UPDATED April 2019

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WOOD PRODUCTS COUNCIL

DESIGN EXAMPLE

A Design Example of a Cantilever Wood Diaphragm

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1 Overview

Complex building shapes and footprints are driving design procedures and code requirements to evolve for all lateral force-resisting systems and materials. As buildings get taller and more complex, there is a greater need to understand the relative stiffness of diaphragms and shear walls, and multi-story shear wall effects. Architecturally demanding exterior wall lines in modern structures do not always provide opportunities to use traditional design approaches.

In mid-rise, multi-family buildings, corridor-only shear wall floor plans, similar to the plan shown in *Figure 1(d)*, are becoming a popular design approach. Low-rise retail buildings, such as the ubiquitous strip mall, are another building type where open-front diaphragms are frequently employed.

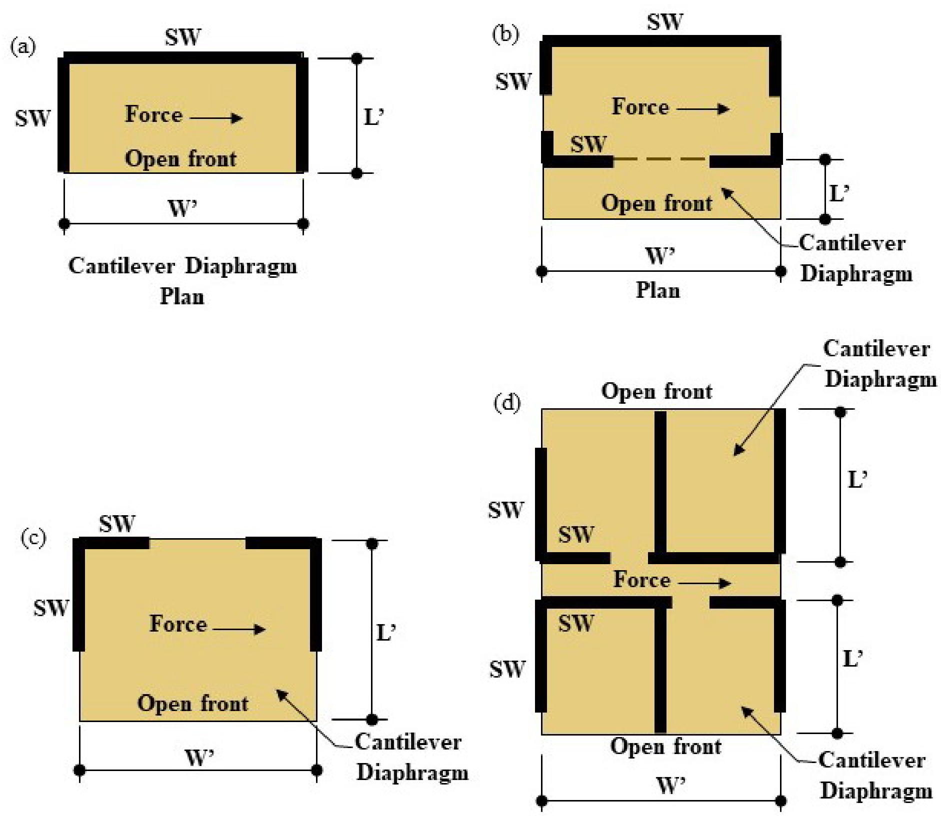


Figure 1. Examples of Open-Front Structures, following Special Design Provisions for Wind & Seismic (SDPWS) Figure 4A

The goal of this paper is to provide an example of how to analyze a single-story structure with a double cantilever diaphragm and help engineers better understand the code and standards issues associated with these types of structures. Limiting it to one story simplifies the example while allowing a comprehensive explanation of an open-front diaphragm design. This method of analysis can also be applied to multi-story structures. A secondary goal is to address common questions about open-front diaphragms, including:

- What is the deflection equation for cantilever diaphragms?

- How is diaphragm flexibility defined for cantilever diaphragms?
- What is the proper method of distributing torsional forces into the diaphragm?
- Do shear walls located along diaphragm chord lines affect the diaphragm chord forces?
- 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?
- How are torsional irregularities determined and addressed for cantilever diaphragms?

This example demonstrates that compliance with code requirements can require balancing the design between the various elements of the lateral force-resisting system to achieve the required structural stiffness. The example is for information purposes only, is not intended to define or authoritatively interpret requirements, and does not represent the only method of analysis available. All of the results and conclusions in this paper are related to this example only. The results for other structures of a similar nature will vary.

2 Codes and Standards Basis of Design

This example is based upon an engineered design using the 2018 International Building Code (IBC 2018)[1], and the following referenced standards:

- ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, (ASCE 7-16)[2]
- American Wood Council, Special Design Provisions for Wind & Seismic 2015 Edition, (SDPWS 2015)[3]
- American Wood Council, National Design Specifications for Wood Construction 2018 Edition (NDS 2018)[4]

3 Example Structure

The floor plan of this example is shown in *Figure 2*. The plan is symmetric about both axes. Such a floor-plan could be similar to a four-unit office or residential building.

The example building is a one-story structure with 10-foot-high walls plus a two-foot parapet. The roof framing is supported off the walls using one of the semi-balloon framing schemes shown in *Figure 3.* Two eight-foot-long shear walls are placed along grid lines A and B as shown. These shear walls have intentionally been reduced in length from what may normally occur to demonstrate the effects of diaphragm and shear wall stiffness on drift and torsional irregularity issues, and to show their effects on the diaphragm chord forces. Three 10-foot-long shear walls are placed along corridor wall lines 2 and 3. Complete loads paths must be established from the diaphragm sheathing down into the shear walls. If the roof framing is oriented perpendicular to the corridor wall line and it is platform framed, shear blocking panels or blocking can be installed over the corridor wall lines to transfer the diaphragm shears down into the corridor shear walls. If the framing members are oriented parallel to the corridor walls, roof framing members can be used to transfer the diaphragm shears down into the shear walls, or a direct connection of the diaphragm to the shear walls can be made similar to *Figure 3*. The exterior walls that occur at grid lines 1 and 4 do not have enough stiffness to act as shear walls creating the open-front structure/cantilever diaphragm condition. For plan north-south lateral loads, the diaphragm cantilevers both directions from the corridor wall lines. In this example, the plan north-south direction is labelled the longitudinal direction, with the loads applied parallel to the corridor walls. This naming choice is selected as the most similar in configuration to multi-family floor plans that have many more than four units and are much longer in the direction of the corridor compared to the width of the building. The diaphragm is a simple span between lines A and B for transverse lateral loads. The selected direction of the roof joists runs parallel to the corridor walls with the roof sheathing installed perpendicular to the joists. A bearing wall line is located at the middle of the building to reduce the roof framing spans. The diaphragm wood structural sheathing panel lay-up is Case 1 for the longitudinal loading and Case 3 for the transverse loading as depicted at the bottom of SDPWS Table 4.2A.

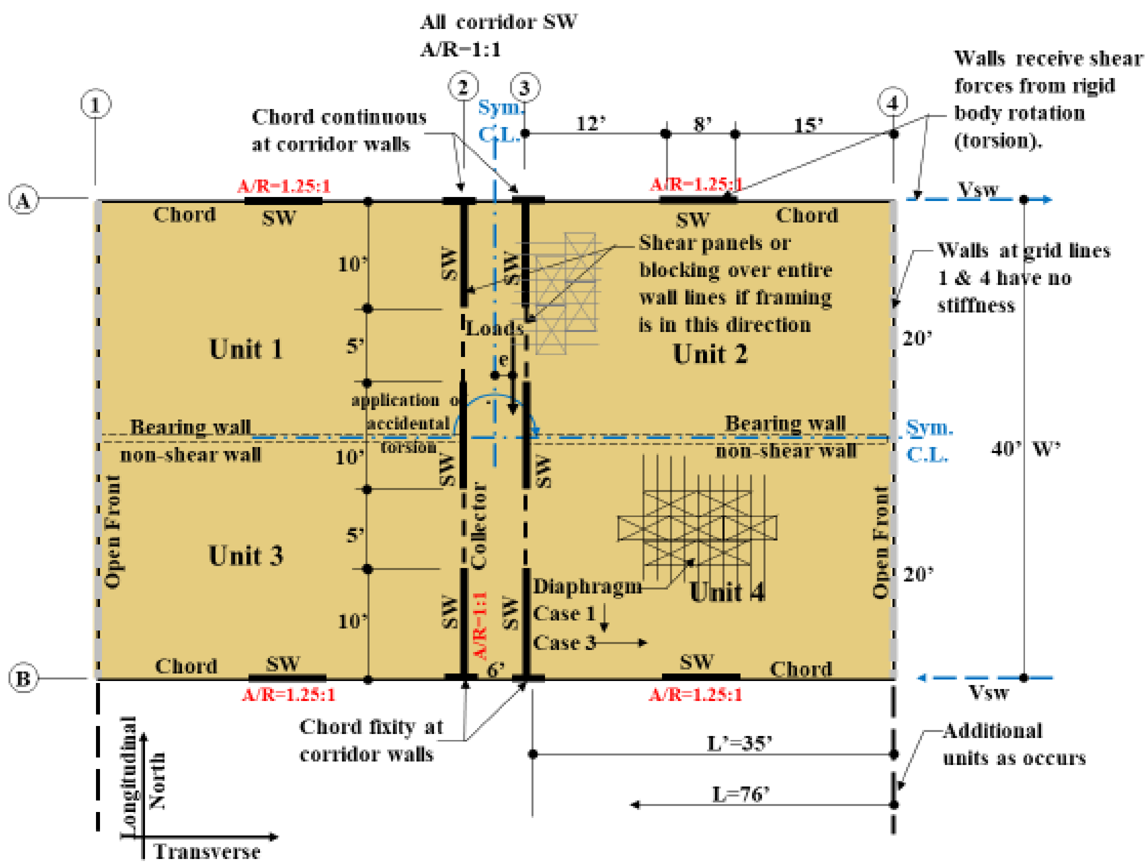


Figure 2. Typical Floor Plan

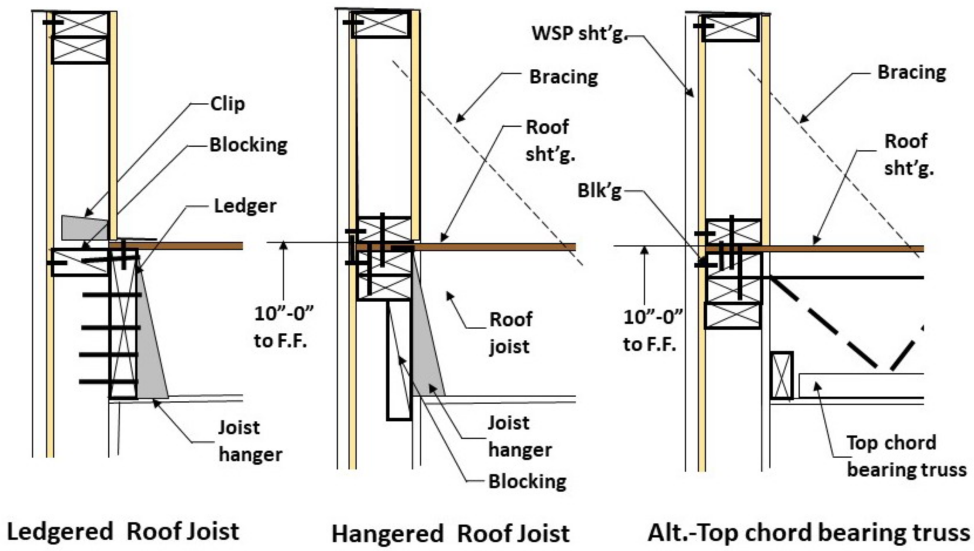


Figure 3. Possible Wall Details at Roof

All shear walls comply with the allowable height to width, (h/b), aspect ratios of 2:1 or less as discussed in SDPWS Section 4.3.4 and Table 4.3.4. If aspect ratios are greater than 2:1, the shear capacity must be reduced for high-aspect ratio walls in accordance with Section 4.3.4.

4 Lateral Design Considerations and Calculations – Seismic

This example currently only examines seismic loading and requirements in detail. Some wind requirements are briefly covered in *Section 7.4.2.*

4.1 Building Design and Loading Information

Occupancy Category B – Office
Construction Type VB – Light wood framing

Given:

a. Framing (NDS):
   Roof – Douglas-fir-larch (DFL), No. 1, E = 1,700,000 psi, joist framing @ 16 in o.c.
   Walls – DFL No. 1, E = 1,700,000 psi, wall studs @ 16 in o.c.

b. Design Loads:
   Calculated dead loads can vary depending on framing, the use of brick veneer or stucco, whether there's a ballasted roof, energy requirements, fire-resistance requirements, and finishes. The calculations below are assumed.

   Dead Loads

Roofing + insulation	8.0 psf
Wood structural panel	

(WSP) sheathing (OSB)	2.0 psf	
Framing	3.5 psf	
GWB ceiling + Misc.	3.2 psf	
Sprinklers	2.0 psf	
Misc., mechanical	2.5 psf	
	21.2 psf	Say 25 psf

Seismic Load

Roof Dead Load	25.0 psf	
Walls*	10.0 psf	Parapet plus interior partitions
	35.0 psf	

Live load 20 psf

Snow load 25 psf

*This is the seismic weight of the walls per floor area. The in-plane wall weight is assumed to be equal to 13 psf.

4.2 Calculate Main Lateral Force-Resisting System (MLFRS) Seismic Forces

Base shear per ASCE 7-16 Section 12.8 Equivalent Lateral Force Procedure, F_x:

Risk category II — Table 1.5-1

Importance factor, $I_e = 1.0$ — Table 1.5-2

Using the USGS Seismic Design Maps Web Tool, 2015 NEHRP Recommended Seismic Provisions[5], adopted into the 2016 ASCE 7 Standard and 2018 International Building Code:

Location: Tacoma, Washington, site coordinates 47.255° N, 122.442° W

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 ft

Approx. fundamental period, $T_a = C_t h_n^x = 0.02(10)^{0.75} = 0.113$ s — Eq. 12.8 – 7

where $x = 0.75$, $C_T = 0.02$ and $h_n = 10$ ft — Table 12.8.2

$S_{DS} > 0.4 \therefore C_u = 1.4$ — Table 12.8-1

Max. fundamental period $T = C_u T_a = 1.4(0.113) = 0.158$ s $< T_L = 6$ — Section 12.8.2

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{1.084}{\left(\frac{6.5}{1}\right)} = 0.167 \text{ controls} \qquad \text{Eq. } 12.8-2$$

$T < T_L = 6$ Section 11.4.6 and Figure 22-14

$$C_{s\,max} = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.571}{0.113\left(\frac{6.5}{1}\right)} = 0.777 \qquad \text{for } T \leq T_L \qquad \text{Eq. } 12.8-3$$

$$C_{s\,min} = 0.044 S_{DS} I_e = 0.044(1.084)1 = 0.048 \geq 0.01 \qquad \text{Eq. } 12.8-5$$

$S_1 < 0.6$ g ∴ Eq. 12.8.6 does not apply Eq. 12.8-6

The resulting seismic base shear is:

$V = C_s W = 0.167(35\text{psf})(76\text{ ft})(40\text{ ft}) = 17{,}769$ lbs Eq. 12.8-1

As a single-story building, the vertical distribution of seismic forces per ASCE 7 12.8.3 is simply:

$F_x = F_y = V = 17{,}769$ lbs

This is equivalent to a distributed lateral load of 5.84 psf over the roof area.

This example follows the common, but not required, practice of using allowable stress design (ASD) for the force capacity design of the shear walls and diaphragms. Strength-level forces are used for shear wall and diaphragm deflections, story drift, and torsional irregularity checks.

4.3 Preliminary Assumptions of Seismic Design

As with any design process, certain assumptions need to be made before design and analysis can proceed. Preliminary assumptions relevant to this design example include diaphragm flexibility, structural irregularities as they relate to modifying the seismic design loads, and structural redundancy.

4.3.1 Diaphragm Flexibility

In this example, a rigid diaphragm assumption will be used for the initial horizontal seismic load distribution and shear wall design. In *Section 7.4*, the acceptability of this assumption will be evaluated.

4.3.2 *Structural (Torsional) Irregularities*

The center of rigidity and center of mass for this example plan occur at the same location; therefore, there is no inherent torsion. Inherent and accidental torsion are only considered for diaphragms that are not flexible. Accidental torsion, as defined by ASCE 7-16 Section 12.8.4.2, is an additional torsion force that is applied to the structure due to inaccuracies or uncertainties inherent in the design. To calculate the accidental torsion, the center of mass is assumed to be displaced from its calculated position by a distance equal to 5% of the dimension of the structure in the perpendicular to the direction of the applied force. In ASCE 7-16 the accidental torsion is applied in all buildings for determining whether a horizontal irregularity exists (e.g., torsional irregularity); but it need not be included in the structural design forces except when a torsional irregularity exists. In buildings with inherent torsion, the combined effect of accidental torsion and inherent torsion should be considered.

In ASCE 7-16 the following was added to Section 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. Accidental torsion moments (M_{ta}) need not be included when determining the seismic forces E in the design of the structure and in the determination of the design story drift in Sections 12.8.6, 12.9.1.2, or Chapter 16, or limits of Section 12.12.1, except for the following structures:

> *1. Structures assigned to Seismic Category B with Type 1b horizontal structural irregularity*
> *2. Structures assigned to Seismic Category C, D, E, and F with Type 1a or Type 1b horizontal structural irregularity*

Unlike ASCE 7-10, this addition states the accidental torsion moment does not need be in the *design forces* of structures which are torsionally regular. The accidental torsion moment does need to be used for the torsional irregularity checks.

For open-front structures, the classification of the structure as having a torsional irregularity (Type 1a) or an extreme torsional irregularity (Type 1b) is especially important. Per ASCE 7 Section 12.3.3.1, structures assigned to SDC E and F are prohibited from an extreme horizontal torsional irregularity (Type 1b). As the example structure is assigned to SDC D, this prohibition does not apply. If the structure has a Type 1a or Type 1b horizontal irregularity, the amplification of the accidental torsion of ASCE 7 Section 12.8.4.3 can impact the design of components of the structure.

The preliminary estimate of building regularity for this example structure per ASCE 7 Table 12.3-1 is:

- A torsional irregularity (horizontal irregularity Type 1a) occurs in longitudinal direction but not the transverse direction due to symmetry of the layout and absence of cantilevers.
- No extreme torsional irregularity (horizontal irregularity Type1b) occurs in longitudinal or transverse directions.

Based on this estimate, design loads for the shear walls and other elements in the structure in the longitudinal direction are to be increased by a torsional amplification factor, Ax, in accordance with ASCE 7 Section 12.8.4.3. The required level of torsional amplification depends on calculated deflections and the degree of torsional irregularity. To expedite the design example and minimize iterations, an initial estimate of Ax for the longitudinal direction is estimated:

$$A_x = \left(\frac{\delta_{Max}}{1.2\delta_{Avg}}\right)^2$$

$A_x = 1.25$

In practice, it can be cumbersome to perform a design based on loads, calculate deflections based on the design, and then perform a torsional irregularity check and realize you need to include or increase the amplification of accidental torsional moment. The authors sympathize and have been there as well. To address this conundrum of design, nothing is more valuable than experience with similar structures.

For the final design, *Section 7.6* will verify the presence of torsional irregularities and *Section 7.6.1* will calculate any required amplification of accidental torsional moment.

For structures assigned to SDC D, E, and F, ASCE 7-16 Section 12.3.4 requires a redundancy factor of ρ = 1.3 unless a condition to justify ρ = 1.0 is met. There are several approaches that can be made regarding the assignment of ρ. Some engineers default ρ = 1.3 to avoid additional calculations and neglect verifying redundancy at the end of the design. While more expedient, this can lead to more conservative designs than required. Others will assume ρ = 1.3 during preliminary design and verify the required value of ρ near the end of the design to see if the design forces can be reduced. Another approach would be to assume that the structure has enough redundancy, setting ρ = 1.0 and verifying that assumption as the design progresses.

ASCE 7-16 Section 12.3.4.1 also allows ρ to be set to 1.0 under certain conditions, including:

- *Drift calculation and P-delta effects*
- *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*
- *Design of members or connections where seismic load effects including over-strength factor of section 12.4.3 are required for design*
- *Diaphragm loads determined using Eq. 12.10-1, including the limits imposed by Eq. 12.10-2 and 12.10-3*

4.3.3 Redundancy

Because the structure is assigned to SDC D, the use of ρ = 1.3 is required unless the conditions in ASCE 7 Section 12.3.4.2 are met to justify ρ = 1.0. Based on experience (or prior calculations), it is estimated that the structural layout in *Figure 2,* will not qualify for 1.0; therefore, the redundancy factors used where applicable are:

$\rho_L = 1.3$
$\rho_T = 1.3$

Verification of redundancy is presented in *Section 7.8*.

