WoodWorks

DESIGN EXAMPLE

Seismic Design of Timber Panelized Roof Structures

2012 International Building Code (IBC) Edition

Developed for WoodWorks by

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Suggestions for Improvement

Comments and suggestions for improvement are welcome and should be e-mailed to WoodWorks at info@woodworks.org.

Overview

Timber panelized roof construction is the dominant form of roof structure along the West Coast of the United States for large flat roof systems. Its simplicity and economy make it the primary choice of developers and contractors of large distribution warehouses, industrial, commercial, and big-box retail buildings. This structural roof system has also been utilized in churches, schools and other building types.

The panelized roof structure was developed in the late 1950s in Northern California to facilitate fast forklift erection of flat roof systems using 4x8 plywood and 4x sawn purlins on an 8-foot module. Faster erection speed and greater worker safety is accomplished by conducting most of the fabrication on the ground and then lifting the large fabricated roof panel into place using forklift equipment. Today, 8-foot wide panel assemblies up to 60 feet long are routinely installed and longer lengths are possible. More than 40,000 square feet can be installed in a single day.

The all-wood panelized system consisting of glue-laminated (glulam) purlins and girders has slowly evolved into the more popular hybrid roof system, which consists of a wood panel diaphragm (plywood or oriented strand board (OSB)) and repetitive 2x or 3x subpurlins, supported on factory installed wood nailers attached to the top chord of open-web steel joists and joist girders (trusses). In concrete tilt-up and masonry wall structures, the roof structure is connected to the interior wall face, allowing the walls to extend above the roof as a parapet.

Warehouse and retail development trends have been toward larger and taller buildings with more clear-space and more clear-height. These trends are placing more lateral force demands on the wood roof diaphragms to span farther with higher shear stresses and greater horizontal deflections. This design example illustrates a large concrete tilt-up building in a seismically active area producing large diaphragm and wall anchorage forces. Special high-load diaphragm designs are used in conjunction with a hybrid style of the panelized roof structure.

Disclaimer

Codes and Reference Documents Used

2012 International Building Code (IBC)

2010 American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

2012 National Design Specification® (NDS®) for Wood Construction – ASD/LRFD

2008 Special Design Provisions for Wind and Seismic (SDPWS-2008)

Given Information

BUILDING USE: Warehouse storage and/or manufacturing

ROOF STRUCTURE: Panelized hybrid roof structure consisting of wood structural panel sheathing over repetitive wood subpurlins, supported by open web steel joists and joist girders

WALL STRUCTURE: Concrete tilt-up wall panels (site cast precast)
Top of wall at 37 feet above finish floor

SEISMIC FORCE RESISTING SYSTEM: Large flexible wood diaphragm supported by intermediate precast concrete shear walls (bearing wall system)

FIRE PROTECTION SYSTEM: Automatic fire sprinkler system
60-foot sideyard clearances on all four sides of building

Figure 1. *Plan View*

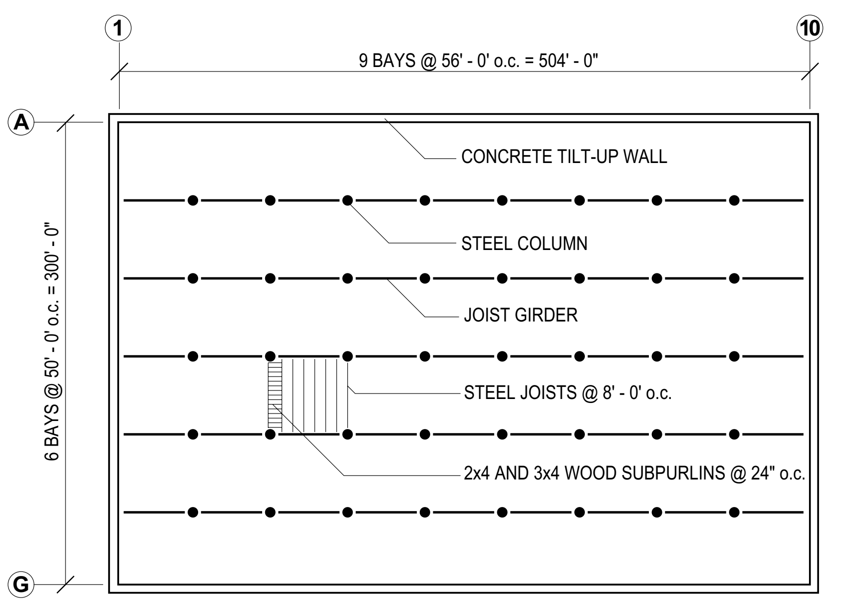


Figure 2. *Building Section*

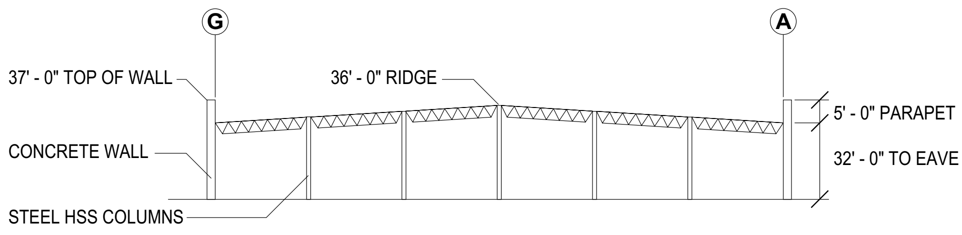
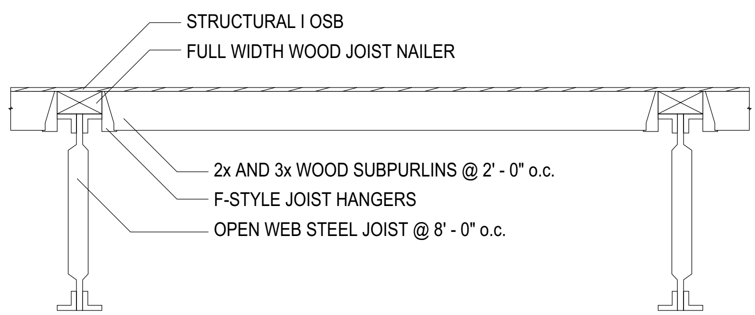


Figure 3. *Enlarged Roof Section View*



ROOF WEIGHTS

Roofing + re-roof	5.0 psf
Sheathing	2.0
Subpurlins	1.0
Steel Joists	2.0
Joist Girders	1.5
Sprinklers	2.0
Miscellaneous	1.5
Dead load	15.0 psf
Live load	20.0 psf (reducible)

WALL WEIGHT

Thickness = 9¼-in concrete

Weight = (150 pcf)(9¼ in)/12 = 116 psf

Height = 37 ft above finish floor surface

WOOD MATERIAL SPECIFICATION

Sawn Dimensional Lumber: Douglas-fir/Larch (DF/L), S-DRY

Sheathing: Structural I OSB in conformance with DOC PS-2 (15/32 in typical)

LATERAL LOADS

Seismic: Mapped Spectral Accelerations

S_S = 1.5g (short period)

S_1 = 0.6g (1-second period)

Risk Category II (ASCE 7-10 Table 1.5-1)

Site Class D

Wind: Assumed not to govern for this example

Outline

This design example illustrates the following selected parts of the design process:

1 – Building Size Limitations

2 – Seismic Force Coefficients

3 – Main Diaphragm Design

4 – Diaphragm Deflection Check

5 – Subdiaphragm Design

6 – Continuous Tie Design

7 – Other Issues of Importance

CALCULATIONS AND DISCUSSION

Part 1 – Building Size Limitations

1a. Structural/Seismic Height Limitation

ASCE 7-10 limits the maximum height of certain seismic force resisting systems (SFRS) depending upon the Seismic Design Category (SDC) of the building (ASCE Section 12.2.1). In high seismic areas, buildings frequently are assigned SDC D or higher, and Table 12.2-1 limits Intermediate Precast Shear Wall systems to 40 feet. Footnote *k* in this table permits the height limit to be increased to 45 feet for single-story storage warehouse facilities. The 40-foot and 45-foot maximum heights for SDC D, E and F were selected during the code's development process to coordinate with the height limits of storage buildings utilizing Early Suppression Fast Response (ESFR) fire suppression systems now commonly specified. This will be further discussed in Part 7. Because this building example involves both manufacturing and warehouse distribution storage, the 40-foot height limit is applicable.

The actual height is measured from the base to the top of the seismic force resisting system. ASCE 7-10 Section 11.2 defines the base of the structure as "the level at which horizontal seismic ground motions are considered to be imparted to the structure." Typically in tilt-up concrete buildings, the concrete slab on grade is utilized as a deadman to transfer lateral loads out of the shear walls through the slab and into the soil. The height of the shear wall is measured to the roof line, not the top of parapet, thus the building's height is approximately 36 feet to the roof ridge at the east and west sides.

SFRS Height = 36 ft < 40 ft *Okay*

1b. Fire and Life Safety Height and Area Limits

For fire and life safety reasons, the IBC limits building area and height based on construction type and building occupancy classification. This example utilizes tilt-up concrete walls and a hybrid panelized roof diaphragm which is commonly classified as Type V-B (non-rated) construction. Type V construction allows the structural elements, exterior walls and interior walls to be of any construction materials permitted by the IBC (Section 602.5).

Buildings of this type are commonly used for warehouse storage and manufacturing, and have occupancy classifications under IBC Chapter 3 of *S* and *F* respectively. *S-1* and *F-1* occupancies are the most stringent, and using IBC Table 503 it can be determined that a 40-foot height limit and a 1-story limit are applicable. The incorporation of an automatic fire sprinkler system as is done in this example increases these limits to a 60-foot height and 2 stories based on IBC Section 504.2.

