)_i} \]

\[ \frac{3}{(EI)_i} \]

\[ \Delta \text{Allow story drift} \]

Total displ. of Tall Wall. More flexible.

A.R. = 3.5:1 flr.-flr.

A.R. = 2:1 flr.-flr.

Acting as a continuous wall - Tall SW

Stiffness based approach

MBA based on stiffness

Not in example
Diaphragm out-of-plane Flexibility

Semi-balloon framed (Very flexible)

If diaphragm out-of-plane stiffness = Flexible
Analyze entire wall as a tall wall

Should consider as flexible because it is unknown where rim joist splices will occur

Compression blocking

Platform framed

If diaphragm out-of-plane stiffness = Rigid (steel beam, conc. beam)
Analyze entire wall as traditional floor to floor
Tall Wall Deflection  

\[
\Delta_i = \frac{\sum M_i H_i^2}{2(El)_i} + \frac{\sum V_i (H^3)}{3(El)_i} + \frac{V_i H_i}{Gv_i \ell v_i} + 0.75H_i e_{n,i} + \frac{H_i}{l_i} d_{a,i} + H_i \sum_{j=1}^{i-1} \left( \frac{M_j H_j}{(El)_j} + \frac{V_j H_j^2}{2(El)_j} \right) + H_i \sum_{j=1}^{i-1} \frac{d_{a,j}}{L_j}
\]

\( \theta_1 (H_2 + H_3 + H_4 + H_5) + \alpha_1 \left( \frac{H_1 + H_2 + H_3 + H_4 + H_5}{L_i} \right) \)

\( \theta_1 (H_2 + H_3) + \alpha_1 \left( \frac{H_1 + H_2 + H_3}{L_i} \right) \)

\( \theta_1 (H_2) + \alpha_1 \left( \frac{H_1 + H_2}{L_i} \right) \)

\( \theta (Wall\ rotation) \)

\( \Delta_1 \)

\( \Delta_2 \)

\( \Delta_3 \)

\( \Delta_4 \)

\( \Delta_5 \)

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

- Woodworks Presentation Slide Archives-Workshop-Advanced Diaphragm Analysis
- SEAOC Seismic Design Manual, Volume 2
- Woodworks-The Analysis of Irregular Shaped Diaphragms (paper). Complete Example with narrative and calculations.
  
- Woodworks-Guidelines for the Seismic Design of an Open-Front Wood Diaphragm (paper). Complete Example
Method of Analysis and Webinar References

Offset Diaphragms

Offset Shear Walls

Diaphragms Openings

Shear Walls with Openings

Mid-rise Design Considerations

Information on Website: Presentation Slide Archives, Workshops, White papers, research reports

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

Example Results

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

https://vimeo.com/woodproductsCouncil/review/212986898/17ca94ef6f

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

https://vimeo.com/woodproductsCouncil/review/220727334/516f37ce1e
Questions?

This concludes Woodworks Presentation on:
Guidelines for the Seismic Design of an Open-Front Wood Diaphragm

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

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Thank You

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