Figure 4 shows several examples of possible cantilever diaphragm structures. Plans A and B have an abundance of low aspect ratio shear walls. This suggests there may be sufficient redundancy to qualify for ρ = 1.0 and torsion and drift may not be issues. For such plans, it may be expedient to start with a preliminary assumption of ρ = 1.0 and no torsional irregularities, and to verify later in the design process.

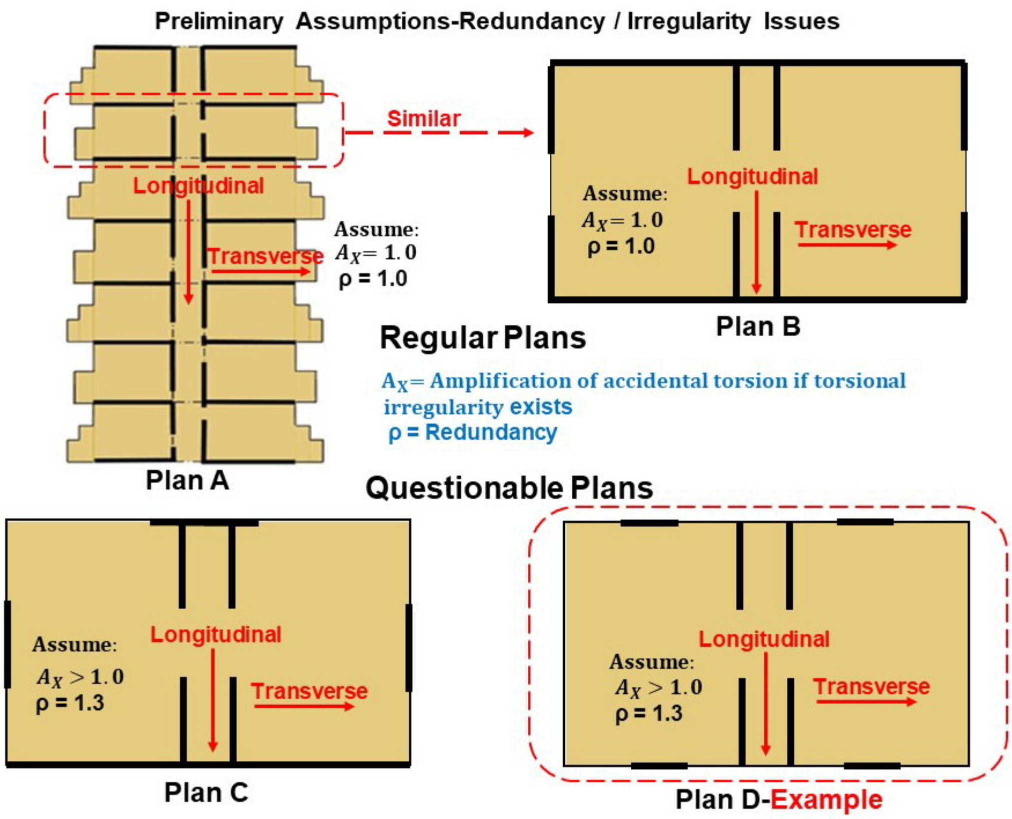


Figure 4. Examples of Cantilever Diaphragm Structures

Plan C is non-symmetrical with shorter shear walls at one of the diaphragm chord lines, raising the possibility that torsion could be an issue. Although plan D is symmetrical, the minimal shear walls suggest that drift, redundancy and torsion may be issues. For this condition, it would be conservative to assume Rho (ρ) = 1.3 and that limited wall could lead to a torsional irregularity. When a torsional irregularity is assumed, accidental torsion must be applied and amplified. The apparent lack of redundancy and questionable stiffness would require more engineering judgement and/or preliminary assumptions at the onset of the design. Regardless of which method is used, it is important to remember that, in some cases, the design of the shear walls and diaphragm cannot be based on strength alone as story drift values may govern the design.

5 Shear Wall Design based on Rigid Diaphragm Analysis

In a rigid diaphragm analysis (RDA), distribution of loads to the walls depends on the location and stiffness of the walls. The stiffness of the walls depends on the construction details of the walls. Unlike in flexible diaphragm analysis, the loads to a wall line change with changes in the construction details of the shear wall. This creates a design process that is often inherently iterative.

The outline of the design used for this example for the longitudinal direction is as follows:

1. Perform an initial rigid diaphragm analysis based on using stiffness proportional to the wall lengths (*Sections 5.1* and *5.2*).
2. Create (or update) the shear wall designs and details based on demand and adjust as required to account for anticipated drift limitation issues (*Section 5.3*).
3. Calculate the shear wall stiffnesses using shear wall design details (*Section 5.4).*
4. Perform revised rigid diaphragm analysis using updated wall stiffnesses (*Section 5.5).*
5. Verify shear wall designs with loads from revised RDA (*Section 5.6).*
6. Calculate diaphragm design forces (*Section 6)* including torsional forces if required (*Section 7.1).*
7. Design the diaphragm (*Section 7.2).*
8. Verify assumption of rigid diaphragm behavior for horizontal distribution of forces (*Section 7.4)*
9. Check story drift limits (*Section 7.5).*
10. Verify presence of torsional irregularities (*Section 7.6).*
11. Verify redundancy factor (*Section 7.7).*

The initial shear wall and diaphragm designs can be undertaken in any order. It is most effective to check the requirements that govern the designs first, and adjust as necessary based on engineering judgement and experience.

5.1 Preliminary Estimate of Wall Stiffnesses

The shear wall layout in *Figure 2* is the starting point for the wall design. For this example, the initial RDA and wall design process will start with the assumption that all of the walls are of similar, but unknown, WSP-sheathed construction. For the initial RDA, using stiffness values proportional to the wall length or capacity (as per SDWPS 2015 4.3.3.4.1, Exception 1) is often an expedient starting point. Since there are no walls that have an aspect ratio greater than 2:1, no reduction in shear capacities are required. Also, all the walls in each line of lateral force-resistance are of the same length and assumed stiffness:

SW 2 and 3: $K_x = 0$, $K_y = 10$
SW A and B: $K_x = 8$, $K_y = 0$

In the initial RDA, only the relative stiffness between the walls is important to distribute lateral load between the walls.

5.2 *Initial RDA*

Per ASCE 7-16 Section 12.8.4.2, the accidental torsion moment needs to be applied to the longitudinal direction design forces if the structure is in SDC D and has a Type 1a horizontal irregularity, as previously assumed.

Based on the assumed stiffness values and assumptions $\rho_L = 1.3$, $A_x = 1.25$, an initial RDA is performed, using a spreadsheet tool to expedite the calculations. For these calculations, the origin is assigned to be at the intersection of the lines of symmetry.

$F_x = F_y = 17{,}769$ lbs, adjusted for redundancy and ASD = 1.3(17,769)(0.7) = 16,170 lbs

Accidental torsion for longitudinal loading:

$e_x = 0.05$ (76 ft) = 3.8 ft accidental torsion, amplified = 1.25(3.8) = 4.75 ft

$T_y = A_x\, e_{x\,\rho}\, F_y$ = (1.25) (3.8 ft) (1.3)(0.7)17,769 lbs = 76,806.5 ft lbs

Rigid diaphragm analysis equations:

$F_V = F_y \frac{k_y}{\sum k_y}$ The direct shear to the walls (based on their stiffness)

$T = F_y e_x$ Torsional moment in the longitudinal direction

$J = \sum k_y d_x^2 + k_x d_y^2$ Polar moment of inertia

$F_T = T\frac{kd}{J}$ Torsional force to walls

$F_{sw} = F_V + F_T$ Sum of direct shear and torsional force to walls

Calculated forces to the shear walls from the initial RDA by wall length are shown in *Table 1*.

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.	FT 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.	FT 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

Table 1. Initial RDA Distribution of Loads Based on Wall Lengths – ASD

5.3 Initial Shear Wall Design (ASD)

For the initial shear wall design, the seismic load effects of ASCE 7-16 Sections 12.4.2 and load combinations in Section 2.4.5 are followed for allowable stress design (ASD) using ρ = 1.3, A_x = 1.25.

An important consideration when designing shear walls is the addition or omission of vertical loads to resist overturning and uplift, which can have a significant impact on horizontal shear wall deflections and stiffness, even for short buildings.

Ignoring dead loads to resist over-turning forces is sometimes done in practice, as most experimental shear wall tests have not included gravity loading. Ignoring or including them is subjective and a matter of preference or engineering judgement.

This example will consider the use of gravity loads to resist overturning. As seen in *Figure 2,* the roof framing runs in the longitudinal direction applying a significant tributary area gravity load to grid lines A/B and a small tributary area to the corridor walls along grid lines 2 and 3.

ASCE 7-16 Section 2.4.5 – Seismic Load Combinations – Allowable Stress Design:

$L = 0$

$E_v = 0.2S_{DS}D$

$E_h = \rho Q_E$

8. $1.0D + 0.7(0.2S_{DS})D + 0.7\rho Q_E = 1.152D + 0.7\rho Q_E$
9. $1.0D + 0.525(0.2S_{DS})D + 0.525\rho Q_E + 0.75S = 1.114D + 0.525\rho Q_E + 0.75S$
10. $0.6D – 0.7(0.2S_{DS})D + 0.7\rho Q_E = 0.448D + 0.7\rho Q_E$

The load combination producing the greatest effect should be used. The design of tension and compression boundary elements have not been included in this part of the analysis, nor are they included in the final wall capacity check due to brevity. Using the distribution of lateral loads based on approximated wall stiffness values, the initial wall construction can be established.

Loads on longitudinal walls (grids 2 and 3) to longitudinal loading – Table 1:

Max wall shear at grid lines 2 & 3 = 8603.1/ 3 walls = 2,868 lbs on each wall

2,868 lbs / 10 ft length = 286.8 plf shear demand

Loads on transverse walls (grids A and B) to transverse loading:

Max wall shear at grid lines A and B = 9,054.6/2 walls = 4,527 lbs per wall

4,527 lbs / 8 ft length = 565.9 plf

Referring to SDWPS 2018 Table 4.2A for shear strength and apparent shear stiffness, 15/32 in OSB wood structural panel sheathing, on one side of the wall, is selected as follows:

Grid lines A and B – use 10d@3 in o.c. as spacing of nails at panel edges

Capacity v_s = (1,200)/2 = 600 plf, > demand 565.9 plf, OK

Apparent shear stiffness G_a = 37 kips/in

Grid lines 2 and 3 – use 10d@4 in o.c. as spacing of nails at panel edges

Capacity vs = (920)/2 = 460 plf > demand 286.8 plf, OK

Apparent shear stiffness, G_a = 30 kips/in

For the walls on lines 2 and 3, 10d@6 inches o.c. has enough capacity for the currently calculated loads (620/2 = 310 plf > 286.8 plf). However, a tighter nail spacing was chosen to increase the stiffness and decrease the wall deflections and story drifts along these lines. Because of the configuration of this example, as will be the case with many open front structures, drift and irregularity requirements will govern much of the design instead of strength requirements.

Additional design selections for the shear walls are:

- 2x6 Douglas-fir-larch (DFL) framing used throughout

- Hold downs = pre-manufactured bucket style hold-downs with screw attachments. The same hold downs are used at all walls for simplification, though in practice they could vary at each wall.
    - (3) 2x6 boundary posts at grid lines A and B, EA = 42,075,000 lbs
    - (2) 2x6 boundary posts at grid lines 2 and 3, EA = 28,050,000 lbs
- ASD capacity of selected hold down = 4,565 lbs Selection based on $SW_{A,B}$.
- Strength capacity = 4,565(1.4) = 6,391 lbs per ESR report
- Displacement at allowable stress load = 0.114 inch per ESR report
- Displacement at strength load = 0.154 inch per ESR report
- Min. wood thickness at hold downs = 3 in

These design selections will be verified after an updated RDA is performed based on the selected shear wall construction.

Shear Wall Moment-Resisting Arm vs. Deflection Wall Length – Bending

When calculating shear wall over-turning for the determination of the hold down anchorage, several options are commonly used to determine the moment-resisting arm length. Some prefer to use the distance between the outside edge of the compression side of the wall to the center of the hold down rod. Others prefer to use the full length of the wall, especially if strap anchors are used. Others use the distance between center of bearing of the stud pack on the compression side of the wall to the center of the hold down rod. This example uses the later approach to determine the hold down forces as shown in *Figures 5* and *7*.

When calculating bending deflection, the full length of the wall is commonly used. However, when calculating the last term of the deflection equation, wall rotation, the wall length as discussed in *Section 5.4.3, Figure 6* should be considered.

5.4 Calculated Nominal Wall Stiffness

Given the selected wall construction details, more accurate shear wall stiffness values are determined.

For an applied lateral force, F, and the corresponding calculated horizontal deflection, δ, a linear stiffness, K, is calculated by:

$$K = F / \delta$$

Wood structural panel shear wall deflections from a given load can be determined using either the SDPWS three-term deflection equation 4.3-1 or four-term deflection equation C4.3.2-1. IBC Equation 23-2 is available for stapled shear walls. The three-term shear wall deflection equation (SDPWS Eq. 4.3-1) will be used for this example to determine wall deflection and stiffness. The differences between the effects of the two equations are further discussed in *Section 7.5*. See *Section 5.4.3, Figure 6* for modification of wall length to be used for wall rotation in the deflection equation.

$$\delta_{SW} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$ SDPWS Eq. 4.3-1

When vertical dead loads are used to resist overturning, commonly accepted deflection calculation methods have some intrinsic complexities. At low levels of horizontal loading, the vertical gravity loads alone can be enough to prevent net uplift from occurring at the boundary elements at the "tension" side of the shear wall. However, once the horizontal forces overcome the resisting moment due to gravity forces, net uplift occurs and any slip and flexibility in the hold down adds to the flexibility of the shear wall, increasing the deflection and decreasing stiffness. This basic model to calculate shear wall deflections creates a non-linear relationship between lateral load and horizontal deflection where the calculated stiffness can vary with both the vertical and lateral loading on the wall. An engineer could attempt to calculate a deflection and wall stiffness consistent with each independent load combination and direction of loading applicable to the structural design; however, coupled with rigid diaphragm analysis where the lateral loads on a wall depend on the wall stiffness, such a process would be a herculean effort, likely accomplished successfully only in a nonlinear structural analysis program, with little demonstrated structural benefit in normal equivalent lateral force seismic design.

Therefore, the following suggestions are provided as one possible rational approach to shear wall stiffness calculations.

5.4.1 Vertical Loading for Wall Stiffness Calculations

For this one-story structure example, an expected gravity loading of 1.0 D is used for shear wall deflections and stiffness calculations. This represents a single "nominal" gravity loading on which to base the deflection calculations to perform an RDA. This approach can be generalized to multi-story buildings where it is common to ignore the presence of any live loading to resist overturning.

Special situations such as high snow or storage loads may prompt a designer to consider a high and low gravity loading as separate conditions for which different shear wall stiffness values are calculated. This is similar to the gravity loads used for non-linear procedures in ASCE 41-13 Eq. 7-3 and the simplification of gravity loading in ASCE 7-16 Section 16.3.2 for non-linear response history analysis procedures.

Similarly, the vertical seismic effects (E_v = +/- 0.2 $S_{DS}D$) are not considered for wall *stiffness* calculations in this example. This approach may not be valid for structures with significant vertical discontinuities in the seismic or gravity load-resisting systems.

5.4.2 Lateral Loading for Wall Stiffness Calculations

A similar approach is used in consideration of the magnitude of the lateral loading when calculating nominal wall stiffnesses. SDPWS Section C4.3.2 shows how SDPWS Equation C4.3.2-2 is a simplification of the four-term shear wall deflection equation C4.3.2-1 by calibrating the three-term equation to match the non-linear four-term equation at the applied lateral load of 1.4 times the ASD shear wall capacity. This simplification removes the nonlinear behavior of the nail slip term, e_n. A similar approach is used here to remove the nonlinear effects introduced through the Δ_a term by calculating the stiffness of the wall at the wall capacity. Given a wall design that has the required strength, the stiffness of the wall used in this example is calculated based on a strength level wall capacity limit, not based on the actual applied loads.

One could follow this approach only considering the shear capacity of the wall sheathing. However, where a hold down has been selected that limits the shear capacity of the wall to a loading lower than the full sheathing capacity, the reduced shear capacity from the hold down capacity is used.

Basis of Lateral Loading on Shear Wall to Determine Nominal Stiffness

The maximum seismic wall capacity used to determine nominal stiffness can be governed by any of the components of the shear wall—e.g., shear capacity (nailing/sheathing), hold down capacity, anchor bolt or sill plate nailing, etc. A designer can consider all of the capacity limits of a particular shear wall design to select the minimum governing capacity to calculate a nominal stiffness. For cases where a lower stiffness is more conservative than a higher stiffness, as in story drift checks, etc., using a higher demand than the minimum governing capacity to calculate the stiffness is an acceptable approach. This is the basis of the simplification of the four-term deflection equations for shear walls and diaphragms into the three-term deflection equations.

For shear walls not requiring hold downs, the lateral wall force used to calculate stiffness can be determined at the point where uplift occurs at the tension side of the wall.

Where an engineer wants to perform more precise calculations, an alternative approach can be to calculate the nominal wall stiffness values by using wall loads greater or equal to the maximum demands from the required LRFD design loads. This could be a feasible route to "sharpen the pencil" to produce higher stiffness values and lower deflections. It is also recommended for use in conjunction with automated calculation tools implementing nominal stiffness calculations using the four-term deflection equations for shear walls and diaphragms.

5.4.3 Detailed Shear Wall Deflections and Nominal Stiffness Calculations

Below are examples of detailed deflection and stiffness calculations using previously selected shear wall details with a vertical applied loading of 1.0D and lateral loading of the selected lateral capacity of the wall. Because wall capacities are used to calculate corresponding deflections, design load amplification factors do not apply (ρ = 1.0 and A_x = 1.0).

<u>Shear walls at grid lines A and B</u> – See *Figure 5* for applied dead loads and calculated wall geometry.

Lsw = 8 ft, hsw = 10 ft

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

Maximum wall capacity (LRFD)

$SW_{A,\,B}$ – Maximum nailing/shear capacity:

$V_{A,\,B}$ = 0.8(1200 plf)(8 ft) = 7680 lbs

$SW_{A,\,B}$ – Use hold down capacity to determine corresponding lateral force capacity:

T = 6,391=[V(10 ft) – 1,836 (7.687 ft) – 3,248 (3.812 ft) + 2,295(0.063 ft)] /7.312

V = 7,308 lbs controls lateral capacity. Use for deflection calculations.

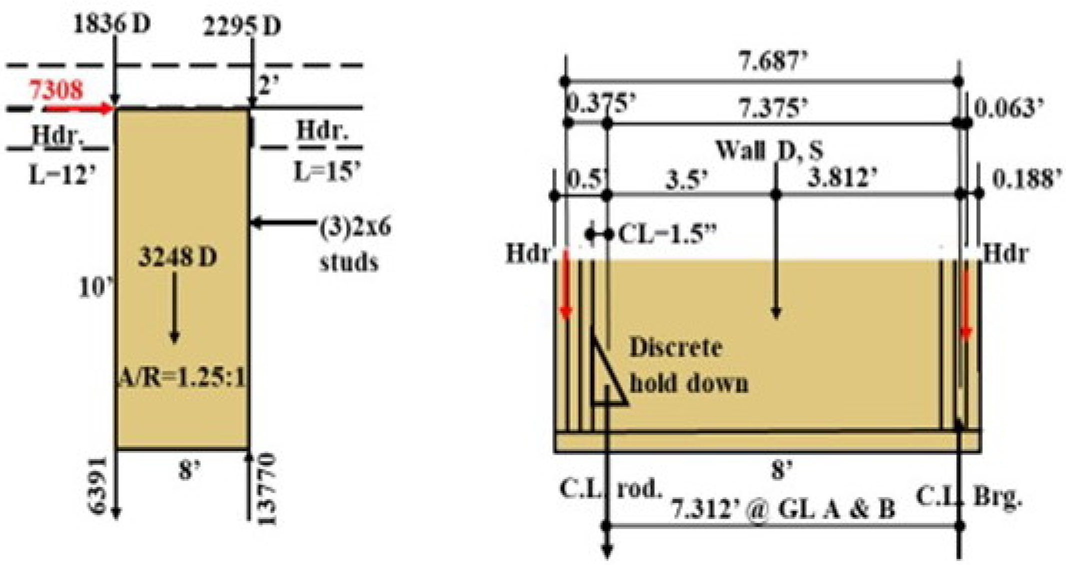


Figure 5. Shear Wall Forces at Grid Lines A and B for Stiffness

Shear wall deflection

$$\delta_{sw\ A,B} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b} \qquad \text{Eq. 4.3-1}$$

Where v = wall unit shear (plf)
h = wall height (ft)
b = Wall width (ft)
G_a = apparent shear stiffness (k/in)
Δ_a=Sum of vertical displacements at anchorage and boundary members (in)

The first term of the deflection equation represents bending deflection. The second term is shear deflection and the third term is wall rotation. Δ_a in the third term is the vertical elongation of the anchorage system which can include hold down deformation, plate crushing and shrinkage.