Max Height IBC Table 503	Sprinkler Increase IBC 504.2	Maximum Building Height
40 ft	+ 20 ft	60 ft
1 story	+ 1 story	2 stories

The actual building height is measured from the grade plane to the average height of the roof surface, where the grade plane is defined as the average of the finished ground level adjoining the building at the exterior walls (IBC Section 502). Because distribution warehouse buildings are often accompanied with a 4-foot dock below the finish floor level, an additional 4 feet of height is created here. In this example, the average roof height between the 36-foot ridge and the 32-foot eave is 34 feet.

Building Height = 34 ft + 4 ft = 38 ft < 60 ft *Okay*

Stories = 1 < 2 maximum *Okay*

IBC Table 503 also limits the building area per story; however, using IBC Section 507.3, occupancies *S* and *F* may have unlimited area if the building is equipped with an automatic fire sprinkler system and surrounded by 60-foot sideyards and/or public ways.

Building Area = 300 ft x 504 ft = 151,200 ft^2 < Unlimited *Okay*

Part 2 – Seismic Force Coefficients   Code Reference

2a. Design Spectral Response Accelerations S_{DS} and S_{D1}

Seismic design forces are typically evaluated at a short period response S_S and a 1-second response S_1. These mapped spectral accelerations are given for a specific geographic location and then modified based on the geologic site class as well as other factors. In this example, the Site Class is given as D (stiff soil).

S_S = 1.5g (short period) .... ***Given***

S_1 = 0.6g (1-second period) .... ***Given***

F_a = 1.0 (short period) .... ***IBC Table 1613.3.3***

F_v = 1.5 (1-second period) .... ***IBC Table 1613.3.3***

$S_{MS} = F_A S_S = 1.0(1.5) = 1.5$ (short period) .... ***IBC EQ. 16-37***

$S_{M1} = F_V S_1 = 1.5(0.6) = 0.9$ (1-second period) .... ***IBC EQ. 16-38***

$S_{DS} = (2/3)S_{MS} = 1.0$ (short period) .... ***IBC EQ. 16-39***

$S_{D1} = (2/3)S_{M1} = 0.6$ (1-second period) .... ***IBC EQ. 16-40***

For regular buildings 5 stories or less in height and having a period of 0.5 seconds or less, ASCE 7-10 Section 12.8.1.3 permits the base shear's C_s to be calculated using a maximum value of 1.5 for S_S. The building's fundamental period *T* will be evaluated:

$C_T = 0.020$ .... ***ASCE 7-10 Table 12.8-2***

$h_n = 34$ ft .... ***Given (mean roof height)***

$T_a = C_T h_n^{3/4} = 0.28$ seconds .... ***ASCE 7-10 EQ. 12.8-7***

Because this single-story building is without a structural irregularity (ASCE 7-10 Tables 12.3-1 and 12.3-2) and has a period T less than 0.5 seconds, S_{DS} need not exceed the value calculated using 1.5 for S_S for the purpose of computing C_s for base shear in Part 2c. The design spectral response accelerations and SDC remain as originally computed.

S_{DS} = 1.0 (short period)

S_{D1} = 0.6 (1-second period)

2b. Seismic Design Category (SDC)

Buildings are assigned an SDC risk designation based on their design spectral accelerations and occupancy category. With the given Risk Category II and the spectral accelerations determined above, the SDC is evaluated for both the short period and 1-second period to determine the worse category.

SDC using IBC Table 1613.3.5(1): SDC = D (short period)

SDC using IBC Table 1613.3.5(2): SDC = D (1-second period)

Governing SDC = D

2c. Seismic Response Coefficient C_S

The building code uses the building's base shear to determine diaphragm forces, and the appropriate analysis procedure for base shear is obtained using ASCE 7-10 Section 12.6 in conjunction with Table 12.6-1. For the seismic response coefficient C_S this example will use the equivalent lateral force procedure of ASCE 7-10 Section 12.8.

The design base shear is:

$V = C_S W$ .... ***ASCE 7-10 EQ. 12.8-1***

$$C_S = \frac{S_{DS}}{R/I_e}$$ .... ***ASCE 7-10 EQ. 12.8-2***

Because this example's seismic force resisting system utilizes a bearing wall system of intermediate precast concrete shear walls, the response modification factor R = 4 (ASCE 7-10 Table 12.2-1). Additionally, the importance factor I_e = 1.0 is based on Risk Category II (ASCE 7-10 Table 1.5-1 and Table 1.5-2).

$$C_S = \frac{1.0}{4/1.0} = 0.25$$

Check the maximum limit for C_S where $T \leq T_L$. The building's fundamental period, $T = 0.28$ seconds, has already been approximated in Part 2a.

$$C_S = \frac{S_{D1}}{T\left(R/I_e\right)} = \frac{0.6}{0.28\left(4/1.0\right)} = 0.53 > 0.25 \quad \textit{Okay}$$ .... ***ASCE 7-10 EQ. 12.8-3***

Checking the minimum allowed value for C_S:

$C_{S\,min} = 0.044 S_{DS} I_e = 0.044 < 0.25$ *Okay* .... **ASCE 7-10 EQ. 12.8-5**

$C_{S\,min} = 0.01 < 0.25$ *Okay* .... **ASCE 7-10 EQ. 12.8-5**

$C_{S\,min} = \dfrac{0.5 S_1}{R/I_e} = 0.075 < 0.25$ *Okay*.......... **ASCE 7-10 EQ. 12.8-6**

The calculated value for $C_S = 0.25$ is between the maximum and minimum allowed, thus:

$C_S = 0.25$

$V = C_S W = 0.25W$

2d. Roof Diaphragm Force Coefficient

The roof diaphragm serves an important function spanning horizontally like a flat beam from shear wall to shear wall, resisting horizontal seismic forces generated by its self-weight as well as the weight of other heavy objects anchored to the diaphragm, especially the concrete walls. The seismic forces are considered separately in each principal orthogonal direction. While it is true the seismic forces travel from the diaphragm to the shear walls, the design seismic force is not necessarily the same for the diaphragm as the walls.

In general, diaphragm forces F_{px} are based on the vertically distributed base shear to each level with adjustments for level mass, maximums and minimums. Because this example involves only a single story, the computations are simplified.

The following formula from ASCE 7-10 Equation 12.10-1 is used to determine the total seismic force F_{px} onto the diaphragm at a given level x within the building:

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

With this building's base shear of V = 0.25w and the fact that it is single story, Equation 12.10-1 becomes:

$F_{px} = 0.25 w_{px}$

F_{px} need not exceed

$0.4 S_{DS} I_e w_{px} = 0.4(1.0)(1.0) w_{px} = 0.4 w_{px}$ .... **ASCE 7-10 Sec. 12.10.1.1**

But shall not be less than

$0.2 S_{DS} I_e w_{px} = 0.2(1.0)(1.0) w_{px} = 0.2 w_{px}$ .... **ASCE 7-10 Sec. 12.10.1.1**

Based on this criteria, the diaphragm seismic force is $F_p = 0.25 w_p$

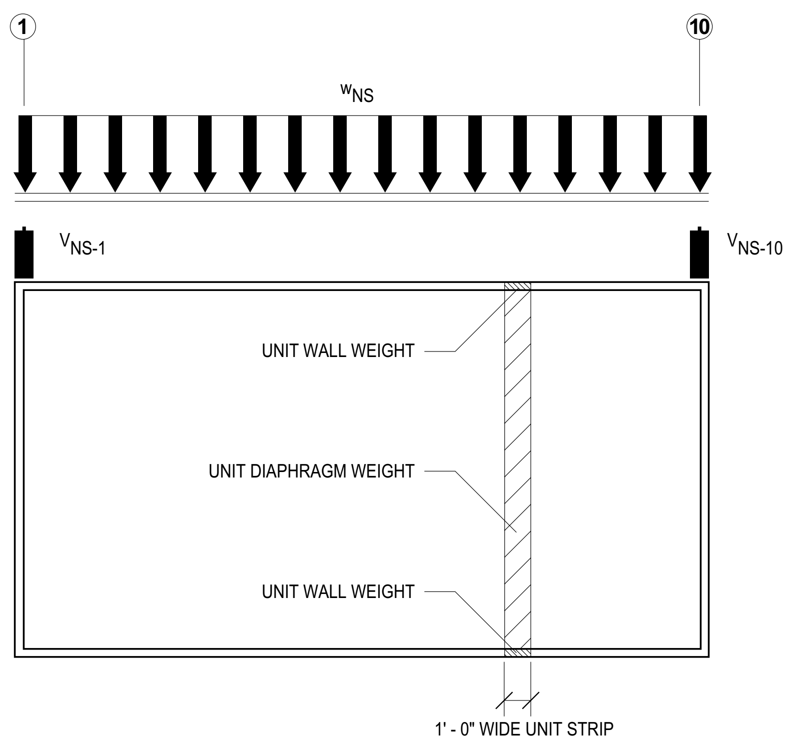
With the seismic force coefficient for the diaphragm established in each direction, the resulting diaphragm shears may be determined. Before that can be done, the actual building weight at the diaphragm level w_p needs to be determined. Because a wood structural panel roof system is permitted to be idealized as a flexible diaphragm (ASCE 7-10 Section 12.3.1.1), the diaphragm may be modeled as a simple flat beam with a horizontally applied distributed load.