New proposals for the next edition of the SDPWS address the inaccuracy of the last term of the deflection equation, $\frac{h\Delta_a}{b}$. The concern is regarding the length of the rotation arm that is to be used for solving the lateral displacement at the top of the wall (See *Figure 6*). The current version of the equation uses the full width of the wall; however, some components of the elongation, Δ_a, occur at the anchorage. Either Δ_a has to be proportionally increased by b/$\bar{x}$ to represent the vertical displacement at the end of the wall when using the full length of the wall, "b", or an effective wall length "$\bar{x}$" has to be used with the calculated vertical displacement, Δ_a. They both produce the same wall horizontal displacement. The SDPWS proposal uses the outside corner of the wall at the compression side for the point of rotation. This example uses centerline of bearing by preference and consistency throughout this example.

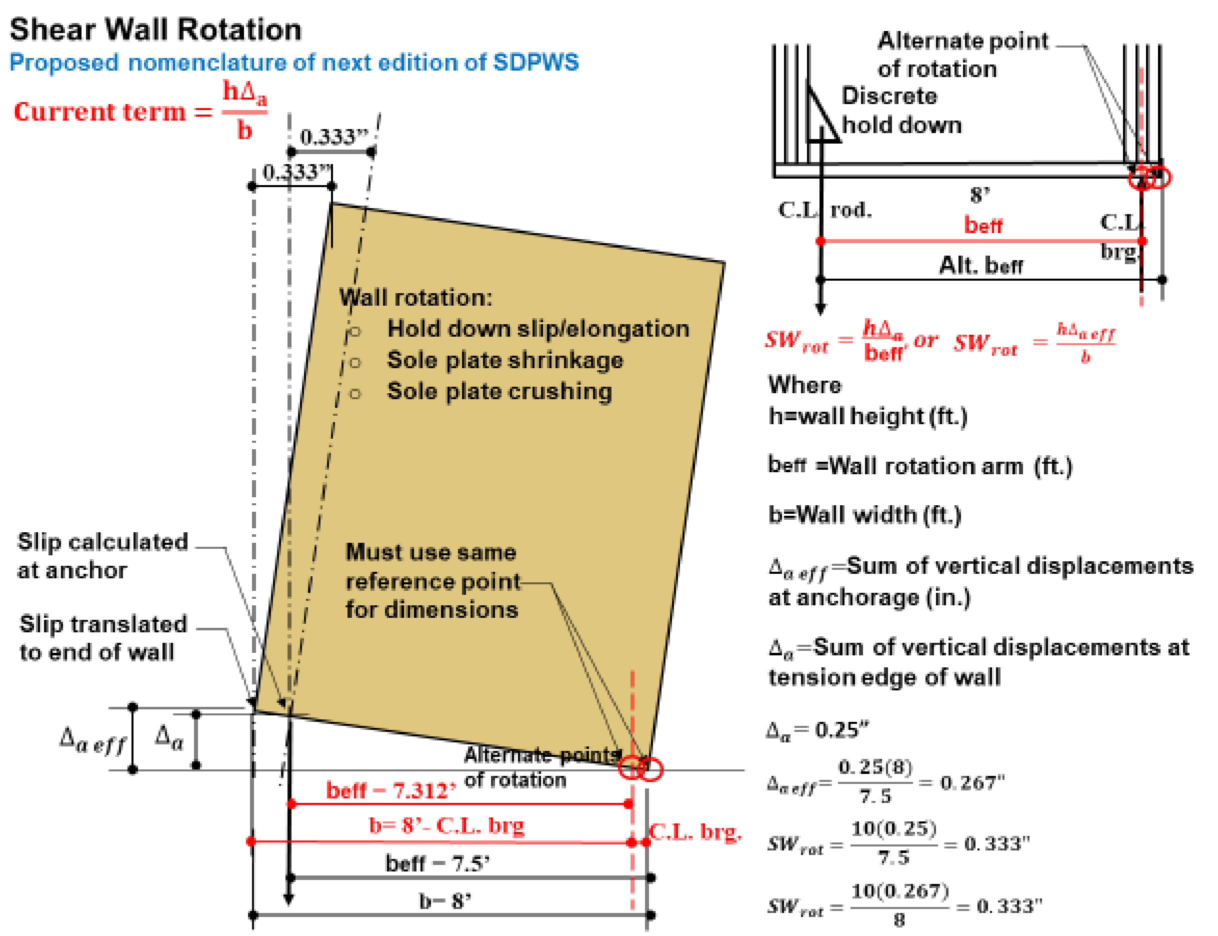


Figure 6. Wall Rotation Moment Arm References

If calculations show that the tension chord member is in compression, vertical elongation of the hold down anchorage and wall rotation for the shear walls do not occur and $\Delta_{a,HD} = 0$ in

$\Delta_{a,HD} = 0.154$ in hold down anchor slip at strength capacity

Crushing contributes to wall rotation which is caused by perpendicular-to-grain bearing stresses between the wall compression chord members and bottom plate. These stresses must be adjusted in accordance with NDS Section 4.2.6. The allowable bearing stresses noted therein are based on steel plate to wood plate contact. For wood to wood contact, an adjustment factor = 1.75 for parallel to perpendicular grain wood contact must be used. Reference NDS C4.2.6[7].

Boundary values for bearing perpendicular-to-grain stresses and crushing using DFL:

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

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

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

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

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

$A_{chord} = 24.75\ in^2$ for (3) 2x6 DFL boundary studs

C = 13770 lbs See *Figure 5*

$f_{c\perp} = \left(\frac{C}{A_{chord}}\right) = \frac{13770}{24.75} = 556.36$ psi>456.3 psi,

$$\Delta_{crush} = 1.75\left[0.04 - 0.02\left(\frac{1-\frac{556.36}{625}}{0.27}\right)\right] = 0.056 \text{ in}$$

Shrinkage of the bottom plate on the compression side of the wall can also contribute to wall rotation and is often considered in the wall rotation and deflection in mid-rise wood construction. This example includes a shrinkage component as an example. For a single-story building, such as the example, the potential impact of wood shrinkage on lateral deflections is small and not typically considered in practice.

Wood has a large amount of naturally occurring moisture. With time, lumber dries to the ambient climate environment, known as its equilibrium moisture content (EMC). There are several ways to calculate the amount of shrinkage in wood members. One simple calculation is to assume a dimensional change of 0.0025 inches per inch of cross-sectional dimension for every 1 percent change in MC. This loss of moisture results in volumetric changes to the lumber, which is often referred to as shrinkage.

For more information on calculating and detailing for shrinkage in mid-rise buildings see the WoodWorks paper, *Accommodating Shrinkage in Multi-Story Wood-Frame Structures*.

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

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

Estimating a 15% initial MC at time of construction and 10% equilibrium MC results in:

$\Delta_{a,\text{shinkage}}$ = 0.0025(1.5)(15-10)= 0.019 in

Δ_a = 0.154 in + 0.056 in + 0.019 in = 0.229 in

The calculated deflection becomes:

$$v = \frac{7308}{8} = 913.5 \text{ plf}$$

EA shear wall chords at grid line A/B = 42,075,000 lbs, (3)2x6 DFL, KD, chords

$$\delta_{sw\ A,B} = \frac{8(913.5)10^3}{42{,}075{,}000(8)} + \frac{913.5(10)}{1000(37)} + \frac{(0.229)(10)}{7.312}$$

$$\delta_{sw\ A,B} = \quad 0.022 \quad + \quad 0.247 \quad + \quad 0.313 = 0.581 \text{ in}$$

$$k = \frac{F}{\Delta} = \frac{7308}{0.581(1000)} = 12.57 \text{ k/in per wall}$$

k = (2) 12.57 = 25.14 k/in for two walls in line

Shear walls at grid lines 2 and 3

For the applied loads and geometry of the shear walls on lines 2 and 3 used to calculate the nominal stiffness, see *Figure 7.*

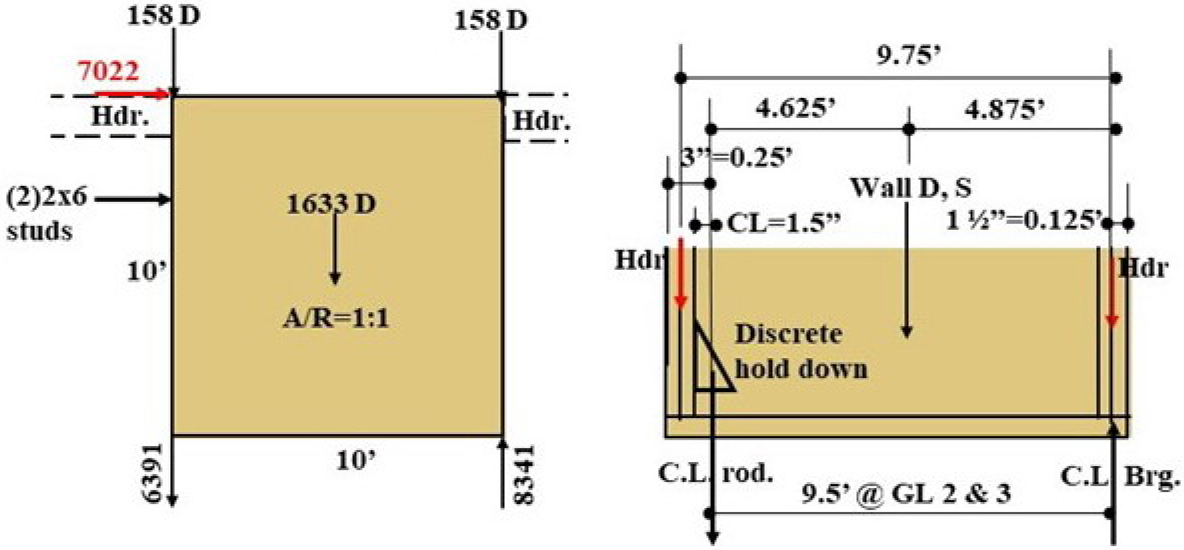


Figure 7. Shear Wall Forces at Grid Lines 2 and 3 for Stiffness

L_{sw} = 10 ft, h_{sw} = 10 ft

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

Maximum wall capacity:

$SW_{2,3}$ – Maximum nailing/shear capacity

$V_{2,3}$ = 0.8(920)(10 ft) = 7,360 lbs

$SW_{2,3}$ Hold down capacity: Back out maximum lateral force based on hold down capacity.

T = 6,391 = [V(10 ft) - 158.3(9.75) – 1,633(4.875)] /9.5

V = 7,022 lbs controls

Vertical elongation of hold down anchorage, $\Delta_{a,HD} = 0.154$ in, ratio of calculated tension force divided by strength force capacity (in this case, ratio = 1)

A_{chord} = 16.5 in^2 for (2) 2x6 DFL boundary studs

$$f_{c\perp} = \left(\frac{C}{A_{chord}}\right) = \frac{8341}{16.5} = 505.5 \text{ psi} > 456.3 \text{ psi}$$

$$\Delta_{\text{a,crushing}} = 1.75\left[0.04 - 0.02\left(\frac{1-\frac{505.5}{625}}{0.27}\right)\right] = 0.045 \text{ in}$$

Shrinkage = 0.0025(1.5)(15-10) = 0.019 in

EA shear wall chords = 28,050,000 lbs,

$$v = \frac{7022}{10} = 702.2 \text{ plf}$$

$$\delta_{sw\ 2,3} = \frac{8(702.2)10^3}{28{,}050{,}000(10)} + \frac{702.2(10)}{1{,}000(30)} + \frac{10(0.154 + 0.045 + 0.019)}{9.5}$$

$$\delta_{sw\ 2,3} = 0.020 + 0.234 + 0.229 = 0.484 \text{ in}$$

$$k = \frac{F}{\Delta} = \frac{7022}{0.484(1000)} = 14.51\text{k/in for one wall}$$

k = (3) 14.51 = 43.54 k/in for three walls in line

The summary of the deflection calculations corresponding to this converged RDA are found in Table 2.

- Shear walls at lines A and B, K = 25.14 k/in
- Shear walls at lines 2 and 3, K = 43.54 k/in

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 Loading													
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

Table 2. Summary of Deflection Calculation for Nominal Stiffness

5.5 Revised RDA Load Distribution from Nominal Wall Stiffnesses

Using the nominal stiffness values of the shear walls calculated above, the rigid diaphragm analysis calculations are updated to calculate the distribution of lateral loading to the shear walls. The revised ASD wall design seismic load distribution to the shear lines is shown in Table 3.

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

Table 3. Summary of Deflection Calculation for Nominal Stiffness

5.6 Capacity Verification of Wall Design

Because the initial design was based on a preliminary estimate of relative wall stiffness values, the selected shear wall design details need to be verified as adequate for the loads calculated with the nominal wall stiffnesses. As in *Section 5.3,* the shear wall capacity checks are performed at ASD level using the assumed ρ = 1.3 and Ax = 1.25.

Grid lines 2 and 3:

Maximum load in initial design in *Section 5.3* = 8,603.1 lb. on line / 3 walls = 2,867.7 lbs per wall

Maximum load in revised RDA in *Section 5.5* = 8,565 lb. on line / 3 walls = 2,855.3 lbs per wall

2,855.3 lbs revised demand < initial design capacity 2,867.7 lbs

∴ Loads have decreased from initial design.

Grid lines A and B:

Maximum load in initial design in *Section 5.3* = 9,054.6 lb. on line / 2 walls = 4,527.3 lbs per wall

Maximum load in revised RDA in *Section 5.5* = 9,057.6 lb. on line / 2 walls = 4,528.8 lbs per wall

4,528.8 lb. revised demand > initial design capacity 4,527.3 lbs

∴ Loads have increased (*very slightly*) from initial design.

The calculations below show detailed strength checks using the allowable design stress load combinations using the revised RDA loading and load combination LC10 = 0.448D + 0.7ρQ_E. ASCE 7-16 Section 2.4.5 Load Combination 10 produces the largest wall uplift. Load Combination 8 and possibly 9 would need to be used for compression capacity checks such as the boundary stud design.

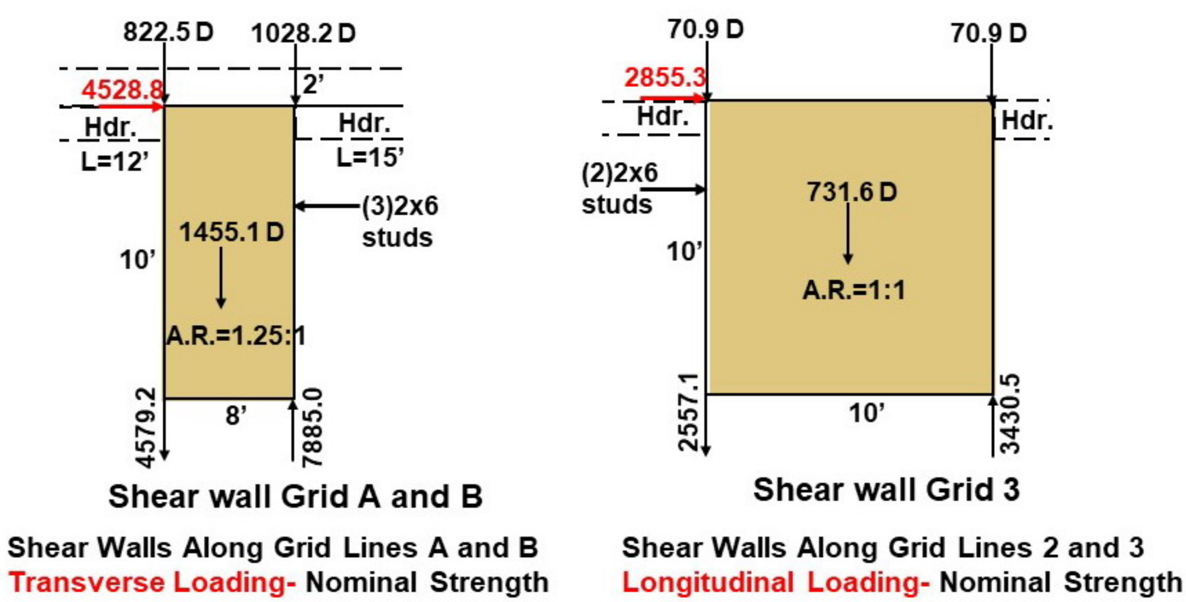


Figure 8. Applied ASD Wall Loads for Walls at Grids A and 3

Shear walls at grid line A – *See Figure 8.*

V_{sw} line A = $\frac{9{,}057.6}{2}$ = 4,528.8 lbs

vs = $\frac{4{,}528.8}{8}$ = 566.1 plf < 600 plf allowed ∴ OK

T = 4,579.2 lbs ≈ 4,565 lbs allowed ∴ hold down OK

Wall meets ASD capacity requirements.

<u>Shear walls at grid line 3</u> – See *Figure 8.*

V_{sw} line 3 = $\frac{8,565}{3}$ = 2,855.3 lbs

vs = $\frac{2,855.3}{10}$ = 285.5 plf. < 460 plf allowed ∴ OK

T = 2,557.1 lbs < 4,565 lbs allowed ∴ Hold down OK

Wall meets ASD capacity requirements.

Therefore, the wall designs initially selected in *Section 5.3* have the capacity to resist the loads resulting from the RDA.

In this example, the revised RDA based on nominal calculated stiffness value did not significantly change the load distribution among the shear walls. This is not always the case, especially where the lateral system includes a combination of relatively narrow and relatively long walls or hold downs with dramatically different anchorage slip values. If some component of the initial design was found to not have sufficient capacity at this step, then the design details would need to be updated, the nominal stiffness values updated, the rigid diaphragm analysis recalculated, and the capacity of the shear walls checked again.

6 Diaphragm Design Forces

Diaphragms are to be designed to resist the seismic design forces of the MLFRS with minimum diaphragm design forces based on estimated inertial roof and floor forces defined in ASCE 7 Section 12.10.1.1:

12.10.1.1 Diaphragm Design Forces

Floor and roof diaphragms shall be designed to resist design seismic forces from the structural analysis, but shall not be less than that determined in accordance with Eq. 12.10-1 as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

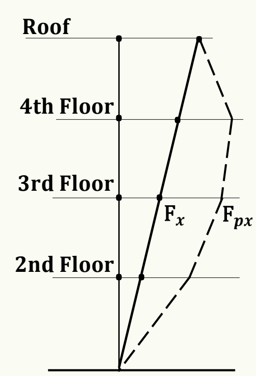
(12.10-1)

where
Fpx = the diaphragm design force at level x
Fi = the design force applied to level i

wi = the weight tributary to level i
wpx = the weight tributary to the diaphragm at level x

The force determined from Eq. 12.10-1 shall not be less than Fpx = 0.2$S_{DS}I_e w_{px}$ (12.10-2)
The force determined from Eq. 12.10-1 need not exceed Fpx = 0.4$S_{DS}I_e w_{px}$ (12.10-3)

Two different sets of calculations are required to determine lateral diaphragm design forces. The first is required for the design of the vertical seismic force-resisting elements, which are proportional to the maximum forces these elements might experience, based on the first mode of response, F_x. Secondly, each floor will have different acceleration histories and should be designed to resist an inertial force proportional to the estimated response acceleration of that floor, F_{px}.



Force Distribution

For inertial forces calculated in accordance with Eq. 12.10-1, the redundancy factor shall equal 1.0 per ASCE 7-16 Section 12.3.4.1, item 7.

For for a single story structure $F_x = F_{px} = \frac{S_{DS}I_e}{R} w_{px} = 0.167W_x$

$F_{px\ min} = 0.2S_{DS}I_e w_{px} = 0.2(1.084)(1)W_x = 0.217W_x$, controls (12.10-2)

$F_{px\ max} = 0.4\ S_{DS}I_e w_{px}\ x = 0.434W_x$ (12.10-3)

<u>Minimum Diaphragm Design Loading:</u>

$F_{px\ min} = 0.217(35)(40)(76) = 23{,}088.8$ lbs

The MLFRS design load in *Section 4.2* above is 17,769 lbs, to which ρ = 1.3 is applied for strength design of the structure in the logitudinal direction, as assumed.

ASCE 7-16 Section 12.10.1.1 requires the diaphragm to be designed to the maximum of these two:

MLFRS diaphragm (structure) load = 17,769(1.3) = 23,099.7 lbs

Minimum diaphragm inertial design load = 23,088.8 lbs

In this single story case, the difference between these value is only rounding error because the ρ/R component of the MLFRS is 1.3/6.5 = 0.2, which matches the 0.2 of ASCE 7 Eq. 12.10-2. This is not generally the case. The diaphragm design load used is:

$V_{total} = 23{,}099.7$ lbs

7 Longitudinal Diaphragm Design

For loading in the longitudinal direction, the diaphragm acts like an open-front/cantilever on both sides of the corridor walls. The diaphragm chords are continuous across the full length of the diaphragm and therefore realtively self-restrained between grid lines 2 and 3. The WSP layout for loading in the longitudinal direction is Case 1 as shown in SDPWS Table 4.2A and in *Figure 2*.

For seismic applications, many considerations during the design of cantilever diaphragms originate from the requirements of SDPWS 2015 Section 4.2.5.2 and in SDPWS Figure 4A.