The applied distributed load to the diaphragm may be computed by simply evaluating the weight of a series of 1-foot-wide strips across the building multiplied by the diaphragm force coefficient. These unit width strips include the roof system weight and the perpendicular wall weights and, as is the case in this building, will produce a uniform load across the diaphragm's span. Irregularly shaped diaphragms or buildings with varying wall weights will have non-uniform distributed diaphragm loadings. The applied load to the diaphragm will be evaluated separately in each orthogonal direction.

3a. Roof Diaphragm Shears

Using our flat-beam analogy, north/south seismic forces are resisted by shear walls on grid lines 1 and 10, and east/west seismic forces are resisted by shear walls on grid lines A and G. A uniform load across this flat-beam is based on the following model.

Figure 4. *Diaphragm Loading Model*



The uniform loads w_{NS} and w_{EW} are computed using the diaphragm lengths and unit weights and the wall heights and unit weights:

Roof dead load = 15 psf

Wall dead load = 116 psf

Roof height$_{NS}$ = 32 ft along grid lines A and G

Roof height$_{EW}$ = 34 ft average along grid lines 1 and 10

Top of wall = 37 ft (above floor)

w_{NS} = (roof loading contribution) + (wall loading contribution)

$$w_{NS} = 0.25(15 \text{ psf})(300 \text{ ft}) + 2\left[0.25(116 \text{ psf})37 \text{ ft}\left(\frac{37 \text{ ft}}{2}\right)\left(\frac{1}{32 \text{ ft}}\right)\right] = 2{,}366 \text{ plf}$$

$$w_{EW} = 0.25(15 \text{ psf})(504 \text{ ft}) + 2\left[0.25(116 \text{ psf})37 \text{ ft}\left(\frac{37 \text{ ft}}{2}\right)\left(\frac{1}{34 \text{ ft}}\right)\right] = 3{,}058 \text{ plf}$$

In this example, the wall weight has been simplified by ignoring any effect that wall openings have in reducing the force to the roof diaphragm. Because wall openings in these buildings are in the lower half of the wall, this simplification has little impact.

The maximum design shears are now computed using simple statics on the uniformly loaded flat beam model. Because of the building's regular shape and uniformly distributed mass, the loading diagram will be uniform and V_{NS} at grid lines 1 and 10 will be equal. However, buildings with changes in wall thicknesses, wall heights, roof weights, building widths, et cetera will have non-uniform loading to the diaphragm.

North/south diaphragm shear:

$$V_{NS} = w_{NS}\,\frac{504 \text{ ft}}{2} = 2{,}366 \text{ plf}\,\frac{504 \text{ ft}}{2} = 596{,}000 \text{ lbs maximum}$$

North/south unit shear:

$$v_{NS} = \frac{V_{NS}}{300 \text{ ft}} = \frac{596{,}000 \text{ lbs}}{300 \text{ ft}} = 1{,}987 \text{ plf maximum}$$

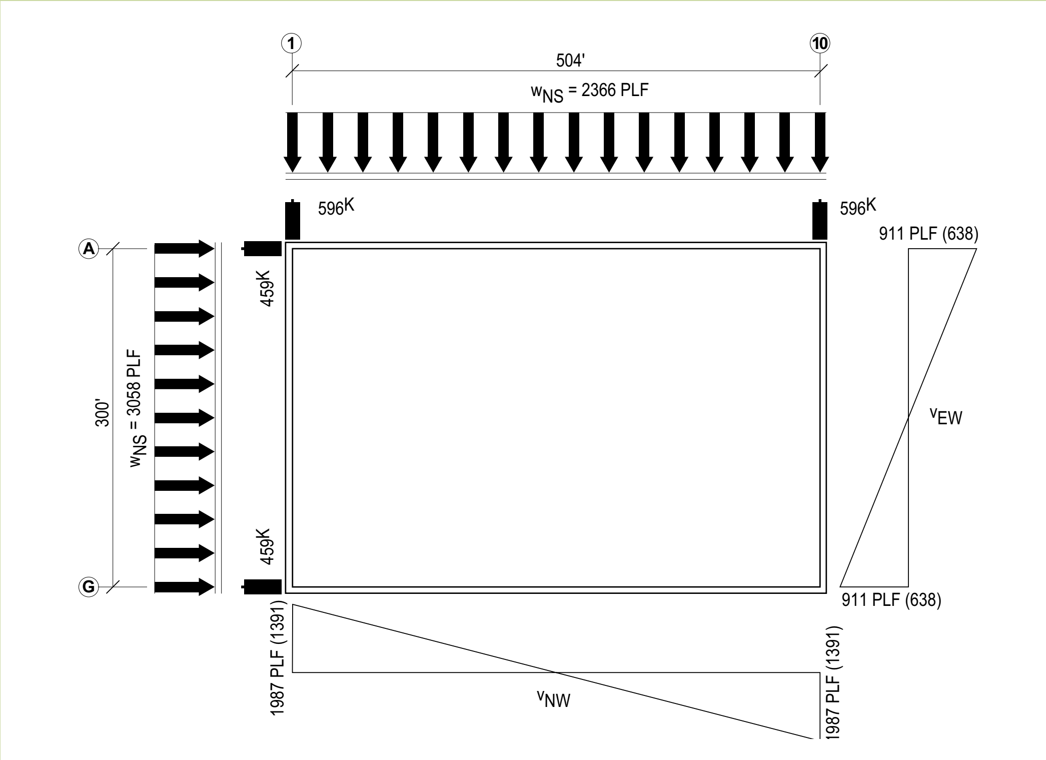
East/west diaphragm shear:

$$V_{EW} = w_{EW}\,\frac{300 \text{ ft}}{2} = 3{,}058 \text{ plf}\,\frac{300 \text{ ft}}{2} = 459{,}000 \text{ lbs maximum}$$

East/west unit shear:

$$v_{EW} = \frac{V_{EW}}{504 \text{ ft}} = \frac{459{,}000 \text{ lbs}}{504 \text{ ft}} = 911 \text{ plf maximum}$$

Figure 5. *Diagram Shear Distribution (Add Values in Parenthesis)*



3b. North/South Diaphragm Shear Design

In Part 3a, the maximum diaphragm shears were determined for each orthogonal direction. The worst unit shear demand is in the north/south direction with v_{NS} = 1,987 plf (unfactored). The allowable diaphragm shear capacities are provided in the *National Design Specification for Wood Construction Seismic Design Provisions for Wind and Seismic* (SDPWS-2008) Tables 4.2A and 4.2B for a wide range of blocked diaphragm conditions. Panelized roof systems are inherently blocked based on their modular arrangement. In general, the timber design industry still uses the Allowable Stress Design (ASD) format; therefore this example will follow ASD convention for the timber diaphragm design.

The basic loading combinations are given in IBC Section 1605.3.1 and those involving earthquake loading have been simplified in ASCE 7-10 Section 12.4.2.3, where load combination (5) will govern the design.

$(1.0 + 0.14S_{DS})D + H + F + 0.7\rho Q_E$ .... ***ASCE 7-10 Sec. 12.4.2.3***

When considering horizontal wind or seismic loads on a structural diaphragm, the vertical loading is not considered when evaluating the lateral diaphragm unit shear stress. Thus, the applicable load combination is simplified using $D = 0$, $H = 0$, $F = 0$ and $L = 0$. Additionally, the redundancy factor (ρ) is set to 1.0 for typical diaphragms per ASCE 7-10 Section 12.3.4.1. Therefore, the applicable basic load combination reduces to $0.7Q_E$.

$V_{NS(ASD)} = 0.7v_{NS} = 0.7(1{,}987) = 1{,}391$ plf

$V_{EW(ASD)} = 0.7v_{EW} = 0.7(911) = 638$ plf

Using these unit shear values, the designer may enter either SDPWS-2008 Table 4.2A or 4.2B to select a diaphragm construction with an appropriate shear capacity. The first aspect to decide upon is the desired sheathing thickness, with options ranging from 3/8 inches to 23/32 inches. A feature unique to panelized roof systems is that the wood structural panel (plywood or OSB) is typically oriented with the long direction parallel to the subpurlin supports (see Figure 6). This is actually spanning the panel's weak direction for gravity loads, but is traditionally done to accommodate the efficiencies of the 8-foot panelized module and to remain fully blocked. In order to support minimum roof dead and live loads spanning the weak direction, 15/32-inch Structural I sheathing is traditionally used as a minimum. Thicker structural use panels are often specified in the Pacific Northwest where heavier snow loads are common and better protection is required against swelling-induced buckling during extended construction rain delays. As an alternative to thicker sheathing, designers in the Pacific Northwest occasionally utilize 8x8 jumbo panels with the strength axis rotated 90 degrees to provide added out-of-plane stiffness for snow loads and protection against moisture buckling. The panelized sheathing system is inherently fully blocked and typically follows layout Case 2 (east/west loading) and 4 (north/south loading) illustrated in SDPWS-2008 Tables 4.2A and 4.2B.

A wood diaphragm resisting seismic forces must have its aspect ratio checked against the limitations in the SDPWS-2008 Table 4.2.4. For blocked diaphragms such as those that occur in panelized construction, the maximum aspect ratio is $L/W = 4$. In this example, the north/south loading direction is the critical L/W ratio.

$L/W = 504/300 = 1.68 < 4$ *Okay*

In the north/south direction, the unit shear demand is greater than the maximum allowable capacities listed in SDPWS-2008 Table 4.2A, thus reference to the high-load diaphragm shears in Table 4.2B will be necessary. The following options are worth considering; all three have similar capacities but have different tradeoffs in terms of material costs and labor costs. The on-center (o.c.) nail spacing shown here requires thicker framing to prevent splitting. The capacities have been reduced from the table nominal strength values to allowable stress design values by dividing in half per SDPWS-2008 Section 4.2.3.