> SDPWS 2015 Section 4.2.5.2: *Open-Front Structures*
> *For resistance to seismic loads, wood-frame diaphragms in open-front structures shall comply with all the following requirements:*
> 1. *The diaphragm conforms to sections 4.2.7.1, 4.2.7.2, or 4.2.7.3.*
> 2. *The L'/W' ratio as shown in SDPWS Figure 4A (a) through (d) is not greater than 1.5:1 when sheathed in conformance with 4.2.7.1 or not greater than 1:1 when sheathed in conformance with 4.2.7.2 or 4.2.7.3. 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.*
> 3. *For loading parallel to the open side, diaphragms shall be modeled as semi-rigid or idealized as rigid, and the maximum story drift at each edge of the structure shall not exceed the ASCE 7-16 allowable story drift when subject to seismic design forces including torsion and accidental torsion and shall include shear and bending deformations of the diaphragm, computed on a strength level basis amplified by C_d.*
> 4. *The cantilevered diaphragm length, L', (normal to the open side) shall not exceed 35 feet.*

SDPWS Section 4.2.5.1 addressing torsionally irregular structures has similar requirements which may govern irregular structures.

Addressing each item of SDPWS 4.2.5.2:

1. The diaphragm is detailed as a WSP-sheathed diaphragm per 4.2.7.1.

2. In this example, $L' = 35$ ft and $W' = 40$ ft for each side of the symmetric structure.

   $L'/W' = 35/40 = 0.875 \leq 1.0$, $\therefore$ OK for this one-story structure with assumed torsional irregularity

   Note: If this example plan was used in a multi-story structure with a torsional irregularity, the allowable aspect ratio of 0.67 is allowed, in accordance with Section 4.2.5.2, item 2 for structures over one-story in height, $0.875 > 0.67$ NG.

3. The diaphragm modeling is rigid and checking the story drift at the edges of the structure is a significant effort that follows.

> SDPWS 2015 open-front requirements have changed from the 2008 edition regarding allowable cantilever length. The 2008 edition limited the maximum cantilever length of an open-front structure to 25 feet; however, it had an exception that allowed an increase in the cantilever length where calculations show that diaphragm deflections can be tolerated. Consequently, there was no hard limit on the cantilever length provided the aspect ratio and all other requirements could be met. The 2015 edition limits the cantilevered length, L', to 35 feet with no exception provided.
>
> Section 4.2.5.2 of SDPWS 2015 has also been significantly changed from the previous edition, requiring the following for cantilever diaphragms with seismic loading:
>
> - Calculation of story drift at the edges of the structure
> - Verification if the building is torsionally irregular
> - Justification of the diaphragm to be idealized as rigid (or modelled as semi-rigid)

4. $L' = 35$ ft is right at the limit, $\therefore$ OK

7.1 Distribution of Torsional Forces to Diaphragm

There are several triggers in ASCE 7-16 and the SDPWS 2015 that can require consideration of accidental torsion load to be applied to a structure. The drift requirements for open-front structures in SDPWS 4.2.5.2 requires the application of the accidental torsion AND consideration of the deformations of the diaphragms. The method by which the accidental torsion is applied can impact the diaphragm deformation.

There are several methods of applying torsional forces to diaphragms and shear walls, including those in *Figure 9*. The calculation of the values in Figure 9 is shown below in *Section 7.2*. Any of the methods shown can produce similar results; however, the differences between methods should be understood. Methods 1, 2A and 2B are described as follows:

Method 1 – Distribution by PSF, No In-plane Wall Loads

Method 1 is a simplified approach, which applies the torsion as a concentrated moment about the center of rigidity. All the torsion is assumed to be resisted by the transverse walls located at grid lines A and B. The support reactions at the corridor walls receive the diaphragm loads by tributary area, without consideration of torsional effects. This is contrary to the results that would occur if a rigid diaphragm analysis was performed (reference *Tables 1* and *3*). In a rigid diaphragm analysis, the corridor walls will take a small portion of the torsional force. The lateral forces are applied to the diaphragms on a psf basis spread uniformly across the entire length of the diaphragm. The in-plane lateral forces of the exterior and corridor walls are included in the calculated per square foot weight. The effects of torsional rotation on the inertial forces applied to the diaphragm are not considered. Compared to the following methods, Method 1 will inaccurately estimate the inertial forces to the diaphragm in both direction of loading.

Method 2A – Distribution by Net Moment, No In-plane Wall Loads

The results of the rigid diaphragm analysis shown in *Table 1* show that not all of the torsion is taken out by the transverse walls. A small portion of the torsional moment is also taken out by the corridor walls. Method 2A distributes the net torsional moment into the diaphragm as an equivalent uniform load which is added to or subtracted from the uniform lateral loads as shown in *Figure 9*. The net torsional moment is equal to the total torsional moment minus the transverse walls resisting moment, or equally, the torsional force applied to the corridor walls multiplied by the distance between the corridor walls. In-plane wall forces at the exterior and corridor wall lines are included in the calculated psf load and are not separated out from the lateral forces as concentrated forces. The net moment takes into account the rotational effects on the loads applied to the diaphragm, increasing the diaphragm shears, chord forces and deflection on the right cantilever. This method is more direct and the simplest approach for applying forces into the walls and diaphragms.

Method 2B – Distribution by Net Moment, with In-plane Wall Forces

Method 2B is the same as method 2A, with the exception that the lateral in-plane wall forces from the exterior and corridor walls are applied as concentrated forces at the ends of the cantilever and at the corridor wall lines. This method may be prefered by some as a more rational approach to determining diaphragm chord forces, shears, deflection and story drift, especially if the exterior walls are facade walls, which can be heavier relative to other walls, or where walls become discontinuous to the foundation. As such, this method, could result in slightly larger chord forces than the other methods, especially if considering the effects of shear walls located along the chord lines (see *Section 7.2.2*).

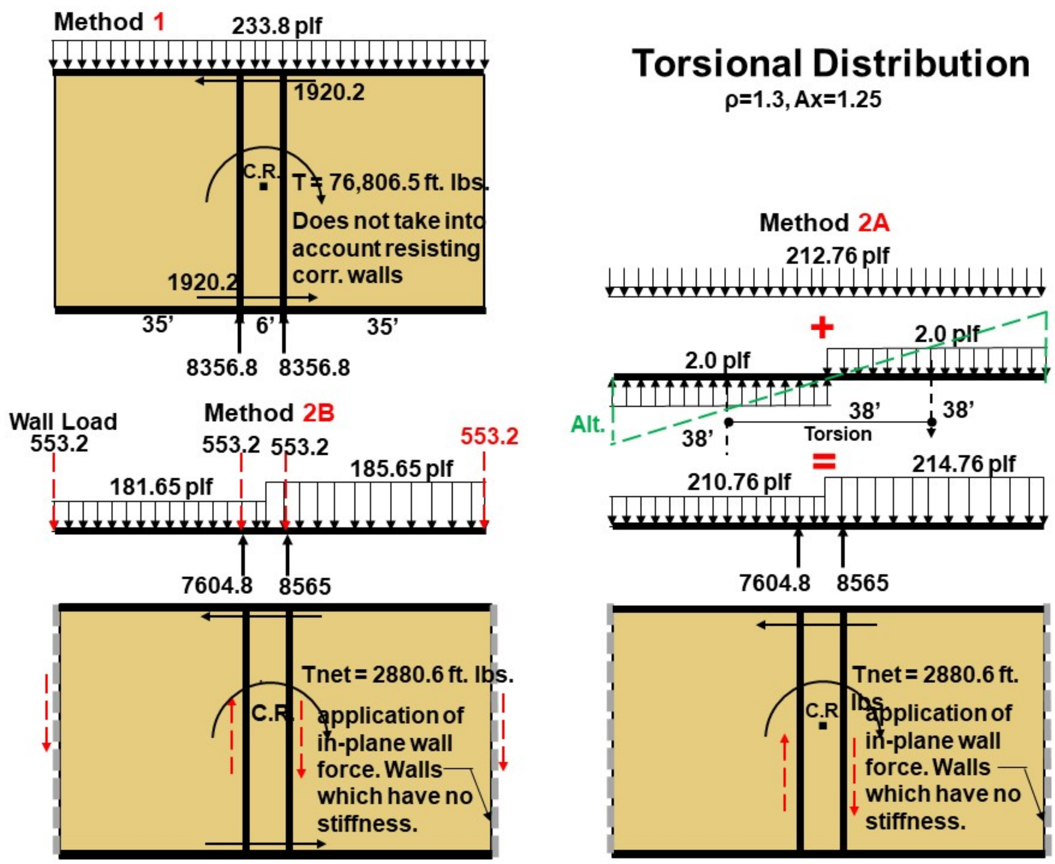


Figure 9. Application of Lateral and Torsional Forces

Method 2B will be used to investigate two of the questions in *Section 1:*

- Do shear walls located along diaphragm chord lines affect the diaphragm chord forces?
- 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?

Method 2A will be used to address diaphragm flexibility, drift, torsional irregularities, redundancy and amplification of accidental torsion.

7.2 Diaphragm Design

As noted in *Section 4.2*, the capacity design of the diaphragm will be done using allowable stress design and diaphragm deflection will be done using strength design.

7.2.1 Distribution of Diaphragm Design Forces

As shown in *Section 6* above, the governing diaphragm seismic design force is equal to the MLFRS seismic forces used for the shear wall design with the final RDA distribution of loads in

Section 5.6. In many cases, the diaphragm design forces in multi-story structures will be larger than the MLFRS forces. Each floor will have different acceleration histories and should be designed to resist an inertial force proportional to the peak response acceleration of that floor, Fpx.

From RDA results, the forces on gridlines 2 and 3 from the amplified torsional moment are:
F_T = 480.1 lbs

Using method 2B, these reactions are resolved into forces applied to the diaphragm which apply a net moment = 480.1(6 ft) = 2,880.6 ft lbs

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

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

The net distributed force on the diaphragm is found by subtracting out the loads of the walls from the total seismic design force:

V = 12,438.3 lbs from Table 3.

V_{net} = 12,438.3 (1.3) - 4(553.2) = 13,957 lbs distributed longitudinal force with a centroid at the center of mass

$W = \frac{13{,}957}{76}$ = 183.65 plf uniform load

$W_T = \frac{2{,}880.6}{38(38)}$ = 2.0 plf: equivalent uniform torsional load acting as Mnet

W_1 = 183.65 – 2.0 = 181.65 plf: uniform load minus torsional load = net uniform load left cantilever

W_2 = 183.65 + 2.0 = 185.65 plf: uniform load plus torsional load = net uniform load right cantilever

The resulting longitudinal loading including Method 2B torsion is shown in *Figure 10*.

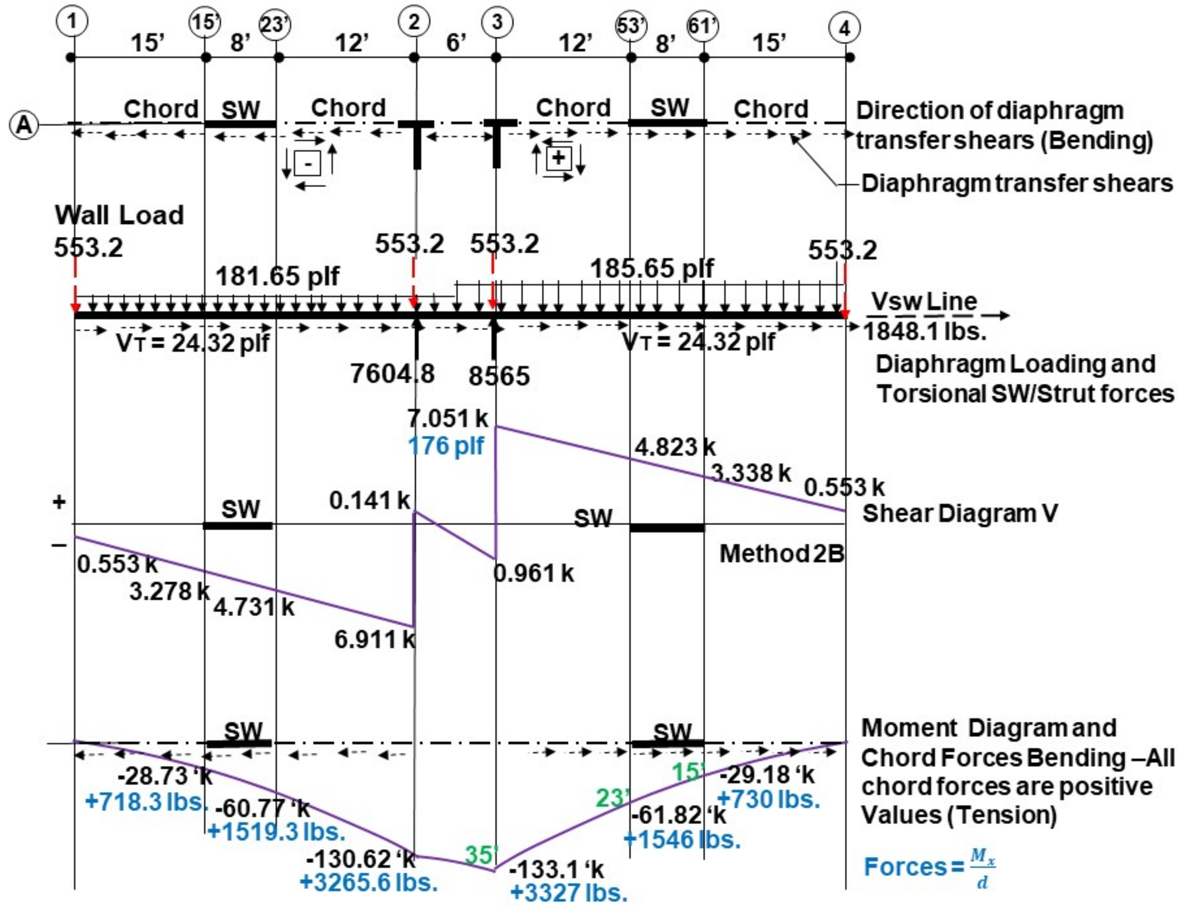


Figure 10. Method 2B Longitudinal Diaphragm Loading and Shear

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.) OSB	PLY	Vs (plf)	Ga (kips/in.) OSB	PLY	Vs (plf)	Ga (kips/in.) OSB	PLY	Vs (plf)	Ga (kips/in.) OSB	PLY	Vw (plf)	Vw (plf)	Vw (plf)	Vw (plf)
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

Table 4. SDPWS Table 4.2A
Nominal Unit Shear Capacities for Wood-Framed Diaphragms

Calculating the shear at grid line 3 from the longitudinal loading as shown in *Figure 10* is

Shear from longitudinal loading on wall line: 7,051 lbs / 40 ft = 176.3 plf.

The maximum diaphragm shear is where this diaphragm shear and the unit torsional shear along lines A and B are additive.

Force acting at grid line A and B = 1,848.1 lbs per Table 3

Unit torsional shear = $\frac{1{,}848.1}{76} = 24.32$ plf

Maximum diaphragm shear = 176.3 + 24.32 = 200.6 plf

Select roof sheathing from Table 4: 2015 SDPWS Table 4.2A – Use 15/32 OSB w/ 10d @ 6 in o.c.:

- 200.6 plf < vs = 0.5(580) = 290 plf. per in SDPWS Table 4A
- G_a = 25, blocked, Case 1 longitudinal, Case 3 transverse

There is a possibility that an unblocked diaphragm could work in some cases or in some areas; however, a blocked diaphragm will be used to provide diaphragm stiffness:

- 10d@6 in o.c. boundary and supported panel edges:
  vs = 0.5(510) = 255 plf, G_a = 15, unblocked, Case 1 – Longitudinal
- 10d@6 in o.c. boundary and supported panel edges
  vs = 0.5(380) = 190 plf, G_a = 10, unblocked, Case 3 – Transverse

7.2.2 Diaphragm Chord Forces – Method 2B (ASD)

The maximum chord forces including the effects of the shear walls placed along the chord lines must be determined before the diaphragm deflection can be calculated.

The forces developed in the chords along lines A and B are caused by bending from the application of lateral forces, the forces caused by rotation of the diaphragm, and from the resisting shear walls located along the line. *Figure 12* shows the direction of the bending and rotational shears that are transferred into the chords.

Before calculating the chord forces, it might be helpful to understand how the shears flow through the diaphragm and into boundary members, collectors and chords, especially with complex layouts. *Figure 11* shows a visual aid that can help provide some clarity on how to calculate the net chord forces. Sheathing element symbols are placed on the plan at appropriate locations, in accordance with the positive or negative shears that occur at those locations.

Sheathing elements represent a 1-foot by 1-foot square piece of sheathing. The arrows shown adjacent to the sheathing element are the shears that act on the edges of the sheathing. The sign convention for positive or negative shears are shown in the figure. The shears that are transferred from the sheathing elements into the chords or collectors are equal in magnitude but act in the opposite direction and are designated as dashed lined arrows. These are termed as transfer shears. A free webinar, *Offset Diaphragm Design*, describes this method in detail with an example application. It can be found at the following link: http://www.woodworks.org/education/online-seminars/.

For bending, sheathing element symbols are placed on the plan based on the direction of the diaphragm transfer shears that would cause tension and compression forces in the chords. These transfer shears are shown as dashed arrows at grid lines A and B. The rotational forces applied to the shear walls and chord members are shown in red and are equal to 1,846 lbs, or 24.29 plf along the chord lines. Shear wall shears at grid lines A and B = 115.38 plf acting opposite to the direction of rotation creating a net shear of 91.09 plf at the walls. *Figures 12* and *13* show the final diaphragm loading condition and location of the chord splices. The (2) 2x6 wall top plates act as the diaphragm chords and extend the full length of the diaphragm. The splice locations were selected to accommodate a situation where an engineer might want to make the top of glulam beam headers flush with the wall top plate along grid line A (i.e., the glulam beam intersection at each end of the shear walls along grid lines A and B).

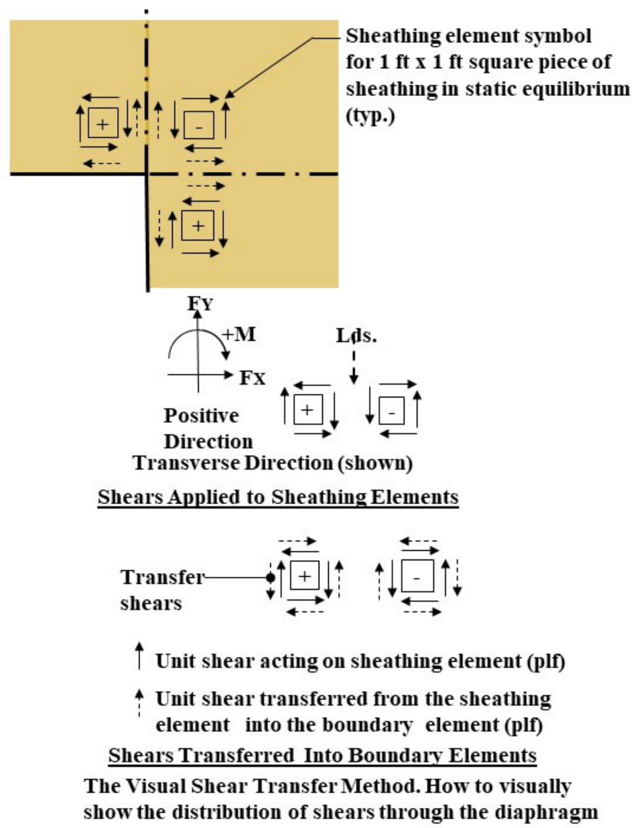


Figure 11. Diaphragm Shear Visual Aid

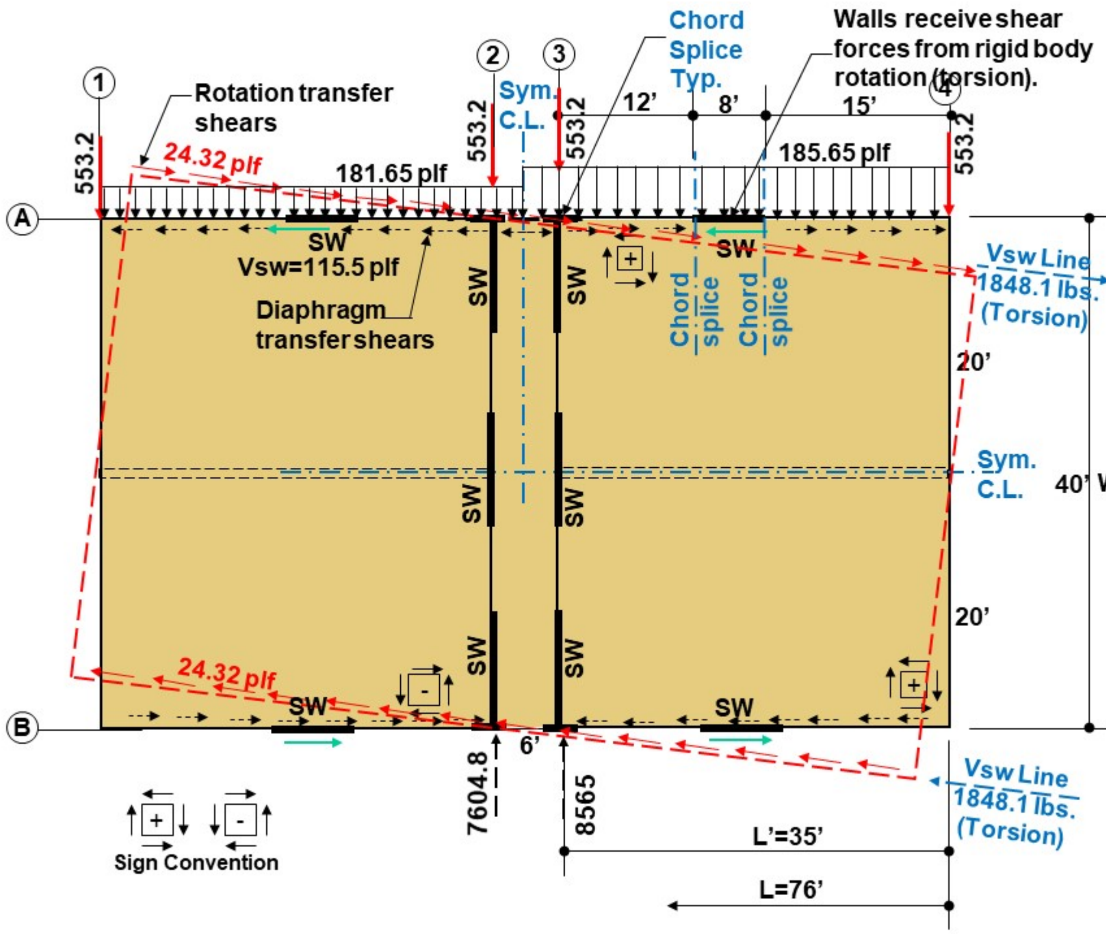


Figure 12. Diaphragm Loading and Rotational Chord Forces

Torsional shear and moment diagrams caused by bending are shown in *Figure 13,* which also shows the method of determining the final chord forces.

By observation, the shear wall at the left cantilever, located between grid lines 15 feet and 23 feet, resists rotation and therefore increases the tension forces in the chord at that location. The shear wall at the right cantilever, located between grid lines 53 feet and 61 feet, also resists rotation, thereby reducing the chord tension forces. The same process is used to calculate the compression chord forces.

Line 1: Shows the direction of the diaphragm shears transferred into the top chord caused by bending.
Line 2: Shows the additional chord forces caused by diaphragm rotation.
Line 3: References the diaphragm moments and chord forces caused by bending = $\frac{M}{d}$, where d = 40 feet.
Line 4: The final chord forces are determined by observing of the direction of applied shear transfer forces and combining the bending chord forces determined on line 2 with those of line 3.

Starting from the left, continuing to the right:
F_{15} ft = +718.3 – 364.8 = 353.5 lbs
F_{23} ft = +1,519.3 + 364.8 = 1,884.1 lbs
F_{35} ft = 3,265.6 + 72.96 = 3,338.6 lbs

Starting from the right, continuing to the left:
F_{15} ft = + 730 + 364.8 = 1,094.8 lbs
F_{23} ft = 1,546 - 364.8 = 1,181.2 lbs
F_{35} ft = 3,327-72.96 = 3,254 lbs

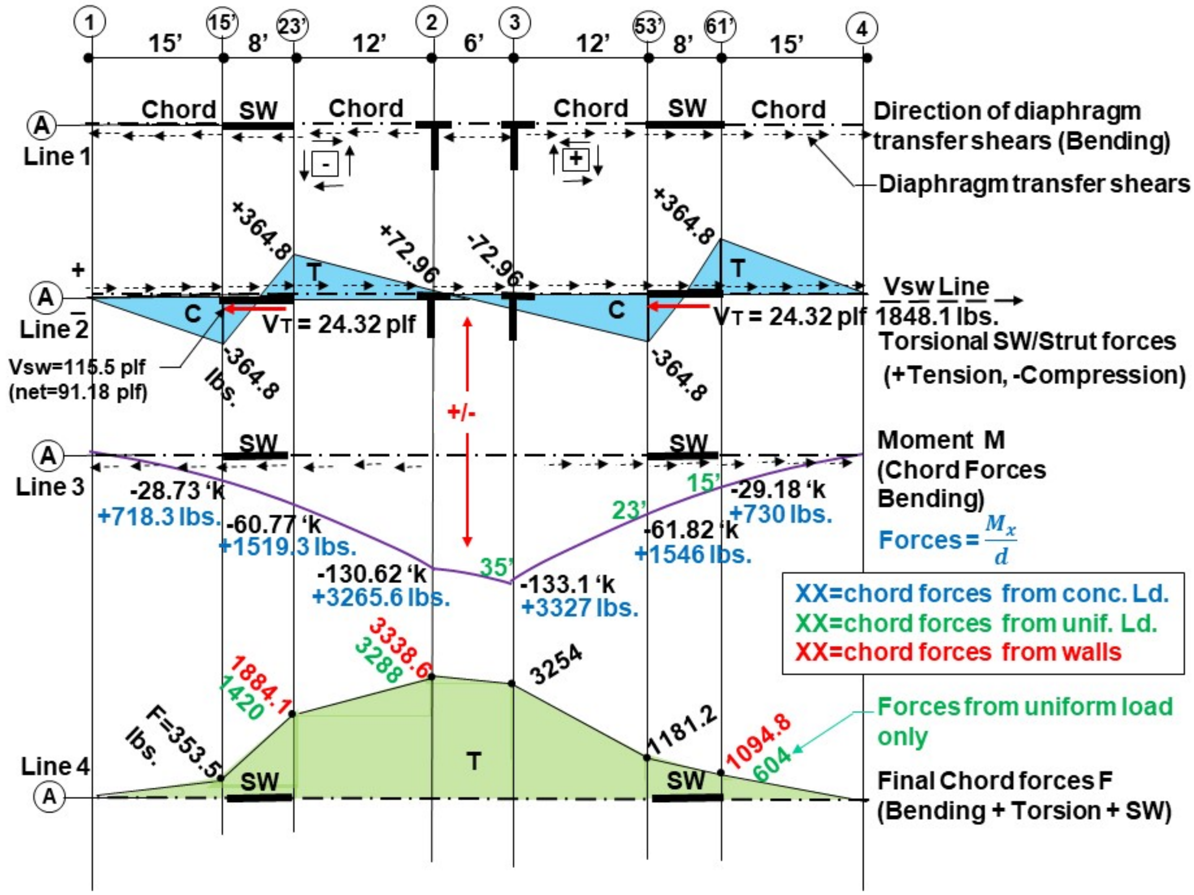


Figure 13. Final Chord Forces at Grid Line A

Overall, the presence of the short shear walls used in this example changes the chord forces by approximately 365 pounds at the start of each shear wall. This suggests that the shear wall effects on the chord forces are minimal for this example, but can be significant if larger eccentricities occur with short shear walls located along the chord lines. The effects of full-length shear walls at grid lines A and B is discussed in *Section 9.2*.

Maximum chord force = 3,338.5 lbs

Using (2)2x6 DFL No.1 wall top plates as the diaphragm chords: 2018 NDS Supplement Table 4A F_t = 675 psi, Fc// = 1,500 psi. Only one 2x6 plate resists the chord forces due to the nailed splice joint.

$$f_t = \frac{3338.5}{8.25} = 404.7 \text{ psi} < 675(1.6) = 1080 \text{ psi} \therefore \text{OK}$$

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

Chord splices left cantilever (nailed):

- At 35 ft, F = 3,338.5 lbs, No. nails $= \frac{3,338.5}{226}$ = 14.8, use (24) 16d nails each side of splice
- At 23 ft, F = 1,884.1 lbs, No. nails $= \frac{1,884.1}{226}$ = 8.3, use (16) 16d nails each side of splice
- At 15 ft, F = 353.5 lbs, No. nails $= \frac{353.5}{226}$ = 1.6, use (8) 16d nails each side of splice

Where the value of 226 is the allowable design value for a 16d common nail adjusted for C_D.

The number of nails used were selected to minimize chord slip and decrease diaphragm deflection.

Chord splices right cantilever (nailed):

- At 35 ft, F = 3254 lbs, No. nails = $\frac{3,254}{226}$ = 14.4, use (24) 16d nails each side of splice
- At 23 ft, F = 1181.2 lbs, No. nails = $\frac{1,181.2}{226}$ = 5.2, use (16) 16d nails each side of splice
- At 15 ft, F = 1094.8 lbs, No. nails = $\frac{1,094.8}{226}$ = 4.8, use (8) 16d nails each side of splice

The number of nails used were selected to reduce chord slip.

7.3 Cantilever Diaphragm Deflection Equations

A task group of AWC's Wood Design Standards Committee developed the following approximate equations to calculate the maximum deflection of cantilever diaphragms. The equations, which use terms similar to those in the 2018 SDWPS Section 4.2.2, are expected to be published in a future edition. Three-term and four-term deflection equations similar to the deflection equations for simply supported diaphragms have been developed. The three-term equation will be used for this example to determine diaphragm deflection and stiffness.

Cantilever Diaphragm Deflection Equations

Three-term equation for uniform load:

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

Where:

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

L' = cantilever diaphragm length, ft

W' = cantilever diaphragm width, ft

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

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

Δ_c = diaphragm chord splice slip, in

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

For a uniform load of w, the induced unit shear at the support $v = w\,L' / W'$

Four-term equation for uniform load:

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

Where:

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

Gvtv = Panel rigidity through the thickness

Similarly, the equations developed for a point load at the end of the cantilever are as follows:

Three-term equation for point load:

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

Where:

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

For the point load of P, the induced unit shear at the support, $v = P / W'$

Four-term equation for point load:

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

7.4 Check Assumption of Rigid Diaphragm (STR)

This section verifies the rigid diaphragm assumption using the right cantilever of the structure and discusses the relevant sections of ASCE 7, IBC and SDPWS. Method 2A will be used for the diaphragm deflection check.

7.4.1 Diaphragm Flexibility – Seismic

Diaphragm flexibility is covered in 2018 IBC Section 1604.4, ASCE 7-16 Section 12.3.1 and SDPWS Section 4.2.5. These sections all refer to story drift for the determination of diaphragm flexibility but have slightly different requirements.

> ASCE 7 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. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semi-rigid modeling assumption).*

ASCE 7 Section 12.3.1.3 provides conditions by which a diaphragm can be justified to be flexible by calculation—when the maximum simple-span diaphragm deflection is greater than 2 times the average story drift at the adjacent supporting walls.

Per ASCE 7 12.3.1.3, a simple-span diaphragm can be idealized as flexible when:

$$\delta_{MDD} > 2 * \Delta_{ADVE}$$

Where δ_{MDD} is the maximum in-plane deflection of the diaphragm and Δ_{ADVE} is the average drift of the adjoining vertical elements (e.g., the average story drift of a simple-span diaphragm structure).

In this example, the diaphragm can qualify to be idealized as flexible under ASCE 7 12.3.1.1 item c. However, a flexible diaphragm analysis is not useful in the cantilever diaphragm configuration per the requirements of SDWPS 4.2.5.2 item 3. This is because flexible diaphragm analysis cannot be used for open-front structures because they cannot transfer torsional forces.

ASCE 7 *does not* provide a calculated rigid diaphragm condition; however, such conditions are in the IBC and the SDPWS. The language of IBC 2018 Section 1604.4 notes that a diaphragm is rigid for distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average of the story drift.

Per IBC 2018 Section 1604.4, a simple span diaphragm can be idealized as rigid when:

$$\delta_{MDD} \leq 2 * \Delta_{ADVE}$$

While the cantilever diaphragm configuration is not specifically covered ASCE 7-16 Figure 12.3-1, *Figure 14* shows a simple application of the conditions above that can apply in cantilever cases. For the cantilever diaphragm condition, one method to check the calculated flexibility checks of ASCE 7 and IBC is to use the drift of the closest shear wall line in the direction of the loading for Δ_{ADVE} as this is the only adjoining vertical element. See *Figure 14* for an illustration of these checks.

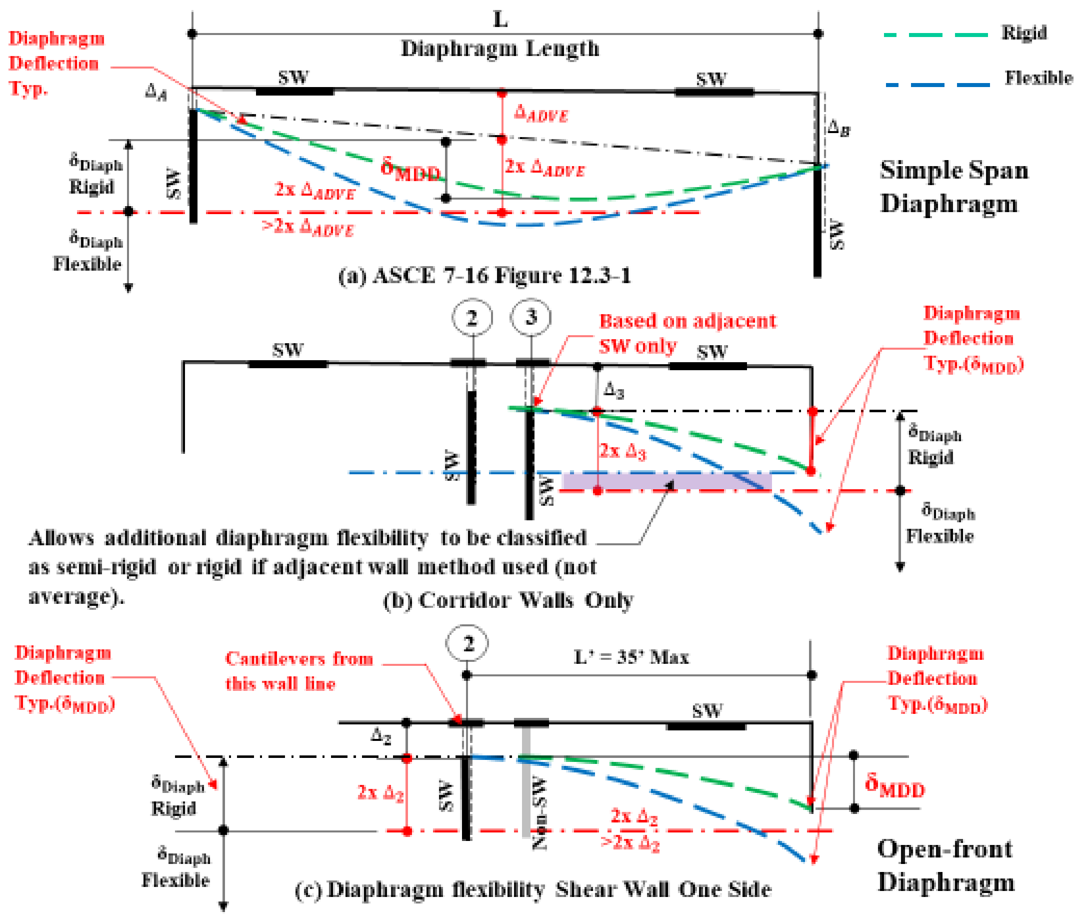


Figure 14. Calculated Diaphragm Flexibility Methods for Diaphragms

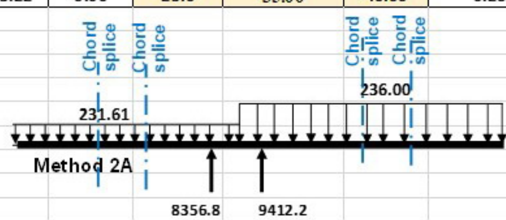
When calculating shear wall and diaphragm deflections for the determination of diaphragm flexibility and story drift, it is permitted to use ρ = 1.0 in accordance with ASCE 7-16 Section 12.3.4.1 (2). Given the assumption that the structure has a torsional irregularity and is assigned to SDC D, ASCE 7-16 Section 12.8.4.2 requires the inclusion of the accidental torsion in the determination of design story drift. Therefore, accidental torsion is applied and amplified by A_x = 1.25 in accordance with 12.8.4.3 (as previously assumed) and shown in *Table 5*. *Table 6* shows the resulting deflections.

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

Table 5. Force Distribution Right Cantilever for Flexibility and Drift, ρ=1.0, A_x = 1.25

	Diaphragm Deflection (STR)											Rt. Cantilever
	Splice Forces (Lbs.)			Σδ_slip	v unif.	v conc.	Ga	L'	W'	δDiaph Unif	δDiaph con	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											
	lincludes 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									

Table 6. Diaphragm Deflections for Flexibility and Drift, ρ = 1.0, A_x = 1.25

Wall displacements from *Table 5* and *Section 5.4.3*:

- $\delta_{\text{Diaph 1}} = 0.26 \text{ in}$, $\delta_{\text{Diaph 4}} = 0.265 \text{ in}$,
- Deflection at grid line 3 = $\frac{F}{1{,}000k} = \frac{9412.2}{1{,}000(43.54)} = 0.216 \text{ in}$

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

0.265 in < 0.432 in ∴ Diaphragm can be idealized as rigid.

7.4.2 Diaphragm Flexibility – Wind

Under the wind provisions of 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). Diaphragm flexibility requirements for wind conditions are embedded within the definitions of ASCE 7-16 Section 26.2, Definitions

– Diaphragm, which states that diaphragms constructed of WSP are permitted to be idealized as flexible.

There is no drift limit requirement in the code for wind design. There are various design office practices and suggested limitations, but no code requirements. ASCE 7-16 Appendix C, Section C.2.2, Drift of Walls and Frames, notes:

> *Lateral deflection or drift of structures and deformation of horizontal diaphragms and bracing systems caused by wind effects shall not impair the serviceability of the structure (i.e., shall show that the resulting drift at the edges of the structure can be tolerated, maintain structural stability and capacity to support lateral loads).*

Under wind loading, an open front diaphragm configuration is possible. Although not required for wind, following SDPWS 4.2.5.2 is considered good engineering practice, including constructing the diaphragm to meet semi-rigid or rigid stiffness requirements.

7.5 Check Story Drift (STR)

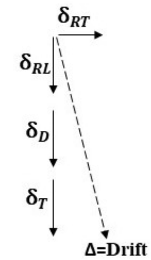
All structures with seismic loading need to meet the story drift limits of ASCE 7 Section 12.12.1. For structures designed using the equivalent lateral force procedure, the story drift values are determined from ASCE 7 Section 12.8.6:

> *The design story drift (Δ) 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). Where centers of mass do not align vertically, it is permitted to compute the deflection at the bottom of the story based on the vertical projection of the center of mass at the top of the story. Where allowable stress design is used, Δ shall be computed using the strength level seismic forces specified in Section 12.8 without reduction for allowable stress design.*
>
> *For structures assigned to Seismic Design Category C, D, E, or F that have horizontal irregularity Type 1a or 1b 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 along any of the edges of the structure. The deflection at level x (δx) (in or mm) used to compute the design story drift, Δ, shall be determined in accordance with the following equation:*

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

where:
C_d = deflection amplification factor in Table 12.2-1;
δ_{xe} = deflection at the location determined by an elastic analysis; and
I_e = Importance Factor determined in accordance with Section11.5.1.

For open-front structures, SDPWS Section 4.2.5.2 (3) applies similar drift checks where the maximum story drift at each edge of the open-front structure shall not exceed the ASCE 7-16 allowable story drift when subject to seismic design forces including torsion and accidental torsion. This check shall include shear and bending deformations of the diaphragm, computed on a strength level basis. The SDPWS drift check applies at the edges of the structure for all open-front structures, with or without torsional irregularities. This example is symmetric, and the load case checked will create the maximum drift at the right cantilever.



δ_{RT} = Transverse component of rotation
δ_{RL} = Longitudinal component of rotation
δ_D=Diaphragm displacement
δ_T = Translational displacement

Drift consists of three components: diaphragm translation and diaphragm rotations from wall displacements and in-plane diaphragm deformations, as shown in SDPWS Figure C4.2.5B.

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

The deflection from translation and rotation are based on the response of the shear walls under the rigid diaphragm assumption. The RDA distribution of loads from longitudinal loading with ρ = 1.0 and Ax = 1.25 is presented in *Table 5*. Using the nominal stiffness values to calculate the deflections of the shear walls:

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

Calculating the longitudinal translation:

$$\delta_{\text{Translation}} = \frac{(\delta_2 + \delta_3)}{2} = \frac{(0.192 + 0.216)}{2} = 0.204 \text{ in}$$

Calculating the diaphragm displacement at grid 4 due to rigid diaphragm rotation, relative to the center of rigidity:

$$\delta_{RL} = \frac{\delta_A + \delta_B}{W'}(L' + 3) = (0.081+0.081)(38)/40 = 0.154 \text{ in}, \ \delta_{RT} = 0.081 \text{ in}$$

Example diaphragm deflection right cantilever:

$$\delta_D = \frac{3vL'^3}{EAW'} + \frac{0.5vL'}{1000G_a} + \frac{\Sigma A_C X_C}{W'} = \frac{3(233.2)(35)^3}{1700000(16.5)40} + \frac{0.5(233.2)35}{1000(25)} + 0.075 = 0.265 \text{ in}$$

The method for checking drift and torsional irregularities should include the diaphragm deflections as well as the rotational translation, δ_{RT}, in the drift values at the edges of the structure.