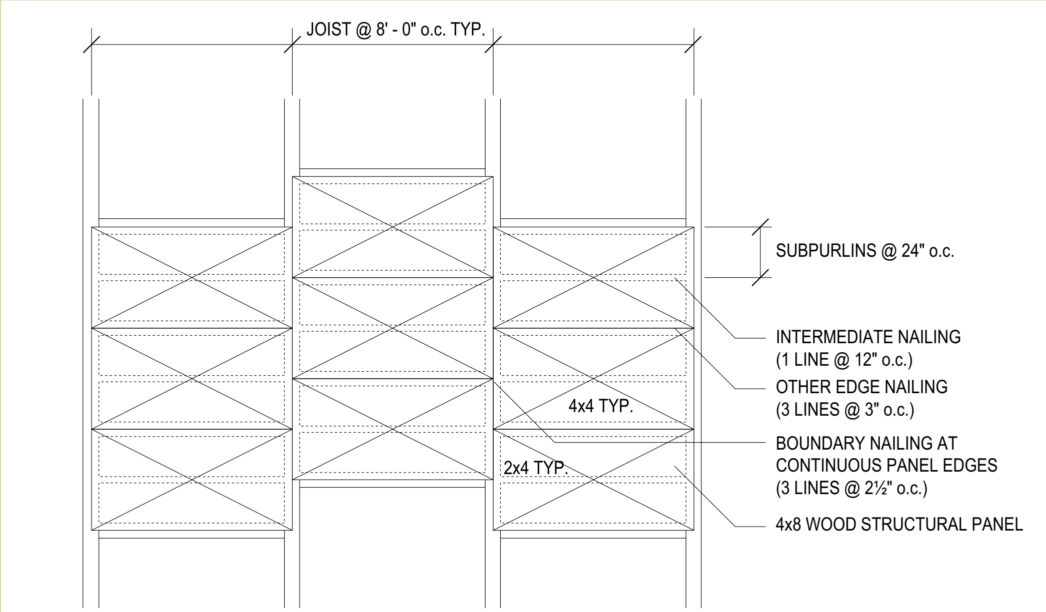
<u>15/32</u>-inch-thick Structural I sheathing 10d nails in <u>*3 lines*</u> at 2½ inches o.c. boundaries and continuous north/south edges 3 inches o.c. other edges 12 inches o.c. intermediate (field) <u>4x framing</u> width at adjoining edges   CAPACITY = 2,790/2 = 1,395 plf (ASD)
<u>19/32</u>-inch-thick Structural I sheathing 10d nails in <u>*2 lines*</u> at 2½ inches o.c. boundaries and continuous north/south edges 3 inches o.c. other edges 12 inches o.c. intermediate (field) <u>4x framing</u> width at adjoining edges   CAPACITY = 2,880/2 = 1,440 plf (ASD)
<u>23/32</u>-in-thick Structural I sheathing 10d nails in <u>*2 lines*</u> at 2½ inches o.c. boundaries and continuous north/south edges 3 inches o.c. other edges 12 inches o.c. intermediate (field) <u>4x framing</u> width at adjoining edges   CAPACITY = 3,130/2 = 1,565 plf (ASD)

While valid arguments can be made for each of the options above, this example will select the 15/32-inch sheathing assembly option described.

$V_{NS(ASD)} = 0.7v_{NS} = 0.7(1{,}987) = 1{,}391$ plf < 1,395 plf *Okay*

The construction of this diaphragm system is illustrated below:

Figure 6. *Sheathing Layout and Nailing*



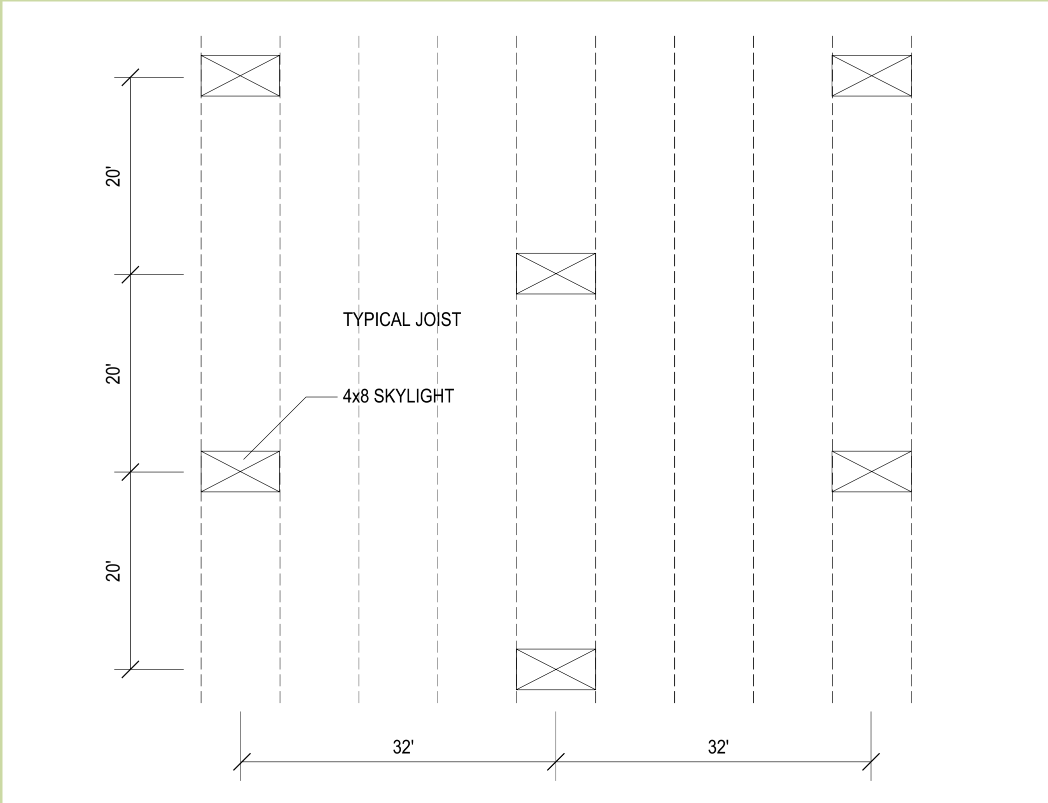
For the nailing configuration selected, a minimum 4x member width is required at adjoining panel edges. Because the steel joists typically have a 5-inch or wider wood nailer, this member requirement is already met along the continuous panel edges in the north/south direction. In the other direction, every other subpurlin will need to be upgraded to a 4x member. This width requirement is necessary to minimize the risk of splitting due to the heavy concentration of diaphragm nailing and to provide an adequate target for the multiple lines of nailing.

3c. North/South Skylight and Smoke Vent Considerations

Warehouse and manufacturing buildings unfortunately often have repetitive skylights and smoke vents across the roof surface, penetrating the diaphragm. In panelized roof construction, these are designed to fit the 4x8-foot module. In this example, it would not be wise to select the 1,395 plf capacity configuration to meet the 1,391 plf demand if the roof penetrations were not considered. Because these penetrations reduce the effective diaphragm cross-sectional width, the unit shears will correspondingly increase at these locations.

In this example, the architect has proposed the repetitive skylight layout as shown in Figure 7, and has asked for the engineer’s input on any restrictions. Appropriately, the architect has spaced the skylights in a manner that is compatible with the panelized roof module (8-foot steel joist spacing and 2-foot subpurlin spacing) to conveniently frame out the openings, and has staggered the layout to minimize disruption to the diaphragm strength in the east/west direction.

Figure 7. *Skylight Penetration Layout*



From the engineer's point of view, these 4x8-foot penetrations will reduce the structural capacity of the diaphragm. It was assumed originally in this example that shears in the north/south direction were being resisted by a 300-foot-wide uninterrupted diaphragm; however, it is now apparent that, when considering the aligned skylights, a 4-foot penetration may occur every 40 feet. In other words, 4 feet of diaphragm is removed within every 40 feet, leaving 90% of the diaphragm intact on average. This will result in an increase in the unit diaphragm shears at these locations. A conservative approach is to simply assume the skylights are adjacent to the diaphragm boundary:

$v_{NS\ AT\ SKYLIGHTS} = v_{NS}$ (1/0.90) = 1,391 (1/0.90) plf = 1,546 plf (ASD) at skylight penetrations

To meet this demand, 23/32-inch-thick Structural I sheathing could be considered as follows:

23/32-inch-thick Structural I sheathing

10d nails in *2 lines* at

2 ½ inches o.c. boundaries and continuous north/south edges

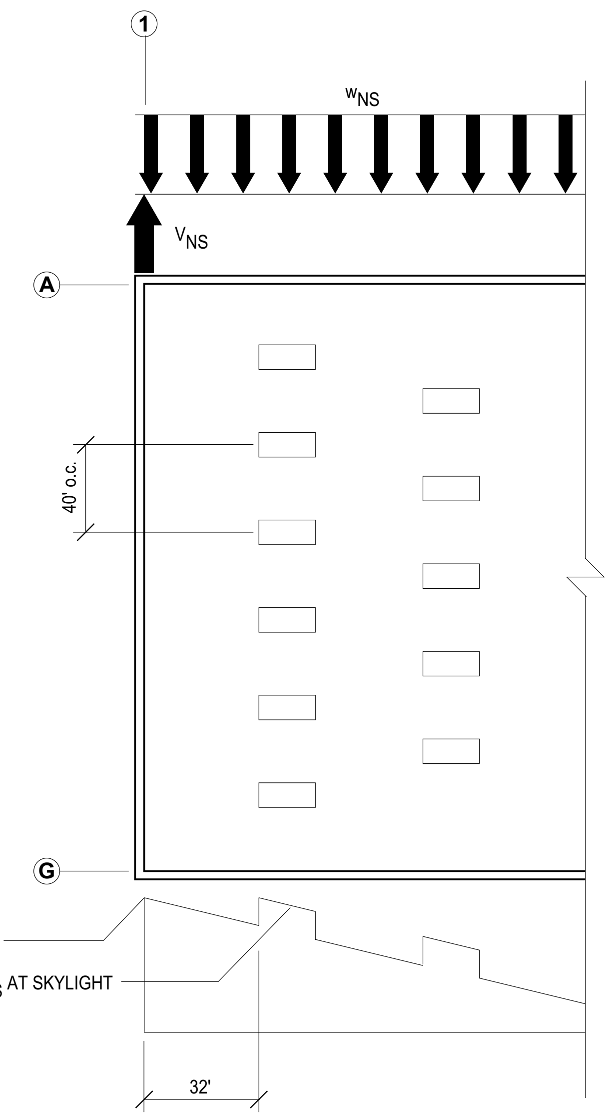
3 inches o.c. other edges

12 inches o.c. intermediate (field)

4x framing width at adjoining edges CAPACITY = 3,130/2 = 1,565 plf (ASD) > 1,546 plf *Okay*

Alternatively, the engineer could keep the originally planned diaphragm nailing system and inform the architect of restrictions on the proximity of the skylights to the diaphragm boundaries. The diaphragm shears diminish toward the center of the building, and may offset the increase in unit shear at the skylights as seen in Figure 8. In this example, let's examine the diaphragm when skylights are not allowed within four joist-bays (32 feet) of the diaphragm boundary.

Figure 8. *Diaphragm Shear Distribution at Skylights*



At the diaphragm boundary, grid lines 1 and 10, the unit diaphragm shear remains as computed:

$v_{NS\ (ASD)} = 1{,}391$ plf

At 32 feet inward, at the first potential series of skylights, the unit diaphragm shear is computed as:

$$v_{NS\ AT\ SKYLIGHTS} = W_{NS}\left(\frac{504\text{ ft}}{2} - 32\text{ ft}\right) = 2{,}366\text{ lbs}\left(\frac{504\text{ ft}}{2} - 32\text{ ft}\right) = 520{,}520\text{ lbs}$$

$$v_{NS\ AT\ SKYLIGHTS(ASD)} = \frac{0.7(V_{NS\ AT\ SKYLIGHTS})}{0.90(300\text{ ft})} = \frac{0.7(520{,}520\text{ lbs})}{270\text{ ft}} = 1{,}349\text{ plf} < 1{,}395\text{ plf}\quad \textit{Okay}$$

By placing restrictions on the skylight placement in proximity to the diaphragm's boundary, the diaphragm's shear design is actually controlled at the wall, not the skylight. Additionally, bending moments between the openings cause a need for subpurlin strapping at the opening corners.

Required strap capacity = 1,349 plf x 36 ft x 8 ft/2 = 5,396 plf (ASD).

3d. North/South Nailing Zones

For a building this size, it would not be an efficient use of labor and resources to install over the entire roof structure the heavy diaphragm nailing and large 4x subpurlins needed at the adjoining panel edges. The maximum unit shears requiring this only occur at the diaphragm boundaries and the first inward line of skylights, and these shears diminish toward the center of the building. The nailing and subpurlin widths may be reduced as the corresponding unit shears also reduce. For this example, Table 1 identifies various diaphragm nailing configurations from the SDPWS-2008 that will be utilized at different portions of the building. The ASD shear values are simply the nominal strengths listed in SDPWS-2008 Tables 4.2A and 4.2B divided by a factor of two per Section 4.2.3.

TABLE 1 – Diaphragm Nailing Schedule

15/32-in Structural I OSB Sheathing (DOC PS-2) with 10d nails (0.148 in dia x 2 in long minimum)					
Zone	Framing Width at Adjoining Edges	Lines of Nails	Nailing Per Line at Boundary & Continuous Edges	Nailing Per Line at Other Edges	ASD Allowable Shear
A	2x	1	6 in o.c.	6 in o.c.	320 plf
B	3x	1	2 in o.c.	3 in o.c.	820 plf
C	3x	2	2½ in o.c.	3 in o.c.	1,150 plf
D	4x	3	2½ in o.c.	3 in o.c.	1,395 plf

At the diaphragm boundaries (grid lines 1 and 10), as well as the first line of skylights parallel to these boundaries, Nailing Zone D was determined to be minimally acceptable. At some location inward, Nailing Zone C will become acceptable as the diaphragm shears reduce farther from the diaphragm boundary. Because a line of skylights could occur anywhere beyond the 32-foot restricted zone thus increasing the unit shears, it will be assumed that only 90% of the diaphragm width can be relied upon. This will give the architect flexibility to shift skylights as long as the same pattern is followed. The demarcation between Nailing Zones D and C may be located as follows using Figure 5:

Shear Demand (ASD) = Shear Capacity (ASD)

0.7[596,000 lbs – (2,366 plf)x] = 1,150 plf (300 ft x 0.90) where

x = the demarcation distance from the diaphragm boundary

Solving for x obtains

x = 64.4 ft

Because a panelized roof system typically consists of 8-foot wide wood structural panels, the joist spacing module is also 8 feet, and the demarcation should be increased to the next 8-foot increment. In this case, it is increased to x = 72 feet.

The demarcations between Zones C and B, and B and A, for the north/south loading direction are found using the same process resulting in the following Table:

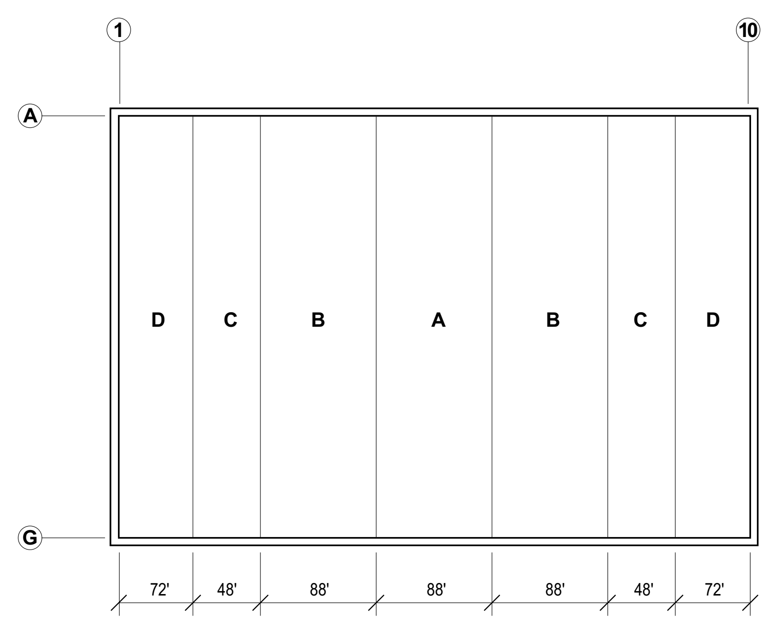
TABLE 2 – Diaphragm Nailing Zones Shear Checks

Nailing Zone	Distance from Boundary	Maximum Unit Shear[1]	ASD Unit Shear	Allowable Shear Capacity
D	0 ft	v_{max} = 1,987 plf	v_{ASD} = 1,391 plf	1,395 plf
C	72 ft	v_{max} = 1,576 plf	v_{ASD} = 1,104 plf	1,150 plf
B	120 ft	v_{max} = 1,156 plf	v_{ASD} = 809 plf	820 plf
A	208 ft	v_{max} = 385 plf	v_{ASD} = 269 plf	320 plf

[1]Skylight penetrations considered; however, no skylights can be located within 32 feet of boundary.

The resulting nailing zone layout for the north/south loading is shown in Figure 9.

Figure 9. *North/South Nailing Zone Layout*

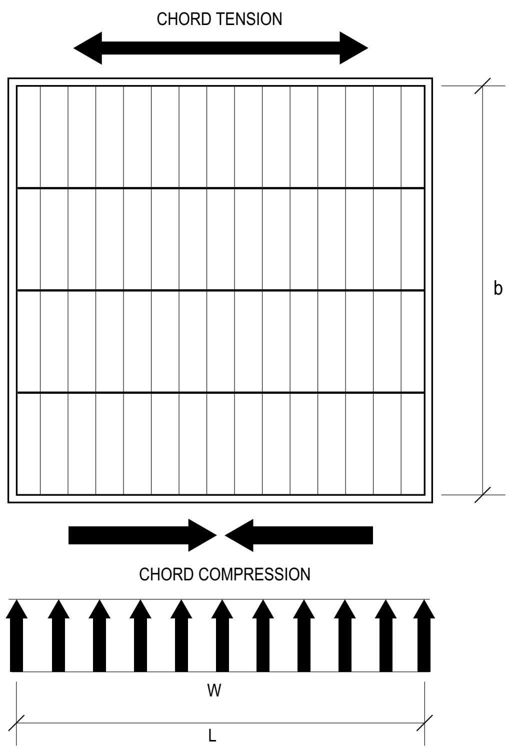


3e. North/South Diaphragm Chord Design

Recall that a flexible diaphragm may be thought of as a flat horizontal beam where the shear resistance is obtained by the wood structural panel sheathing across the roof surface. However, tensile forces from bending are not considered to be resisted by the sheathing, but by bending chords instead. It may be helpful to consider an analogy with a wide flange steel beam, where the flanges resist bending similar to diaphragm chords and the web resists shear similar to diaphragm sheathing.