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

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

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

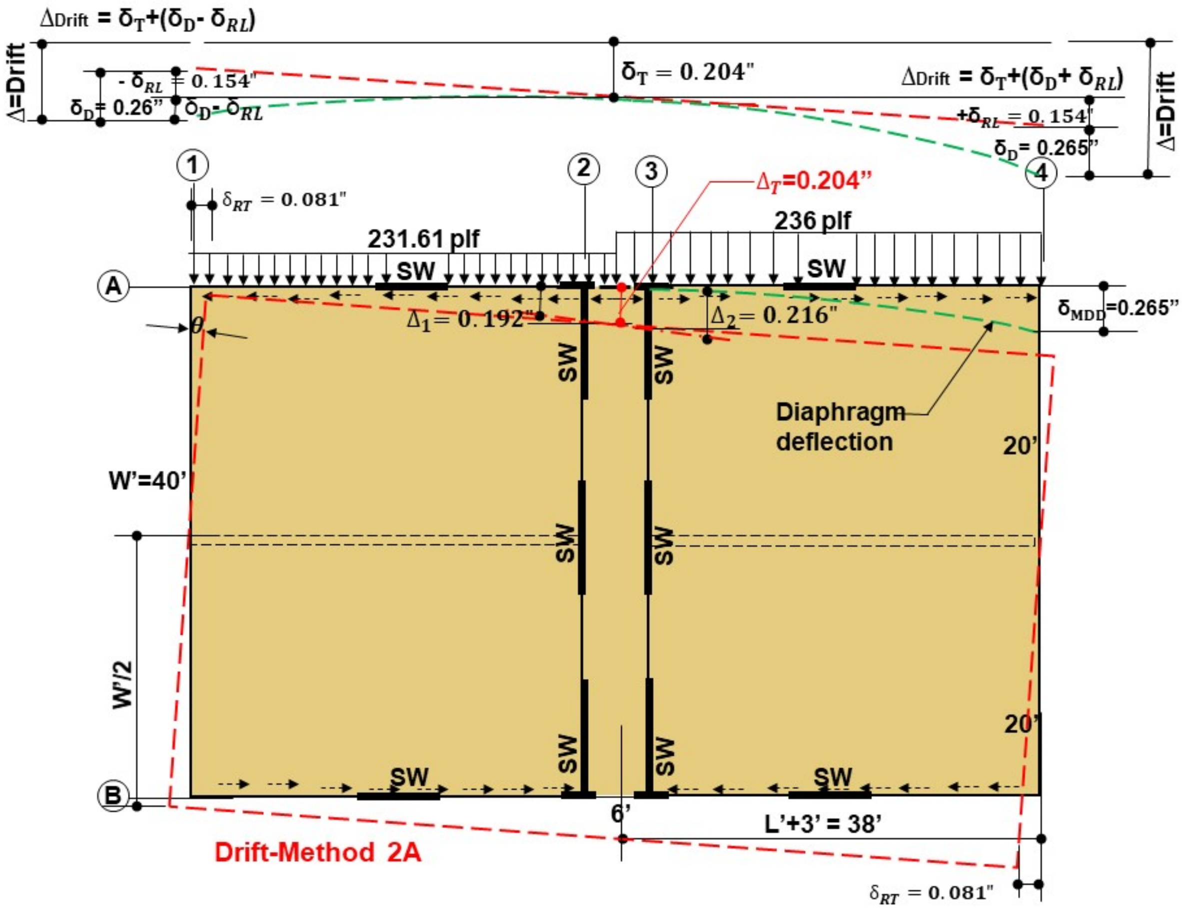


Figure 15. Story Drift

Combining all the terms of the diaphragm deflection at the edge of the structure:

$C_d = 4$, $I_e = 1$

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

With a single-story structure, the story drift equals the displacement.

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	**$0.020h_{sx}$**	**$0.015h_{sx}$**	**$0.010h_{sx}$**

ASCE 7-16 Table 12.12-1 – Allowable Story Drift

In ASCE 7-16 Table 12.12-1, the allowable story drift limit of 0.025 hsx or 0.02 hsx for the example structure depends on the non-structural components and detailing. Under the first category, one of the requirements is that interior walls, partitions, ceilings, and exterior walls can accommodate the higher story drift limit. Most sheathed wood-framed walls can undergo the 2.5% drift level while providing life safety performance at the seismic design level; however, window systems or architectural finishes may not be able to perform. The selection of the higher 2.5% drift limit should be taken only with consideration of the non-structural wall and window performance possibly under consultation with the architect and the jurisdiction having authority over the project. Otherwise, the 2% drift limit requirements should be used. For this example:

$0.025\ h_{sx} = 0.025(10)(12) = 3.0 \text{ in} > 2.51 \text{ in} \therefore$ Drift OK for 2.5% limit, and
$0.02\ h_{sx} = 0.02(10)(12) = 2.4 \text{ in} < 2.51 \text{ in} \therefore$ Drift Not OK for 2% limit

Note: If the drift limit only needed to be checked at the center of mass, we see 4(0.204 in) = 0.816 in < 0.02 hsx < 0.025 hsx. Without showing the calculation, it can be shown that using an unblocked diaphragm would not meet the 2% drift level. If drift values needed to be reduced, the portion of the drift coming from the deflection of the corridor walls and diaphragm deflection are roughly equal, which suggests that stiffening the walls or the diaphragms could help reduce the story drift at the building edge. SDPWS Figure C4.2.5B notes that the deflection amplification factor, C_d, applies to the drift calculations at the edges of the structure.

> Applying C_d to the cantilever diaphragm provisions may be somewhat of a surprise as there is a common belief among designers that ASCE 7 minimum diaphragm design load provisions for diaphragms results in a diaphragm which responds approximately elastically during a design-level earthquake, while inelastic behavior is isolated in the vertical elements. For structures with wood structural panel shear walls and diaphragms with significantly less capacity in the walls than the diaphragms, much more inelastic deformation is expected in the shear walls. However, for linear elastic analysis as typically performed for wood structures, quantifying the amount of inelastic deformations in the walls vs. diaphragms is challenging and the use of the C_d for wood structural panel shear walls to amplify wood structural panel diaphragm deflections is an acceptable approach.
>
> For more information on the current thoughts on inelastic behavior of diaphragms, see the NEHRP *Recommended Seismic Provisions for New Buildings and Other Structures*, 2015 Edition[8], and the *Volume II, Part 3 Resource Papers*[9] available at https://www.fema.gov/media-library/assets/documents/107646

If the allowable drift limit is exceeded, additional stiffness must be provided in either the diaphragm or the shear walls:

a. **Diaphragms**: Increasing the stiffness of the diaphragm is not as easy as it might seem using the three-term deflection equation. Although increasing nail size, decreasing nail spacing or increasing sheathing thickness can increase the unit shear capacity, it will not, in many cases, increase the diaphragm stiffness in the tabulated G_a. Observations from SDPWS Table 4.2A show that the apparent shear stiffness diminishes as you decrease the nail spacing from a 6 inches o.c. boundary nailing to 4 inches o.c., increases between 4 inches o.c. to 2.5 inches o.c. boundary which is still less than the 6 inches o.c. nailing, and increases more at 2 inches o.c. boundary nailing. The reason for this is that under a given nailing capacity shown in the table, the maximum load per nail used to determine the nail slip is based on "other edges" and not the boundary edges. The "other edges" nail spacing for the 4/6/12 nail pattern is the same as the 6/6/12 nail pattern, but the shear load per nail increases, which increases the nail slip[10].

b. **Shear walls**: Contrary to the diaphragm, decreasing the nail spacing on the shear walls will increase the wall stiffness when using SDPWS Eq. 4.3-1 in SDPWS Table 4.3A, the apparent shear stiffness, G_a, increases as the nail spacing decreases.

c. **Other options to increase stiffness**:
   - Lengthen or add shear walls
   - Increase the stiffness of the walls by sheathing on both sides
   - Increase the size (and stiffness) of hold downs
   - Increase the number of boundary studs

d. **Calculation Method:** A final option which may increase the calculated system stiffness and reduce the deflections is to use the four-term deflection equations for the shear wall and diaphragm deflections. This is particularly useful to reduce deflections when the force demands are significantly less than the force capacity. *Figure 16,* below, based on SDPWS Figure C4.3.2, shows conceptually how the four-term deflection equation can result in a deflection (δ_4) significantly below that resulting from the three-term equation (δ_4) at low demand to capacity ratios. When calculating the diaphragm flexibility using the four-term equation approach, or performing semi-rigid diaphragm analysis, it is advisable to use the four-term equations for all shear walls and diaphragms to avoid introducing an artificial bias in the results by selectively combining three-term and four-term deflection calculations.

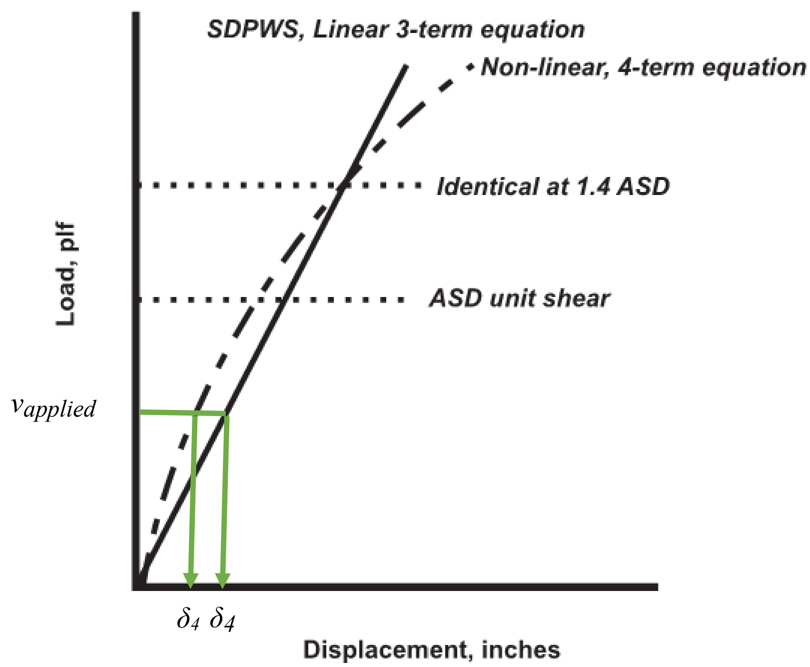


Figure 16. Comparison of Three-Term vs. Four-Term Deflection Equation, Shear Applied to Walls

Figure 17 shows a similar comparison using the applied force vs. shear wall deflection for one of the 10-ft-long walls in this example considering an assumed 200 plf applied gravity load restraining overturning. Note that, at 100 plf of applied shear, the gravity loading on the wall is in balance with the horizontal load resulting in zero uplift of the tension side of the wall. When uplift initiates with greater than 100 plf of applied shear, there is an idealized instantaneous slip from the combined effects of any slack in the system from hold down slip and any included shrinkage effects. With further increased lateral loading, the hold down introduces additional flexibility resulting in a softer stiffness above the uplift point than below. The continued softening of the four-term equation beyond the transition point is a result of the non-linear relationship between the applied lateral loads and the nail slip term, e_n. For this specific wall example, after the initiation of uplift, the four-term equation deflection is up to 30% less than the three-term equation deflection.

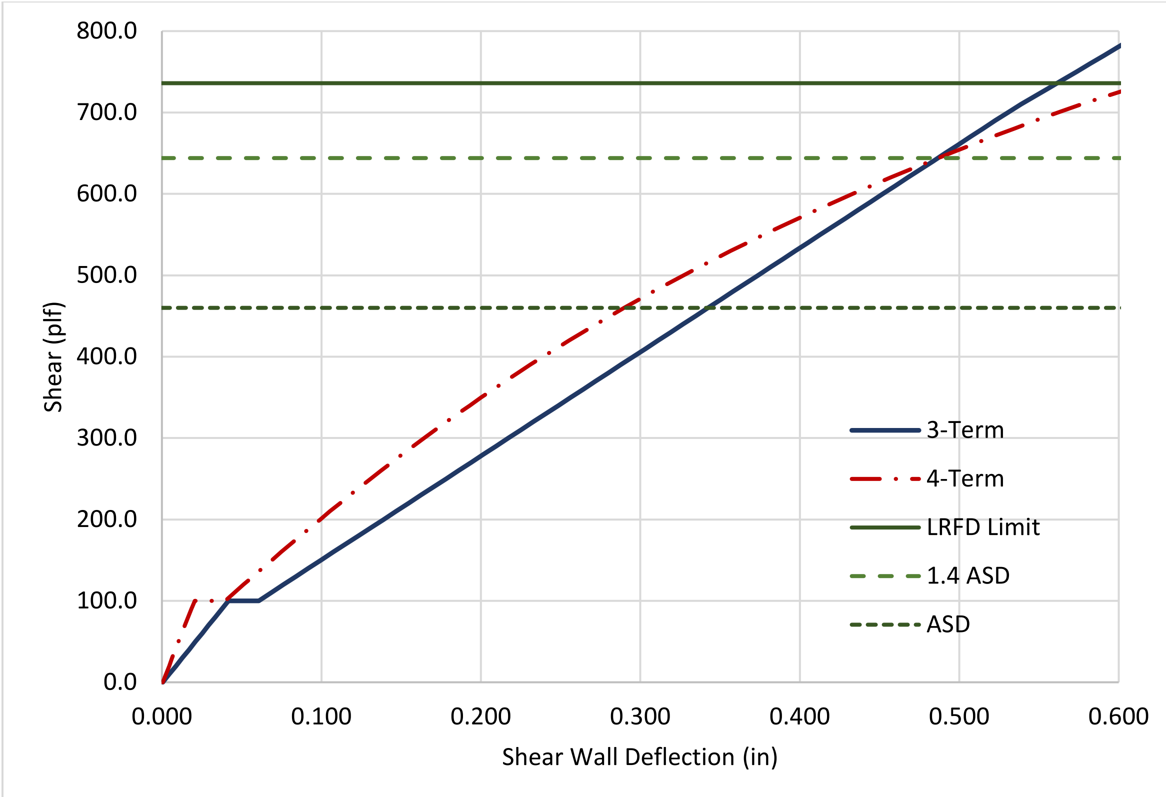


Figure 17. Comparison of Three-Term vs. Four-Term Shear Wall Deflection Equations

An alternative to applying C_d to the diaphragm that is implied by ASCE 7-16 and the 2015 NEHRP provision is to design the diaphragms to exhibit elastic response under estimated design earthquake. This can be accomplished using the alternative seismic design forces for diaphragms in ASCE 7-16 12.10.3 with an R_s = 1.0 resulting in increased force demands corresponding to an expected elastic diaphragm response for the design-level earthquake. If such an approach is taken, the shear wall deflections to the MLFRS design loads would be amplified by C_d as usual; however, the diaphragm deformations would not need be amplified. Such an approach is not within the design standards and would need to be undertaken in consultation with and approval by the jurisdiction and any structural reviewers.

The alternative diaphragm design forces, F_{px}, per ASCE 7 12.10.3 with R_s = 1.0 in this example results in a total diaphragm design force of 53.23 kips compared to the current diaphragm design force of 23.07 kips. While this is a large increase in force demands, the current demand to capacity ratio of the diaphragm in the longitudinal direction is 200 plf / 280 plf = 0.71, as shown in Section 7.2.1, so the construction details for the higher loads are achievable.

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 =35,000 lbs/in depth SDPWS Table C4.2.2A

using 15/32 in structural sheathing, 32/16 span rating, 4-ply

v = 233.2 plf

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

The 2 in the numerator accounts for the top and bottom chord slip.

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

Drift $\Delta_4 = \sqrt{(0.204 + 0.245 + 0.154)^2 + (0.081)^2} = 0.608$ 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 A_x decreases (See *Section 7.6.1*).

7.6 Verification of Torsional Irregularity (STR)

An early assumption of this design was that the structure has an ASCE 7 and SDPWS torsional irregularity, but not an ASCE 7 extreme torsional irregularity.

> ASCE 7-16 Table 12.3-1 Type 1a Torsional Irregularity:
> *Torsional irregularity is defined to exist where the maximum story drift, computed including accidental torsion with A_x =1.0, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semi-rigid.*

This is essentially the same definition of torsional irregularity found in SDPWS. Section 4.2.5.1 Table 12.3-1 defines a horizontal structural irregularity Type 1b – Extreme Torsional Irregularity as the same criteria with a limit of 1.4 instead of 1.2. In Section 12.3.3.1, an extreme torsional irregularity, horizontal irregularity Type 1b, is allowed in structures assigned to Seismic Design Categories B, C, and D, but not in E, or F.

ASCE 7-16 Table 12.3-1, Type 1a and 1b irregularities note that A_x = 1.0 when checking for torsional irregularities. Therefore, ρ = 1.0 and A_x = 1.0 are used in the torsional irregularity checks.

Longitudinal Loading	e=3.8’, T = 67522.2 ft. lbs. ρ=1.0, Ax=1.0									
Grid Line	kx	Ky	dx	dy	kd	kd²	Fv		FT	Fv+FT
2	43.54		3		130.63	391.89	8884.5		-422.2	8462.3
3	43.54		3		130.63	391.89	8884.5		422.2	9306.7
A		25.14		20	502.74	10054.73			1624.7	1624.7
B		25.14		20	502.74	10054.73			-1624.7	-1624.7
Σ	87.09	50.27			J=	20893.23	17769			

Corridor Walls

Walls at Grid lines A & B

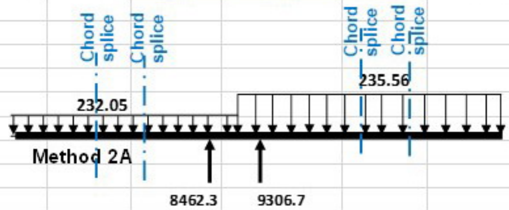
Table 7. RDA Load Distribution for Torsional Irregularity Check

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

lincludes 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									

Table 8. Diaphragm Deflections for Torsional Irregularity Check

The RDA results shown in Table 7 and shear wall deflections for this loading condition are shown below. The diaphragm deflections are shown in Table 8 including the accidental torsion, with $A_x = 1.0$.

Rigid diaphragm center of mass translation:

$\delta_{SW2} = 0.194$ in displacement at line 2

$\delta_{SW3} = 0.214$ in displacement at line 3

$\delta_T = \frac{(\delta_{SW2}+\delta_{SW3})}{2} = 0.204$ in displacement at center of mass (diaphragm translation)

Rigid diaphragm rotation:

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

Using similar triangles, the longitudinal displacement at lines 1 and 4 from rigid diaphragm rotation is:

$\delta_{RL} = \frac{2\delta_{SWA,B}(L'+3')}{W'} = 0.124$ in

Diaphragm deformations:

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

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

The method for checking a torsional irregularity can include the diaphragm deflections as well as the rotational translation, δ_{RT}, in the drift values at the edges of the structure.

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

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

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

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

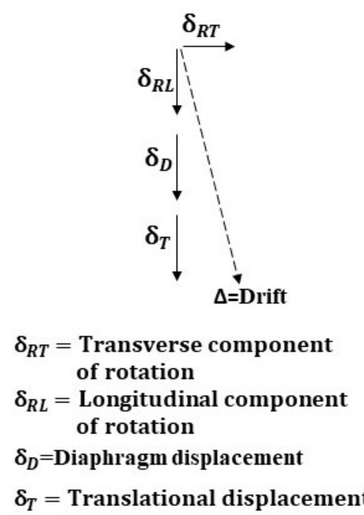
Checking the irregularity criteria of ASCE 7-16 Table 12,3-1:

0.592 > 1.2(0.467) = 0.56 in, ∴ Horizontal torsional irregularity Type 1a exists.

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

The building has a torsional irregularity (Type 1a) but not a (Type 1b), as originally assumed.

ASCE 7-16 Table 12.3-1 Irregularity Requirements: Type 1a and 1b irregularities both trigger sections 12.3.3.4 and 12.8.4.3 for SDC D. The former section requires a 25% increase in the connections of the diaphragm to the vertical elements and collectors; and, the collectors and their connection to the vertical force-resisting elements. Exception – Forces calculated using the seismic load effects, including over-strength of Section 12.4.3 need not be increased. The diaphragm shears do not have to be increased 25%. Section 12.8.4.3 requires an amplification of the accidental torsion.



7.6.1 Calculate Amplification of Accidental Torsional Moment

Amplification of accidental torsion is intended to account for an increase in torsional moment caused by potential yielding of the perimeter SFRS (i.e., walls, shifting of center of rigidity) leading to dynamic torsional instability. When computing A_x for each level, absolute displacements, δ_x, of each level are used, not story drifts, Δ. The displacements used to find A_x are calculated using $\rho = 1.0$ and $A_x = 1.0$.

> ASCE 7-16 12.8.4.3 Amplification of Accidental Torsional Moment.
> *Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:*
>
> $$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad 12.8\text{-}14$$
>
> *where*
> *δ_{max} = maximum displacement at level x computed assuming $A_x = 1$, and*
> *δ_{avg} = average of the displacements at the extreme points of the structure at level x computed assuming $A_x = 1$.*

The torsional amplification factor, (A_x), shall not be less than 1 and is not required to exceed 3.0.

As a one-story structure, the displacements, δ, equal the story drifts, Δ.

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

At this point, consideration could be given to revising the calculations with the lower A_x in order to reduce the forces to the structure.

7.7 Verification of Redundancy Factor (STR)

The plan layout shown in *Figure 2* suggested redundancy factor of 1.3 could apply and was assumed in the design. While nothing is required to be verified to use rho = 1.3, an example verification is provided.

ASCE 7 requires the following for structures assigned to SDC D through F:

> 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 is met, whereby ρ is permitted to be taken as 1.0. A reduction in the value of ρ from 1.3 is not permitted for structures assigned to Seismic Design Category D that have an extreme torsional irregularity, Type 1b. Seismic Design Categories E and F are not specified because extreme torsional irregularities are prohibited (see Section 12.3.3.1).*

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

b. *Structures that are regular in plan at all levels provided that the seismic force-resisting systems consist of at least two bays of seismic force-resisting perimeter 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 shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height, hsx, for light-frame construction.*

The example structure does not have an extreme torsional irregularity or two bays of seismic force-resisting perimeter framing on each side of the structure in each orthogonal direction and is not considered "regular in plan" in the longitudinal direction, so condition "b" does not apply. The calculated condition "a" will be considered.

ASCE 7-16 *Table 12.3-3* for shear walls or wall piers with a height-to-length ratio greater than 1.0 notes:

> *Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0* ***within*** *any story, or collector connections thereto, would not result in more than a 33% reduction in story strength; nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b). The shear wall and wall pier height-to-length ratios are determined as shown in Fig. 12.3-2.*

There are no walls that have an aspect ratio greater than 1:1 in the longitudinal direction. However, the removal of one shear wall at line A, which has an aspect ratio of 1.25:1, effects torsional resistance and is therefore removed for the extreme torsional irregularity check as shown in *Figure 18*. If one shear wall symbolically fails (is removed), the center of rigidity would shift towards grid line B which would require re-calculation of the distribution of forces using RDA. Although this relocation of the center of rigidity creates inherent torsion in the transverse direction, symmetry between the center of rigidity and center of mass remains in the longitudinal direction.

The elastic method shall be used to verify the redundancy.

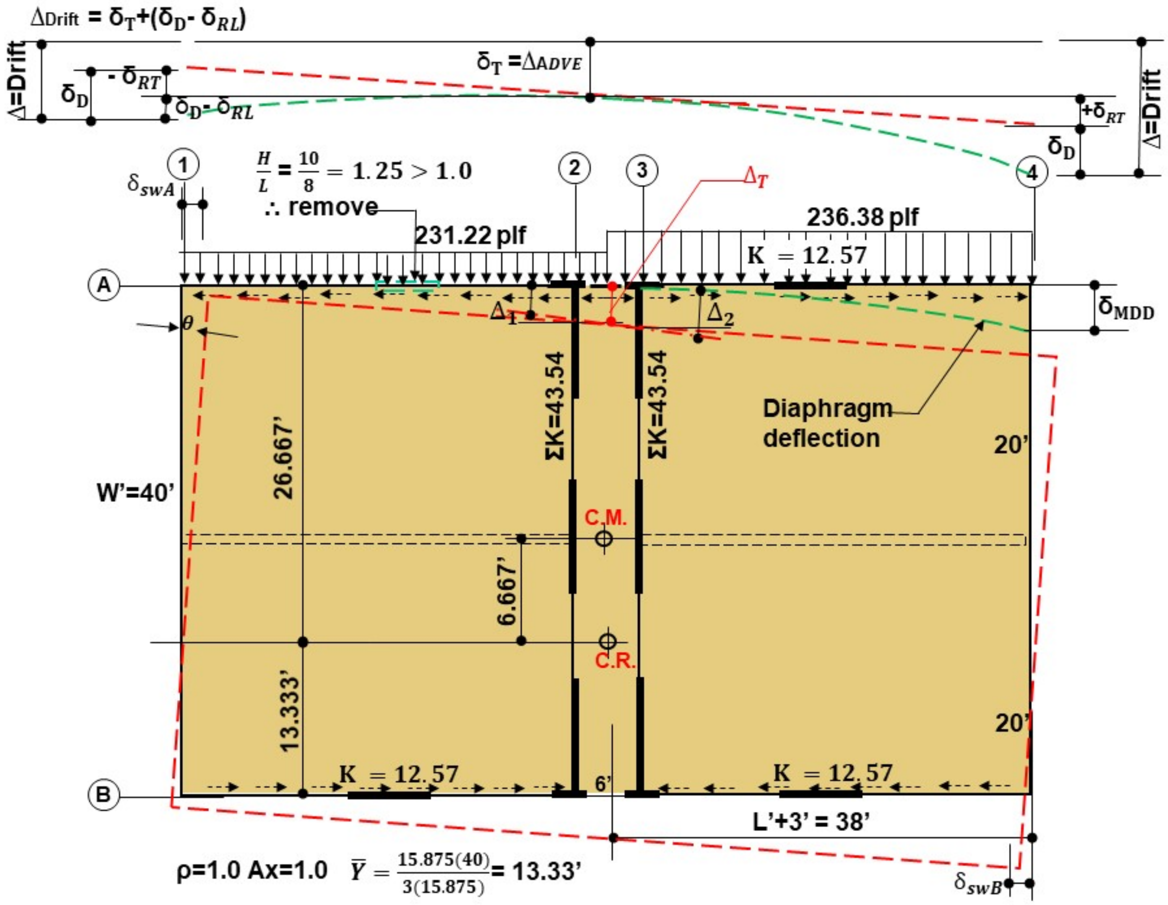


Figure 18. Verification of Redundancy

Spreadsheet Results: $\rho = 1.0$ and $A_x = 1.0$

$e = 3.8$ ft

$F_{A,\,B} = 1{,}594.9$ lbs

$F_2 = 8{,}263$ lbs

$F_3 = 9{,}506$ lbs

$\delta_{SW2} = 0.190$ in

δ_{SW3}=0.218 in

$\boldsymbol{\delta_{T\ Aver}}$ = 0.204 in translational displacement of center of mass

$\delta_{RT\ A}$ = 0.127 in, Note: Single 8 ft wall

$\delta_{RT\,B}$ = 0.063 in, Note: Two 8 ft walls

Use $\delta_{RT\,A} = \delta_{RT}$ as greater translational displacement along A than B

$\delta_{RL} = \frac{(0.127)(38)}{26.667}$ = 0.181 in longitudinal component of the rotational displacement

$\delta_{Diaph,1} = 0.260$ in

$\delta_{Diaph,4} = 0.256$ in

Torsional Irregularity Check

Combining to find the worst-case drift at a corner, including diaphragm deformation:

$$\text{Drift}_{\Delta_4} = \sqrt{(0.204 + 0.260 + 0.181)^2 + (0.127)^2} = 0.657 \text{ in}$$

$$\text{Drift}_1 = \sqrt{(0.204 + 0.256 - 0.181)^2 + (0.127)^2} = 0.307 \text{ in}$$

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

Checking the irregularity criteria of ASCE 7-16 Table 12,3-1:

0.657 < 1.4(0.482) = 0.674 in, ∴ Horizontal torsional irregularity Type 1b does not exist and ρ = 1.0.

For the purpose of determining redundancy, an extreme torsional irregularity, Type 1b, does not exist as the results of the removal or failure of a shear wall on line A. Since none of the corridor walls must be removed because their aspect ratio does not exceed 1:1, there is no reduction in story strength in the direction of the load. Condition "a" has been satisfied and ρ = 1.0.

7.8 Calculate Corridor Collector Forces

Collectors must be designed in accordance with ASCE 7-16 Section 12.10.2. Since this example is in SDC D and the structure is all light-framed wood shear walls, the exception of section 12.10.2.1 controls the design of the collectors and the over-strength factor does not need to be applied.

The calculation of the collector forces at the corridor wall lines are shown in *Figure 19* using ρ = 1.3 and A_x = 1.25. Before collector force diagrams can be calculated, all shears along the corridor lines must be converted to net shears. The net shears at the shear walls are equal to the shear wall shears minus the diaphragm transfer shears on each side of the shear wall. The forces at the end of the shear walls are equal to the net shears at the wall multiplied by the length of the wall. The collector forces between the shear walls are equal to the sum of the transfer shears on each side of the wall line multiplied by the collector lengths.

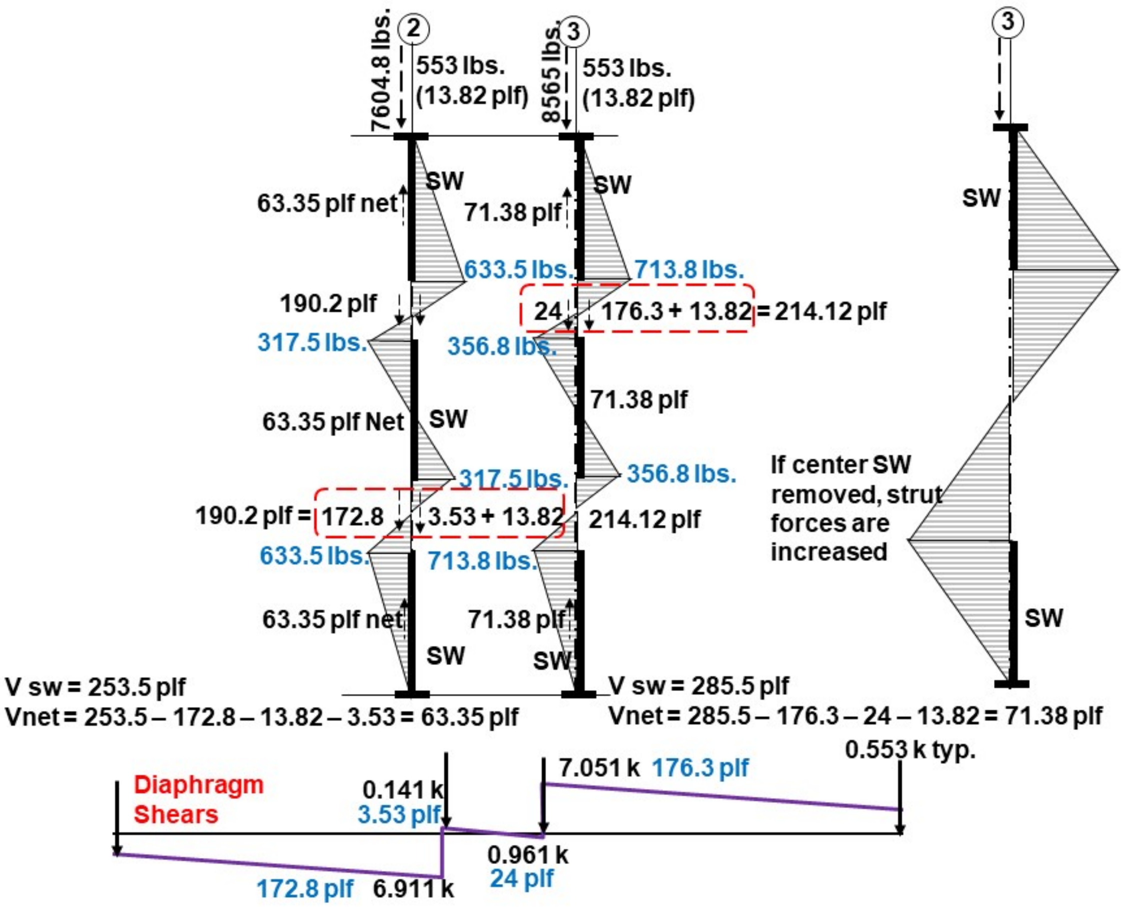


Figure 19. Corridor Collector Forces

8 Transverse Diaphragm Design (ASD)

Preliminary design considerations outlined in *Section 4.3* assumed that the diaphragm in both directions could be idealized as rigid. It has already been established that the diaphragm is rigid in the longitudinal direction. This will be verified for the diaphragm in the transverse direction in *Section 8.2*. It was also assumed that torsional irregularity Type 1a does not exist in this direction and that redundancy could be an issue. At this point, it is unclear if drift or redundancy will be issues, due to the short shear walls along lines A and B. The same assumptions for redundancy used for longitudinal loading will be used for the application of loads in this direction. Therefore, $\rho = 1.3$ and accidental torsion will not be applied, except for the torsional irregularity check as required by ASCE 7-16 Table 12.3-1.

For loading in the transverse direction, current practice for light-frame construction commonly assumes that WSP-sheathed diaphragms are flexible for the purpose of distributing horizontal forces to shear walls. Compliance with ASCE 7-16 Section 12.3.1.1 allows diaphragms in light-frame structures meeting all the following conditions to be idealized as flexible:

1. All light-frame construction
2. Non-structural concrete topping ≤ 1 ½ in over WSP
3. Each element of the seismic line in the vertical force-resisting system complies with the allowable story drift of Table 12.12-1

The structure in the transverse direction meets all the conditions noted above, except for item 3, which has yet to be verified. Therefore, the diaphragm, at this point, can be idealized as flexible. Flexible diaphragms cannot transfer torsional forces; therefore, accidental torsion does not apply for this flexibility condition either.

8.1 Verification of Shear Wall Design and Deflections (ASD)

Wall sheathing, nailing and hold downs have been maintained from the longitudinal analysis. The strength capcity of the shear walls, in both the longitudinal and transverse direction, was reviewed in Setion 5.6 and found to meet all strength capacity requirements. Diaphragm design strength OK by inspection.

8.2 Check Diaphragm Deflection and Flexibility (STR)

Check Diaphragm Flexibility Condition, ρ =1.0 and A_x = 1.25:

Aspect ratio = $\frac{40}{76}$ = 0.526:1 < 3:1 unblocked and 4:1 blocked$\therefore$ OK

V_A = 9057.6 (ASD) lbs, from spreadsheet assuming rigid diaphragm

$v_{max\ diaph}$=119.2 plf < 464 plf diaphragm shears o.k.

Original selection of sheathing and nailing for longitudinal loading OK, 15/32 in OSB w/ 10d@6 in o.c., vs = 0.8(580) = 464 plf, blocked, G_a=25, Case 3

From computer output (Flexibility):

δ Diaph.= 0.066 in

From spreadsheet: $\Delta_{SW\ A}$= 0.396 in, $\Delta_{SW\ B}$= 0.311 in, 2x Δ_{SWAver}= 0.707 in

0.066 in < 0.707 in $\therefore$ Rigid diaphragm, as initially assumed

8.3 Check Story Drift (STR)

ρ =1.0 and A_x = 1.25 – Continuing with flexible diaphragm assumption, no irregularity:

C_d = 4, I_e = 1

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

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

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

8.4 Check for Torsional Irregularity (STR)

Rigid diaphragm, ρ =1.0 and A_x = 1.0 as required by ASCE 7 Table 12.3-1 (See *Figure 20*):

e = 0.05 (40)(1.0) = 2.0 ft Accidental torsion only

T = V_e = 17769(2.0)(1.0) = 35,538 ft lbs

F_T = 222.1 lbs at corridor walls from rigid analysis

M_R = 222.1(6) = 1,332.6 ft lbs, resisting moment at corridor walls

M_{net} = 35,538 - 1,332.6 = 34,205.4 ft lbs

$$W_T = \frac{34{,}205.4}{20(20)} = 85.51 \text{ plf}$$

$$W_{Unif} = \frac{17{,}769(1.0)}{40} = 444.2 \text{ plf}$$

W_1 = 444.2– 85.51 = 358.7 plf

W_2 = 444.2 + 85.51 = 529.7 plf

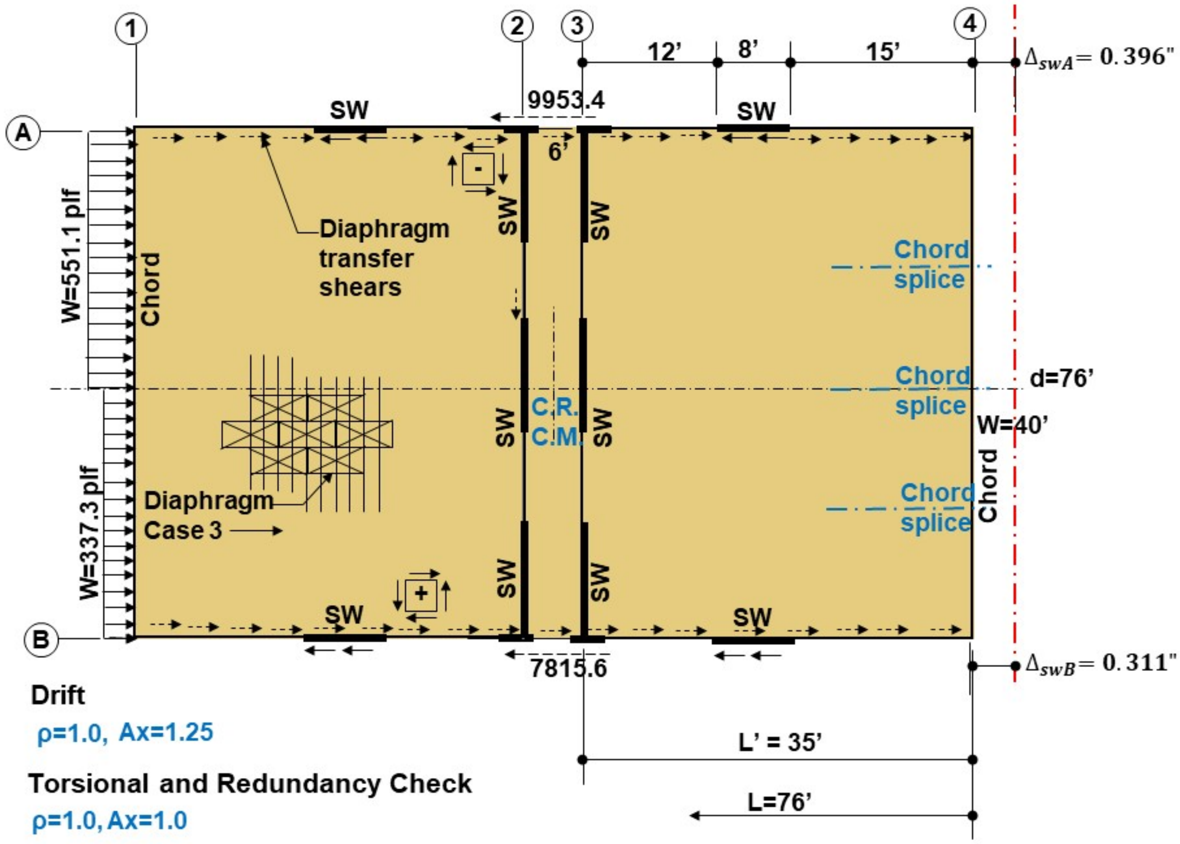


Figure 20. Transverse Loading, Chord Splice Locations and Deflections

δ_{SWA} = 0.387 in From spreadsheet

δ_{SWB} = 0.319 in From spreadsheet

$\Delta_{\text{Average}} = \frac{0.387+0.319}{2} = 0.353$ in

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

8.5 Check for Redundancy (STR)

ρ =1.0, A_x = 1.0

Removal of one shear wall at line A only results in a 25% reduction in story strength < 33% allowed

$$\delta_A = \frac{9{,}739.6}{1{,}000(12.57)} = 0.775 \text{ in From spreadsheet}$$

$$\delta_B = \frac{8{,}029.4}{1{,}000(25.14)} = 0.320 \text{ in From spreadsheet}$$

$$\Delta_{ADVE} = \frac{0.775 + 0.32}{2} = 0.547 \text{ in}$$

0.775 in > 1.4(0.547) = 0.765 in ∴ Type 1b irregularity exists.

A Type 1b torsional irregularity exists in this direction. ∴ ρ = 1.3 as assumed.

9 Other Issues

9.1 Unsymmetrical Plans

Symmetrical plan layouts will not always be available. The plan shown in *Figure 21* has the same shear wall configurations at grid line A as previously designed, but full-length shear walls have replaced the shear walls along grid line B. The increased stiffness of these new walls shifts the center of rigidity closer to grid line B, increasing eccentricity and creating inherent torsion in the structure which needs to be included in rigid or semi-rigid diaphragm analysis.