To resist bending efficiently, the diaphragm chords are traditionally placed at the extreme sides of the diaphragm. In buildings with masonry or concrete perimeter walls, these chords often are designed as embedded steel reinforcing at or near the roof line. Chord forces are determined using simple statics as shown in Figure 10. Maximum chord forces will occur at the center of the diaphragm's span where the maximum moment coincides.

Figure 10. *Traditional Diaphragm Chord Forces*



w = DIAPHRAGM LOAD (KIPS/FOOT)
L = DIAPHRAGM LENGTH (FEET)
b = DIAPHRAGM WIDTH (FEET)
M_{DIAPH} = DIAPHRAGM BENDING MOMENT (FT-KIPS)
T = C = CHORD FORCE COUPLE (KIPS)

$$M_{DIAPH} = \frac{wL^2}{8}$$

$$T = C = \frac{M_{DIAPH}}{b}$$

Using the equations derived in Figure 10, the maximum tensile chord force for our building example is determined:

$$T = \frac{M}{b} = \frac{W_{NS}L^2}{8b} = \frac{2{,}366 \text{ plf } (504 \text{ ft})^2}{8(300 \text{ ft})} = 250{,}417 \text{ lbs}$$

The chord is designed here using strength design (ACI 318-11) with ASTM A706 Grade 60 reinforcing (F_Y = 60 ksi). Consulting IBC Section 1605.2, the applicable load factor for seismic forces is 1.0. The area of steel required is:

$$A_S = \frac{ØT}{F_y} = \frac{0.9(250.4 \text{ kips})}{60 \text{ ft}} = 3.76 \text{ in}^2$$

Using #8 reinforcing bars (A_S = 0.79 in^2 each), the number of bars required is:

$$\text{Number of \#8's} = \frac{3.76 \text{ in}^2}{0.79 \frac{\text{in}^2}{\text{bar}}} = 4.8 \text{ bars}$$

Use (5) #8 reinforcing bars for diaphragm chord embedded in concrete walls.

Because this building example is comprised of concrete tilt-up wall panels, the reinforcing chord bars will be interrupted at each vertical wall panel joint. Often these joints are between 20 feet and 30 feet apart, and a welded splice connection is necessary at each joint. Due to the size of this building, it would be an efficient use of materials to consider the reduction in chord forces toward the ends of the diaphragm and to correspondingly reduce the number of chord bars. The design of the rebar splices is not a part of this example, but needs to be properly considered by the designer.

3f. East/West Diaphragm Shear Design

Having completed the diaphragm design in the north/south loading direction, the east/west design is the logical next step. The east/west design will follow the same sequential process as the north/south design.

In Part 3a, the maximum diaphragm shear in the east/west direction was determined to be v_{EW} = 911 plf (unfactored). Converting to allowable stress design (ASD), the maximum diaphragm shear is as follows:

$v_{EW(ASD)} = 0.7v_{EW} = 0.7(911 \text{ plf}) = 638 \text{ plf}$

Using this unit shear demand, the designer enters SDPWS-2008 Table 4.2A to select a diaphragm construction system with an appropriate shear capacity. 15/32-inch Structural I sheathing was selected for the north/south loading direction, and thus must match here to be consistent. The north/south loading was a Case 4 diaphragm loading configuration, but for the east/west load direction this becomes

Case 2. The applicable Case determines the nail spacing along the continuous adjoining panel edges in the direction parallel to load.

In the east/west loading direction, the maximum unit shear demand is within the maximum allowable capacities listed in SDPWS-2008 Table 4.2A, and are thus also within the diaphragm capacities outlined previously in Part 3d's Table 1. Comparing the diaphragm's shear demand with Table 1, Nailing Zone B is selected:

$V_{EW(ASD)}$ = 638 plf < 820 plf (Nailing Zone B)

For the nailing configuration selected, a minimum 3x member width is required at adjoining panel edges. Because the steel joists typically have a 5-inch or wider wood nailer, this member requirement is already met along the continuous panel edges. In the other direction, every other subpurlin will need to be upgraded to a 3x member. This width requirement is necessary to minimize the risk of splitting due to the heavy concentration of diaphragm nailing and to provide an adequate target for the multiple lines of nailing. At interior areas of the diaphragm where nail spacing increases and shear demand decreases, utilizing 2x framing members is most economical as done in Nailing Zone A.

3g. East/West Skylight and Smoke Vent Considerations

Similar to the north/south loading direction, the diaphragm capacity in the east/west loading direction is also affected significantly by the repetitive skylight and smoke vent penetrations across the roof surface. As seen in Figure 7, aligned 8-foot penetrations interrupt the diaphragm every 64 feet. In other words, 8 feet of diaphragm is removed every 64 feet, leaving 87.5% of the diaphragm intact on average. This will result in an increase in the unit diaphragm shears at these locations.

Intact diaphragm capacity = 1 – (8 ft/64 ft) = 0.875 or 87.5%

$v_{EW\ AT\ SKYLIGHTS}$ = v_{NS} (1/0.875) = 638 plf (1/0.875) = 729 plf (ASD) at skylight penetrations

To meet this demand, 15/32-inch-thick Structural I sheathing at Nailing Zone B is still acceptable.

$v_{EW\ AT\ SKYLIGHTS}$ = 729 plf < 820 plf *Okay*

3h. East/West Nailing Zones

At some distance away from the diaphragm boundary (walls at grid lines A and G), Nailing Zone A in Table 1 will be acceptable due to the diminishing unit diaphragm shears. For efficient use of materials and a reduction in labor costs, it is desired to use Nailing Zone A where possible.

Because a line of skylights could occur anywhere, it will be assumed that only 87.5% of the diaphragm width can be relied upon. This will give the architect flexibility to shift skylights as long as the same pattern is followed. The demarcation between Nailing Zones B and A may be located as follows using Figure 5:

Shear Demand (ASD) = Shear Capacity (ASD)

0.7[459,000 lbs – (3,058 plf)x] = 320 plf (504 x 0.875)

Where

x = the demarcation distance from the diaphragm boundary.

Solving for x obtains

x = 84.2 ft

Because a panelized roof system typically consists of 2-foot spaced repetitive subpurlins, the demarcation should at a minimum be increased to the next 2-foot increment. Some engineers and contractors prefer to place demarcation limits at the girder grid lines, in which case the demarcation is increased to the next 50-foot increment. In this case, it is increased to x = 100 feet.

The following table summarizes the demarcation between various nailing zones for the east/west loading direction.

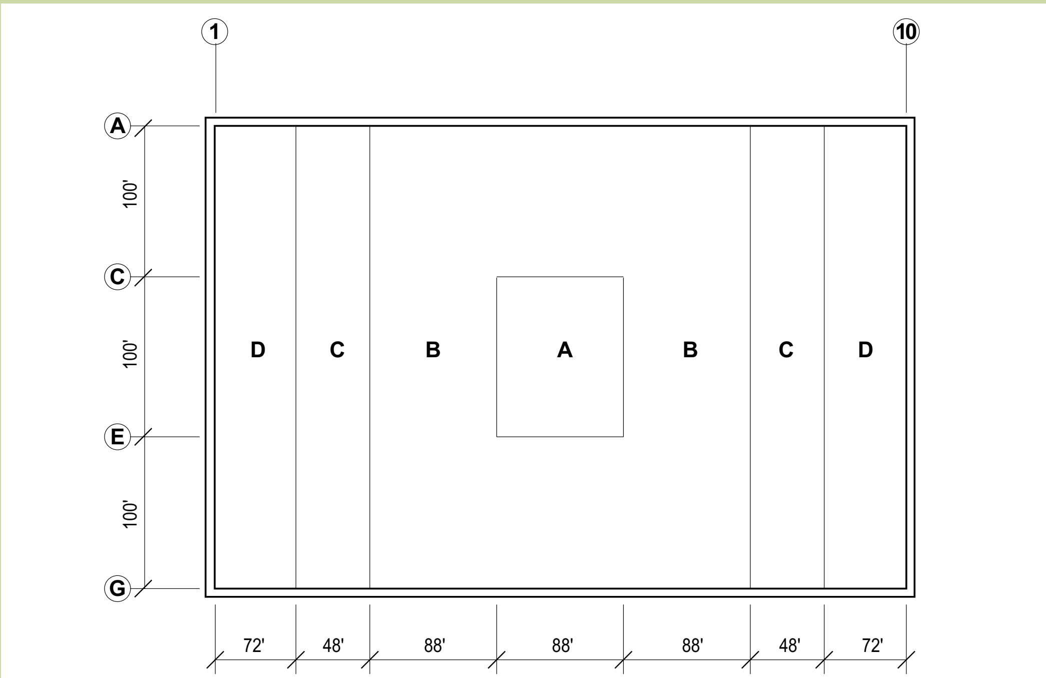
TABLE 3 – Diaphragm Nailing Zones Shear Checks

Nailing Zone	Distance from Boundary	Maximum Unit Shear[1]	ASD Unit Shear	Allowable Shear Capacity
D	N/A	N/A	N/A	1,395 plf
C	N/A	N/A	N/A	1,150 plf
B	0 feet	v_{max} = 1,041 plf	v_{ASD} = 729 plf	820 plf
A	100 feet	v_{max} = 347 plf	v_{ASD} = 243 plf	320 plf

[1]Skylight penetrations considered

Combining the nailing requirements for the north/south loading with the east/west loading results in Figure 11. It should be mentioned that it is possible to have many more intermediate nailing zones further refining the diaphragm design and reducing material costs; however, at some point a large number of different nailing zones may cause excessive complexity to the construction process.