When the plan is unsymmetrical, the rotation will occur about the center or rigidity and the rotational term of the deflection equation then becomes:

$$\frac{\delta_{SWA}}{W''} = \frac{\delta_{Rot}}{L' + 3'}$$

$\delta_{Rot} = \frac{\delta_{SWA}(L' + 3')}{W''}$ longitudinal component only

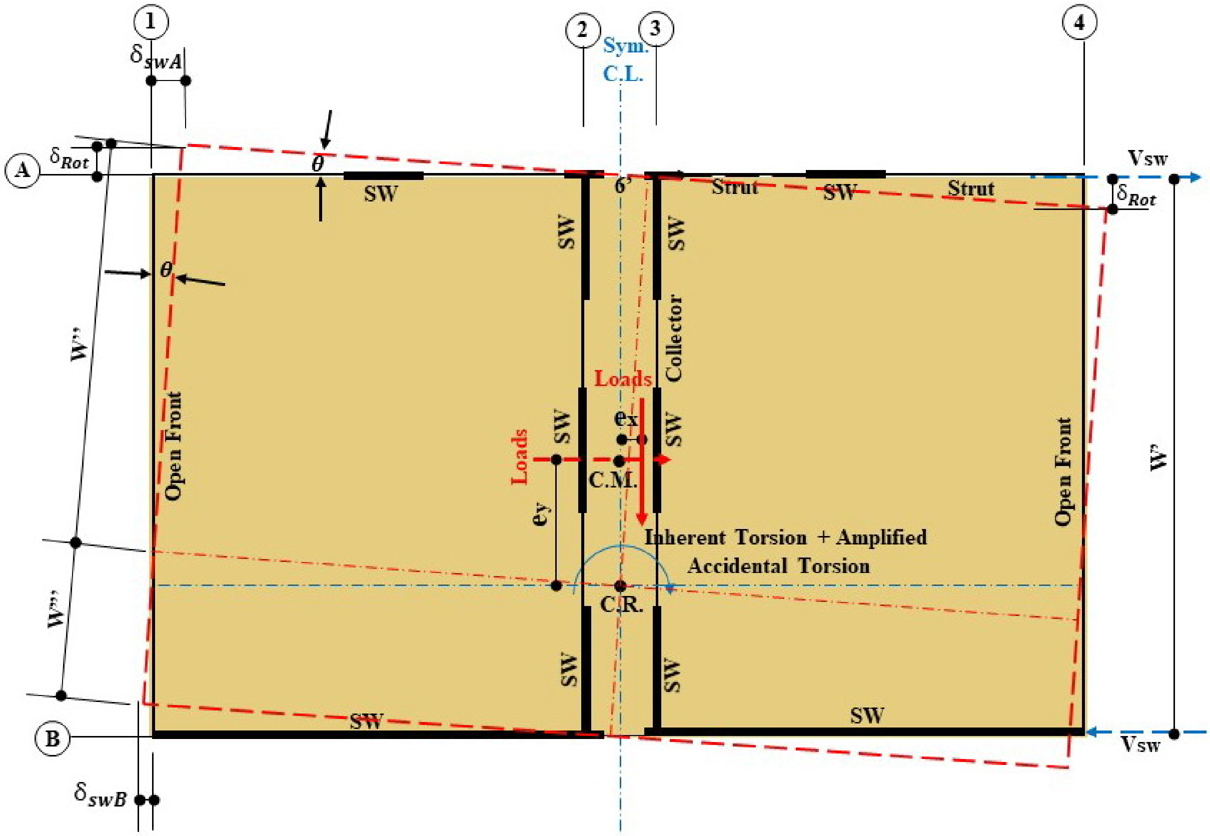


Figure 21. Unsymmetrical Plans

9.2 Full-Length Shear Wall Effects at Grid Lines A and B – Chord Forces

The effects of partial-length shear walls on diaphragm chord forces was explored in *Section 7.2.2*. Transverse shear walls at grid lines A and B, which commonly occur in mid-rise structures, are typically full length from the exterior wall line to the corridor as shown in Plan A of *Figure 4* and line B of *Figure 21*.

To get a complete view of how shear walls along the chord lines affect diaphragm chord forces, a new RDA using the new wall stiffnesses at grid lines A and B will be required. The results of the new force distribution to the walls are shown in *Figure 22.*

F_T = 1,848.1 lbs

$v_T = \frac{1{,}848.1}{76} = 24.32$ plf

Two 35-foot walls will be used.

$v_{sw} = \frac{1{,}848.1}{70} = 26.4$ plf

v_{net} = 26.4 - 24.32 = 2.08 plf net

Line 2 shows that the net unit shears from the rotational forces minus the shear wall shears is minimal, at 2.08 plf. The resulting force these net shears cause at grid lines 2 and 3 is equal to 73 lbs. This force must be added to or subtracted from the bending chord forces to get a final chord forces. If the walls are continuous from grid line 1 to 4 (no door opening), the net shears from rotation would be zero (i.e., rotational unit shears are equal to the shear wall shears), and the chord forces would be caused only by the lateral bending forces. The results show that full-length walls along the diaphragm chord lines have minimal effect on the final chord forces.

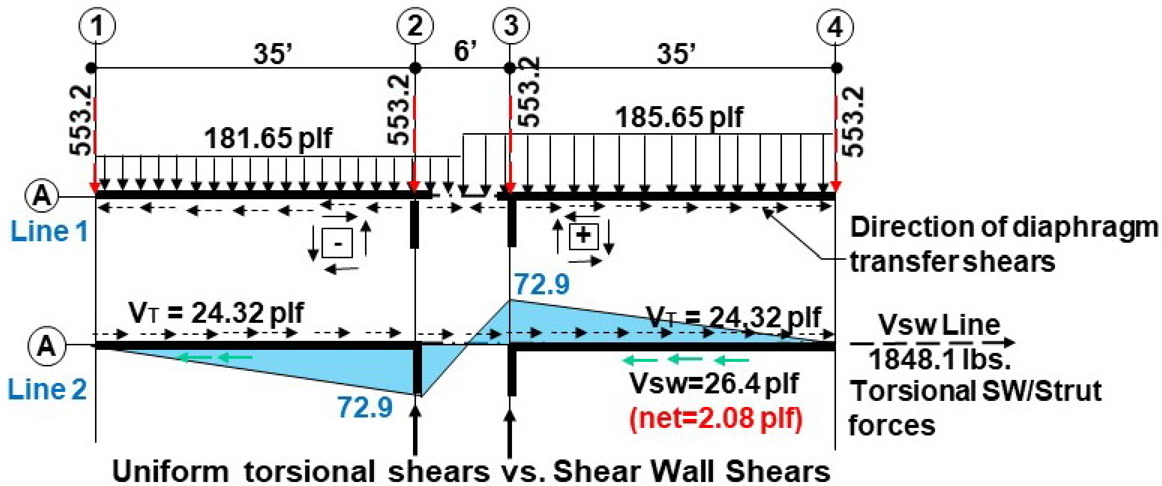


Figure 22. Chord Forces – Full-Length Walls

9.3 Corridor Shear Walls One Side Only

Figure 23 shows an optional layout for using only one side of the corridor as shear walls. Although fewer shear walls are required for this condition, symmetry is lost, and a greater eccentricity is created. This will double the demand on the shear walls at grid line 2 and could increase the torsional forces significantly. For this condition, it could potentially cause drift to be exceeded and an extreme torsional irregularity Type 1b. Increased shear wall stiffnesses would be required to control drift and reduce irregularities.

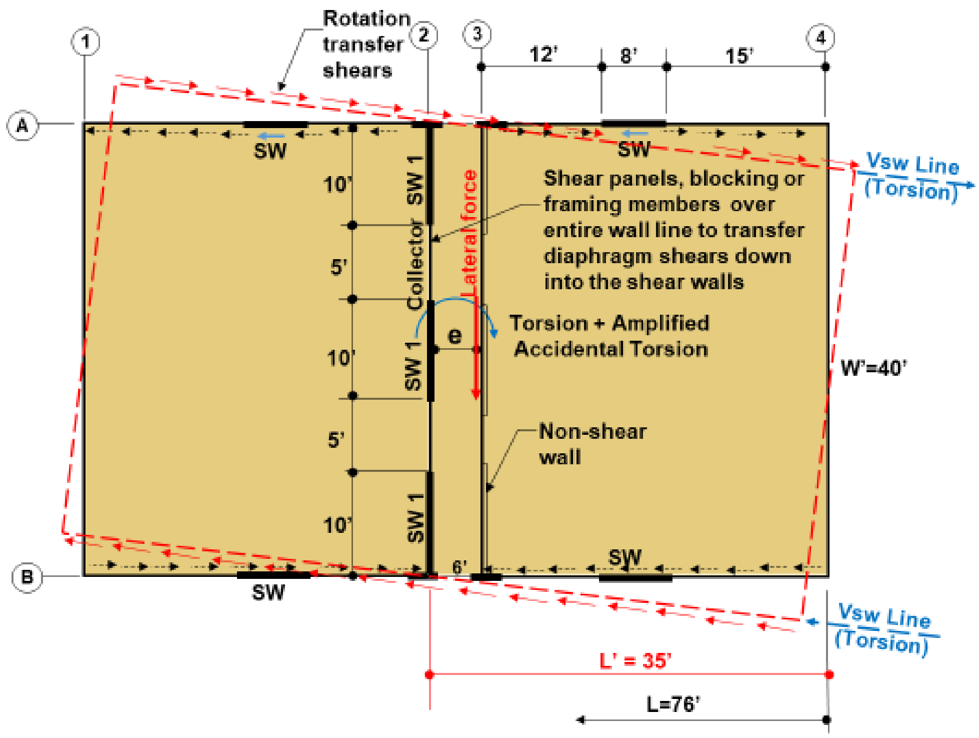


Figure 23. Single-Sided Corridor Shear Walls

9.4 Complex Diaphragm Layouts

Diaphragms with horizontal offsets or large openings can further complicate the analysis. *Figure 24* shows a cantilever diaphragm with horizontal offsets which have become commonplace. Reduced depths in the diaphragms at these offsets change the diaphragm stiffness, which can increase the diaphragm deflection. Under seismic loading, the uniform load to the diaphragm will also vary because of the reduced depths. Because of this, the affected terms in the deflection equation can be modified to account for these changes.

Each offset creates a discontinuity in the diaphragm chord. Connection of the discontinuous chords into the main body of the diaphragm act like a chord splice causing additional chord slip, which must be taken into account.

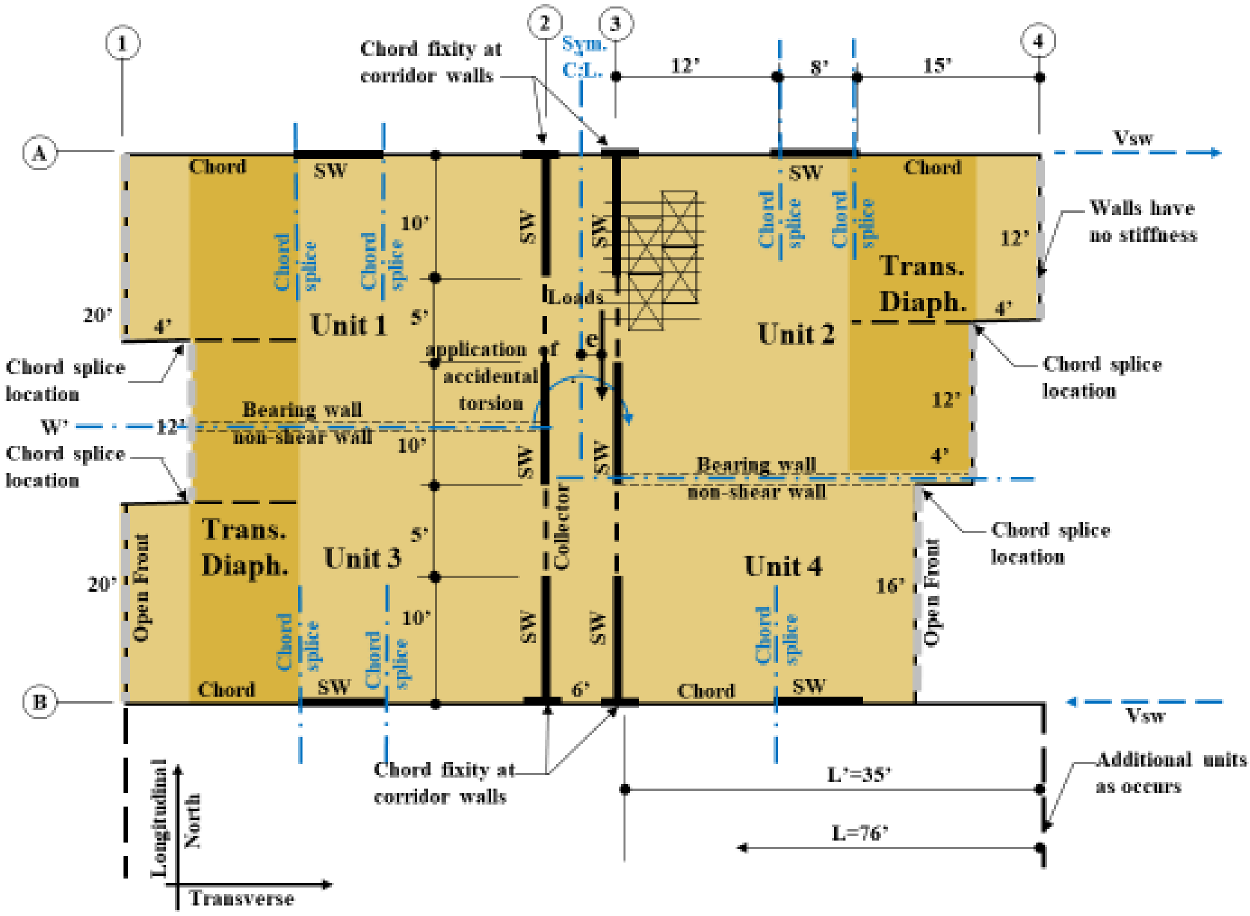


Figure 24. Complex Diaphragm Layouts

9.5 *Mid-Rise Multi-Family*

Nationwide, there has been an increase in the demand for multi-story mixed-use and multi-residential structures. Common configurations include up to five stories of residential use over single or multiple stories of podium, which can include retail, commercial, office and/or parking occupancies. These plans are frequently rectangular shaped with or without exterior shear walls, or they can have multiple horizontal offsets as shown in *Figure 25*. The lateral force-resisting system for the flexible upper portion is often built with wood-framed shear walls sheathed with WSP. Many, if not all, of the transverse walls separating the dwelling units can be used as interior shear walls that resist torsional forces. Lateral forces in the longitudinal direction are typically resisted by the exterior walls and/or corridor walls. Increasingly, due to complicated architectural layouts and limited options for exterior walls, such plans often require an open-front analysis approach.

Additional complications arise when an increasing number of stories are involved. Hand calculations previously presented can be useful as a guideline on how to design these structures. As this example demonstrates, when designing shear walls or diaphragms, or checking diaphragm flexibility, drift, torsional irregularities, redundancy and amplification of the accidental torsion, the values of ρ and A_x vary. These must all be addressed in the engineering models and analysis.

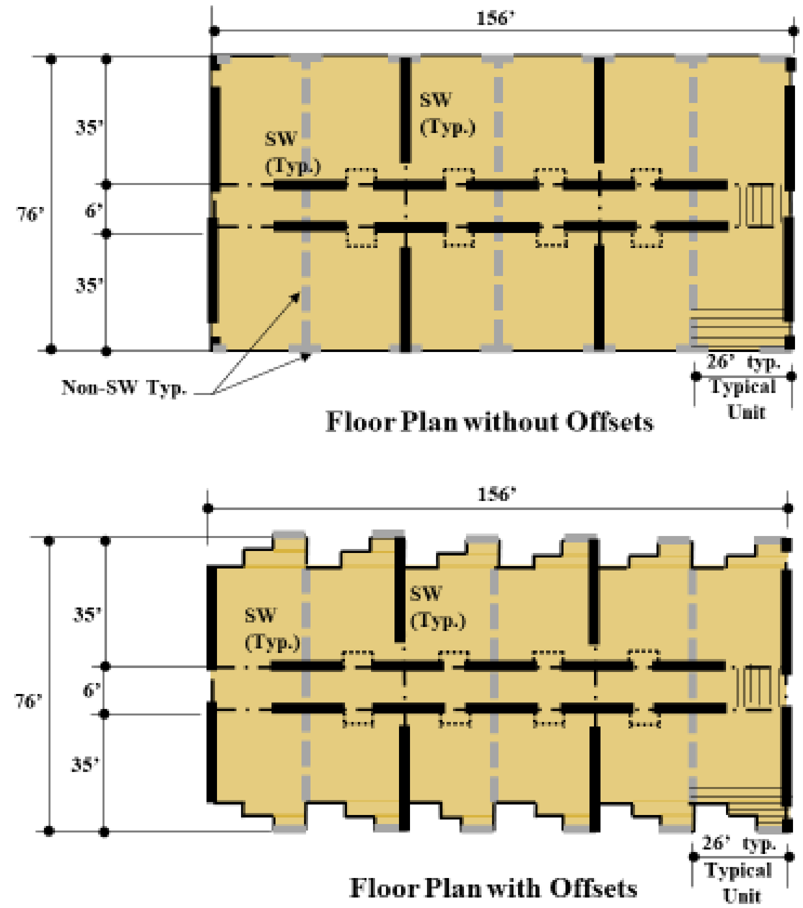


Figure 25. Typical Cantilever Diaphragm Mid-Rise, Multi-Family Floor Plans

10 Conclusions

The example diaphragm proved to fall in the semi-rigid or rigid category in both the longitudinal and transverse direction, leaving the shear walls as the controlling stiffness elements. Minimizing additional iterations caused by stiffness and torsional deficiencies requires considerable judgement in assigning preliminary shear wall construction. The chosen wall construction was based on anticipated drift, redundancy and torsion problems.

Story drift was met in both directions without additional modifications to the shear walls. Torsional irregularity, Type 1a, occurred in the longitudinal direction, but not the transverse direction, therefore A_x is greater than 1.0. An extreme torsional irregularity, Type 1b, occurred in both directions when checking for redundancy, therefore, rho = 1.3 in both directions.

Overall, there still appears to be some room for additional adjustments, if required or desired. The summary of options provided for stiffening up the structure at the end of *Section 7.4* allows opportunities to fine-tune the structural design and potentially make it more efficient.

11 References

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3. American Wood Council (AWC). 2015. Special Design Provisions for Wind and Seismic with Commentary. 2015 ed., Leesburg, VA.
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5. USGS Seismic Design Map Tool, 2015 NEHRP Recommended Seismic Provisions
6. ASCE-41 Seismic Rehabilitation of Existing Buildings
7. NDS Commentary reference-Bendsten, Galligan et.al., Modelling the Stress-Compression Relationships in Wood in Compression Perpendicular-to-grain, Madison, WI, Forest Products Research Society, Forest Products Journal, Vol.29, No.2: 42-48, 1979.
8. NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, 2015 Edition
9. NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, 2015 Edition, Volume II, Part 3
10. APA- the Engineered Wood Association-Report 138 Plywood Diaphragms, Form E315H, Revised July 2000, Tacoma, Washington.

Appendix – Summary of Load Combinations, Rho and A_x

The following summary of requirements are for open front structures with diaphragms that are not flexible.

Condition	Load Combination	Rho	Accidental Torsion	Accidental Torsion, A_x	Include Diaphragm Deflections?	Notes
MLFRS Design: Shear Walls	Basic Seismic: ASCE 7 2.3.6 (LRFD) or ASCE 7 2.4.5 (ASD)	Applies	No, except in torsional irregular structures per ASCE 7 12.8.4.2	As calculated in ASCE 12.8.4.3	No, if RDA justified	
Diaphragm Design Forces: MLFRS	Per MLFRS Design	Applies	Per MLFRS Design	Per MLFRS Design	Per MLFRS design	
Diaphragm Design Forces: Fpx	ASCE 7 12.10-1 to 12.10-3	1.0	No	No	N/A	
Diaphragm Flexibility Checks	Per MLFRS Design @ Strength Level	1.0	Per MLFRS Design	Per MLFRS Design	Yes	
Drift Checks. ASCE 7 12.8.6 and 12.12.1	Per MLFRS Design @ Strength Level	1.0	Per MLFRS Design	Per MLFRS Design	Not at center of mass. Otherwise?	Check at center of mass, except check at edges of structure in SDC C to F with torsional irregularity.
SDWPS 4.2.5.2 Drift Checks	Per MLFRS Design @ Strength Level	1.0 (Implied)	Yes even if not required by ASCE 7	Per MLFRS Design?	Yes, even if RDA justified	Check at each edge of structure.
Torsional Irregularity Checks ASCE 12.3.2 and 12.8.4.2	Per MLFRS Design @ Strength Level	1.0	Yes	$A_x = 1.0$	Unspecified	
Redundancy Check via ASCE 7 Table 12.3-3	Per MLFRS Design @ Strength Level	1.0	Yes	$A_x = 1.0$	Unspecified	Extreme torsional irregularity check when removing shear walls per Table 12.3-3
Nominal Wall Stiffness	D+Eh	1.0	No	No	No	Use a seismic load onto wall equal to the LRFD capacity of the wall or a load greater or equal to the maximum demand from all required capacity checks.