Figure 11. *North/South and East/West Nailing Zones Combined Layout*



3i. East/West Diaphragm Chord Design

Assuming the diaphragm chords are placed at the extreme sides of the diaphragm, the maximum chord forces in the east/west loading direction are computed as follows:

$$T = \frac{M}{d} = \frac{W_{EW}L^2}{8d} = \frac{3{,}058 \text{ plf } (300 \text{ ft})^2}{8(504 \text{ ft})} = 68{,}259 \text{ lbs}$$

Using ASTM A706 Grade 60 reinforcing (F_Y = 60 ksi) in conjunction with ACI 318-11, the area of steel required is:

$$A_S = \frac{ØT}{F_y} = \frac{0.9(68.3 \text{ kips})}{60 \text{ ksi}} = 1.02 \text{ in}^2$$

Using #8 reinforcing bars (A_S = 0.79 in^2 each), the number of bars required is:

$$\text{Number of \#8's} = \frac{1.02 \text{ in}^2}{0.79 \frac{\text{in}^2}{\text{bar}}} = 1.3 \text{ bars}$$

Use (2) #8 reinforcing bars for diaphragm chord embedded in concrete walls.

Some designers are tempted to reduce this reinforcing along the wall toward the building corners as the chord forces reduce, but it is important to also consider the subdiaphragm chord forces that occur in this same reinforcing. See Part 6e later.

3j. Alternative Collective Chord Design

In Parts 3e and 3i, a traditional chord design approach is presented where the chord is thought to solely be located at the extreme diaphragm boundaries. This approach is well accepted and can be somewhat conservative. In reality, the actual distribution of chord forces within the diaphragm may be quite different. For example, in Parts 5 and 6, continuous ties associated with wall anchorage will be provided and these interconnected elements could be utilized as a series of collective chords distributed across the diaphragm. Additional information is available within the paper *Thinking Outside the Box: New Approaches to Very Large Flexible Diaphragms* (available at www.works.bepress.com/jwlawson).

Part 4 – Diaphragm Deflection Check   Code Reference

As any flexible diaphragm is laterally loaded, it will deform and deflect similar to a flat beam. Excessive deflections may impact the building's structural integrity, compromise the attached non-structural elements or cause the building to pound against adjacent structures. Excessive deflections may result in excessive second-order $P\Delta$ loading effects, causing a runaway collapse. ASCE 7-10 Section 12.12.2 limits diaphragm deflections to the amount that will permit the attached elements to maintain structural integrity and to continue supporting their prescribed loads. For structural elements, the intent is to ensure structural stability by avoiding the formation of a collapse mechanism in the gravity support system and to avoid excessive $P\Delta$ loading effects that could lead to a runaway collapse. For non-structural elements, the intent is to prevent failure of connections or comprise self-integrity that could result in a localized falling hazard.

4a. Deflection of North/South Diaphragm

Horizontal diaphragm deflections under lateral loading may be computed using the procedures in SDPWS-2008 Section 4.2.2. Equation 4.2-1 provides a simplified method of computing deflections of wood structural panel diaphragms by considering flexural bending, shear deformation, nail slip and chord slip.

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1{,}000G_a} + \frac{\sum(x\Delta_c)}{2W}$$ .... ***SDPWS-2008 Eq. 4.2-1***

Reviewing our diaphragm, the following parameters are determined below:

v = 1,987 plf (see Figure 5)

L = 504 ft

W = 300 ft

E = 29,000,000 psi

A = 5 #8 bars = 5 x 0.79 = 3.95 in^2 (see Part 3e)

G_a = 24 kips/in Zone A (SDPWS-2008 Table 4.2A)

= 26 kips/in Zone B (SDPWS-2008 Table 4.2A)

= 56 kips/in Zone C (SDPWS-2008 Table 4.2B)

= 70 kips/in Zone D (SDPWS-2008 Table 4.2B)

Δ_c = 0 (Welded chord connections have no slip)

The flexural deformation contribution $(5vL^3)/(8EAW)$ is derived from a horizontal beam with a uniformly applied distributed load. This approach assumes a simplified model where the compression chord involves only the steel reinforcing. In reality, the concrete walls themselves provide tremendous chord deformation resistance, and thus this equation below has some conservative assumptions.

$$\delta_{dia\ flexure} = \frac{5vL^3}{8EAW} = \frac{5(1{,}987 \text{ plf})(504 \text{ ft})^3}{8(29{,}000{,}000 \text{ psi})(3.95 \text{ in}^2)300 \text{ ft}} = 4.63 \text{ in}$$

The shear deformation and nail slip contribution $0.25vL/(1{,}000G_a)$ is derived from a horizontal beam with uniform shear stiffness and uniform diaphragm nailing, as well as a uniformly applied distributed load. Large diaphragms seldom have uniform diaphragm nailing across the building and thus the shear deformation term needs to be modified. For a diaphragm with various strips of nailing zones, there are several approaches to this modification. The one that will be used here is derived from virtual work principles and is as follows:

$$\delta_{dia\ shear} = \sum \frac{0.5v_{i\ ave}L_i}{1{,}000G_{ai}}$$

where

$v_{i\ ave}$ = the average diaphragm shear within each shear stiffness zone

L_i = the length of each shear stiffness zone measured perpendicular to loading

G_{ai} = the apparent shear stiffness of each shear stiffness zone being considered

This approach assumes the nailing zones are simple strips oriented from north to south as illustrated in Figure 9; however, the east/west diaphragm design caused Nailing Zone B to wrap around Nailing Zone A as illustrated in Figure 11. To simplify the analysis, the added nailing from Zone B (north and south of Zone A) will be ignored which is a conservative approach.

The following table provides an organized approach to this analysis working from grid 1 to 10:

Zone	v_{left}	v_{right}	$v_{i\ ave}$	L_i	G_a	$\frac{0.5v_{i\ ave}L_i}{1{,}000G_{ai}}$
D	1,987	1,576	1,782	72	70	0.92 in
C	1,576	1,156	1,366	48	56	0.59 in
B	1,156	385	771	88	26	1.30 in
A	385	0	193	44	24	0.18 in
A	0	385	193	44	24	0.18 in
B	385	1,156	771	88	26	1.30 in
C	1,156	1,576	1,366	48	56	0.59 in
D	1, 576	1,987	1,782	72	70	0.92 in
					∑ =	5.98 in

$$\delta_{dia\ shear} = \sum \frac{0.5 v_{i\ ave} L_i}{1{,}000 G_{ai}} = 5.98 \text{ in}$$

The chord slip contribution to diaphragm deformation $\sum(x\Delta_c)/(2W)$ is found to be zero because of the use of welded chord splices ($\Delta_c = 0$).

$$\delta_{dia\ chord\ slip} = \frac{\sum(x\Delta_c)}{2W} = 0 \text{ in}$$

Considering all the diaphragm deflection contributions we obtain

$\delta_{dia} = \delta_{dia\ flexure} + \delta_{dia\ shear} + \delta_{dia\ chord\ slip} = 4.63 + 5.98 + 0 = 10.6$ in

This diaphragm deflection amount is from unfactored loading; however, for strength-based design the applicable load combinations contain 1.0*E*. Thus 10.6 inches is also the strength-based diaphragm deflection.

But recall that the base shear coefficient C_S in Part 2c involves dividing the maximum expected building response by the seismic system's response modification coefficient *R*. To properly estimate the diaphragm's maximum expected deflection, the earlier effects of dividing by *R* must be reversed in part, and this is accomplished with the use of the deflection amplification factor C_d. ASCE 7-10 Equation 12.12-1 (or 12.8-15) may be used to compute the maximum expected diaphragm deflection δ_x.

$$\delta_M = \frac{C_d \delta_{max}}{I_e}$$

$C_d = 4$ (ASCE 7-10 Table 12.2-1)

$\delta_{max} = 10.6$ in (an elastic response under strength-level forces)

I = 1.0 (importance factor)

$$\delta_M = \frac{C_d \delta_{max}}{I_e} = \frac{4(10.6 \text{ in})}{1} = 42.4 \text{ in}$$

4b. Limits on Diaphragm Deflection – Deformation Compatibility

One objective to limiting diaphragm deflection is to address deformation compatibility within the structure. Excessive diaphragm deflections may negatively impact the building's structural integrity or compromise the attached non-structural elements. It is important for the building's designer to be familiar with the expected building motion and to consider how that motion may be incompatible with the structural and non-structural elements.

ASCE 7-10 Section 12.12.2 and SDPWS-2008 Section 4.2.1 both state the following in terms of maintaining structural integrity during diaphragm deflections:

> "Permissible deflection shall be that deflection that will permit the diaphragm and any attached elements to maintain their structural integrity and continue to support their prescribed loads as determined by the applicable building code or standard."

Neither the IBC nor the ASCE 7-10 contain prescriptive or other specific procedures to evaluate deformation compatibility, and much is left to the designer's own rational judgment.

Panelized roof structures most commonly occur in shear wall buildings comprised of tilt-up concrete or masonry walls. In concrete tilt-up and masonry shear wall buildings, diaphragm deflection results in the columns and perpendicular walls rotating about their bases due to horizontal diaphragm translation at the top. Assuming the columns and walls were modeled with pinned bases during their individual design, this base rotation is permitted to occur even if some unintentional fixity exists.

Unintentional fixity may be the result of standard column base plate anchorage or wall-to-slab anchorage combined with any wall-to-footing anchorage. The governing building codes for concrete and masonry slender wall design (ACI 318 and ACI 530) require these slender walls to be tension-controlled and flexurally controlled. This allows the walls to better accommodate out-of-plane any localized yielding at the base while continuing to carry the vertical loads. The assumption of plastic hinges forming at the base of various elements during δ_M is acceptable provided that these hinges do not result in an unstable structural mechanism or a loss of support condition.

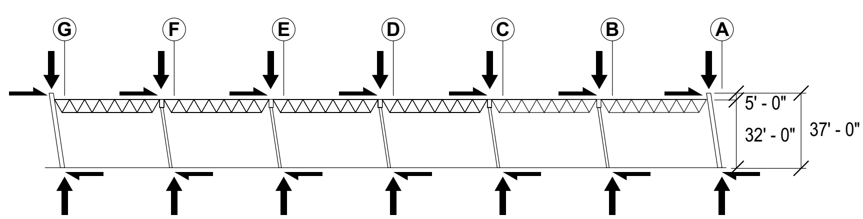
Note: When evaluating the building's horizontal translation at the roof level, the in-plane shear wall drift is typically insignificant compared with the diaphragm deflection and is usually ignored.

One source of possible instability is from the $P\Delta$ effect causing additional horizontal forces on the system. The axially loaded gravity bearing walls and columns, when subjected to horizontal translation at the top, will begin to induce a horizontal thrust into the diaphragm, further exacerbating the deflection (see Figure 12). Although it was not originally intended to be used to evaluate diaphragm deformations, the stability coefficient θ in ASCE 7-10 Section 12.8.7 can be used to investigate the diaphragm system stability under $P\Delta$ effects.

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

This equation evaluates the relative magnitude of horizontal load added to the lateral force resisting system at the system's maximum expected deformation. An increase of 10% or less ($\theta \leq 0.10$) is considered tolerable without a more detailed investigation.

Figure 12: *Deflected Building Section*



The vertical load acting on the translating system P_x has two different components in this building: $P_{x\ roof}$ and $P_{x\ wall}$. $P_{x\ roof}$ is the weight of the translating roof system, and excludes the roof live load based on the applicable load combinations. $P_{x\ wall}$ is the translating concrete wall dead load, and because the wall's center of mass is only translating approximately half of the roof's translation, the $P_{x\ wall}$ may be computed as the upper half of the wall plus any parapet. The load factors are 1.0 for this investigation.

$$P_{x\ roof} = 15 \text{ psf } (504 \text{ ft})(300 \text{ ft}) = 2{,}268 \text{ kips}$$

$$P_{x\ wall} = \frac{9.25 \text{ in}}{12}(150 \text{ pcf})\left(\frac{32 \text{ ft}}{2} + 5 \text{ ft}\right)504 \text{ ft (2 sides)} = 2{,}448 \text{ kips}$$

$$P_x = P_{x\ roof} + P_{x\ wall} = 2{,}268 \text{ kips} + 2{,}448 \text{ kips} = 4{,}716 \text{ kips}$$

Δ = the average horizontal translation. A deflecting flexible diaphragm will approximate a parabolic shape, and thus the average translation of the roof and perpendicular walls will be two-thirds the maximum translation.

$$\Delta = \frac{2}{3}\delta_M = \frac{2}{3}(42.4 \text{ in}) = 28.3 \text{ in}$$

V_x = the seismic shear force acting on the translating system under consideration

$$V_x = 2{,}366 \text{ plf } (504 \text{ ft}) = 1{,}192 \text{ kips}$$

h_{sx} = the height of the translating system under consideration

$$h_{sx} = 32 \text{ ft} \times 12 \text{ in/ft} = 384 \text{ in}$$

C_d = 4 (ASCE 7-10 Table 12.2-1)

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d} = \frac{4{,}716 \text{ kips } (28.3 \text{ in})1.0}{1{,}192 \text{ kips } (384 \text{ in})4} = 0.073 \leq 0.10 \quad \textit{Okay}$$

Because the stability coefficient is less than 10%, the $P\Delta$ effects on story shears, moments, and drifts need not be considered further.

It is worth mentioning here that diaphragm deflection is not included when evaluating the story drift limits of ASCE 7-10 Section 12.12.1. These limitations on building drift were developed primarily for the classic flexible frame system with a rigid diaphragm to prevent excessive distortion within the plane of the frame or shear wall. In masonry and concrete tilt-up buildings, these vertical elements deflect very little in-plane, with the bulk of translation occurring at other elements. The story drift limits of the building code do not apply to the diaphragm deflection.

4c. Limits on Diaphragm Deflection – Building Separations and Setbacks

Buildings are required to have minimum separations from each other and from property lines (setbacks) to prevent seismic pounding during earthquakes (ASCE 7-10 Section 12.12.3). For this evaluation, δ_M is computed for the shear walls' in-plane drift and the diaphragm's in-plane deflection and added together to obtain the overall deflection. However, the stiff concrete or masonry shear walls have insignificant in-plane drift compared with the diaphragm, and this is therefore typically ignored. Also ignored is the slender wall's out-of-plane deflection when considering building separations.

The required setback from property lines is δ_M. When evaluating two buildings on the same property, the required separation between those buildings may be evaluated as the square-root-of-the-sum-of-the-squares (SRSS) of the two independent δ_M's per ASCE 7-10 Equation 12.12-2.

In this example, the building is assumed to be isolated from other buildings and property lines; thus building separations and property line setbacks will not be investigated.

Part 5 – North/South Wall Anchorage   Code Reference

Wall anchorage is a very important consideration when designing concrete and masonry wall buildings with flat flexible diaphragms. Historically, the most critical element of building performance has been this wall-to-roof connection. Consequently building codes have repeatedly revised the wall anchorage force and detailing requirements after various significant earthquakes. Early panelized wood roof structures were popular in California in the 1960s, but the 1971 San Fernando (Sylmar) earthquake revealed a significant problem with most of their designs. Wall anchorage was being justified by the plywood edge nailing to the 4x wood ledgers that were bolted to the inside face of the concrete and masonry walls. In many cases, the seismic loads pulled the heavy walls from the wood roof, failing the wood ledgers in cross-grain bending, allowing the roof to partially collapse.

As a result of the damage observed, the *1973 Uniform Building Code* expressly prohibited cross-grain bending and new wall anchorage detailing requirements were instituted. As the *Uniform Building Code* and its successor the *International Building Code* continued to evolve from edition to edition, the wall anchorage design requirements have increased in response to lessons learned from building performance and building instrumentation data under strong ground motions. Current code provisions were developed around the understanding that roof top accelerations in flexible diaphragms may be three to four times the ground acceleration at the building base, and that wall-to-roof connections traditionally have limited overstrength and ductility. The wall anchorage requirements in ASCE 7-10 Section 12.11.2.1 apply to concrete and masonry wall buildings with additional requirements of Section 12.11.2.2 for buildings in Change to SDC C and higher.

The wall anchorage force equation $F_p = 0.4S_{DS}k_aI_eW_p$ contains an amplification factor k_a to account for the amplification of wall anchorage forces in flexible diaphragms. This amplification factor increases to 2.0 maximum for diaphragm spans equal to or greater than 100 feet as measured between the vertical elements that provide lateral support to the diaphragm in the direction being considered. In large flexible diaphragms, the design wall anchorage force is double the normal wall anchorage force at rigid diaphragms and three to four times the typical building base shear, which is in line with the expected roof top amplification associated with flexible diaphragm behavior.

5a. North/South Wall Anchorage Forces

In this example building, the purlins (steel joists) are spaced 8 feet o.c. as is typical in panelized roof construction, and provide a convenient anchorage point for the north and south walls. The wall anchorage forces to the flexible diaphragm are dictated by ASCE 7-10 Equation 12.11-1.

$F_p = 0.4S_{DS}k_aI_eW_p$ .... ***ASCE 7-10 EQ 12.11-1***

$S_{DS} = 1.0$

$k_a = 1.0 + 501\text{ ft}/100 = 6.0$ Use 2.0 max. .... ***ASCE 7-10 EQ 12.11-2***

$I_e = 1.0$

$W_p = 116$ psf (9¼-in-thick concrete)

$F_p = 0.4S_{DS}k_aI_eW_p = 0.8(116\text{ psf}) = 92.8\text{ psf}$

The computed wall anchorage force needs to be checked against various wall anchorage force minimums to determine if any control the design. Refer to ASCE 7-10 Sections 1.4.5 and 12.11.2.1.

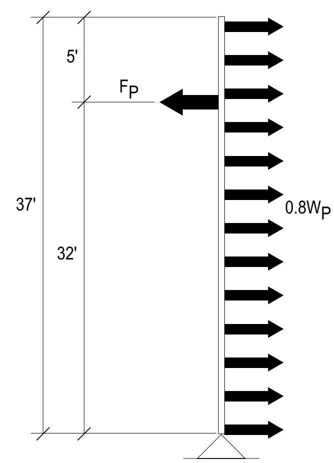
$F_{p\ min} = 5\text{ psf} < 92.8\text{ psf}$ *Okay* .... ***ASCE 7-10 Section 1.4.5***

$F_{p\ min} = 0.2W_p = 23.2\text{ psf} < 92.8\text{ psf}$ *Okay* .... ***ASCE 7-10 Section 1.4.5***

$F_{p\ min} = 0.2k_aI_eW_p = 46.4\text{ psf} < 92.8\text{ psf}$ *Okay* .... ***ASCE 7-10 Section 12.11.2.1***

Evaluating the wall's full height cross-section and performing simple statics to sum moments about the wall's base, the wall's anchorage force is computed at the 8-foot connection points (see Figure 13).

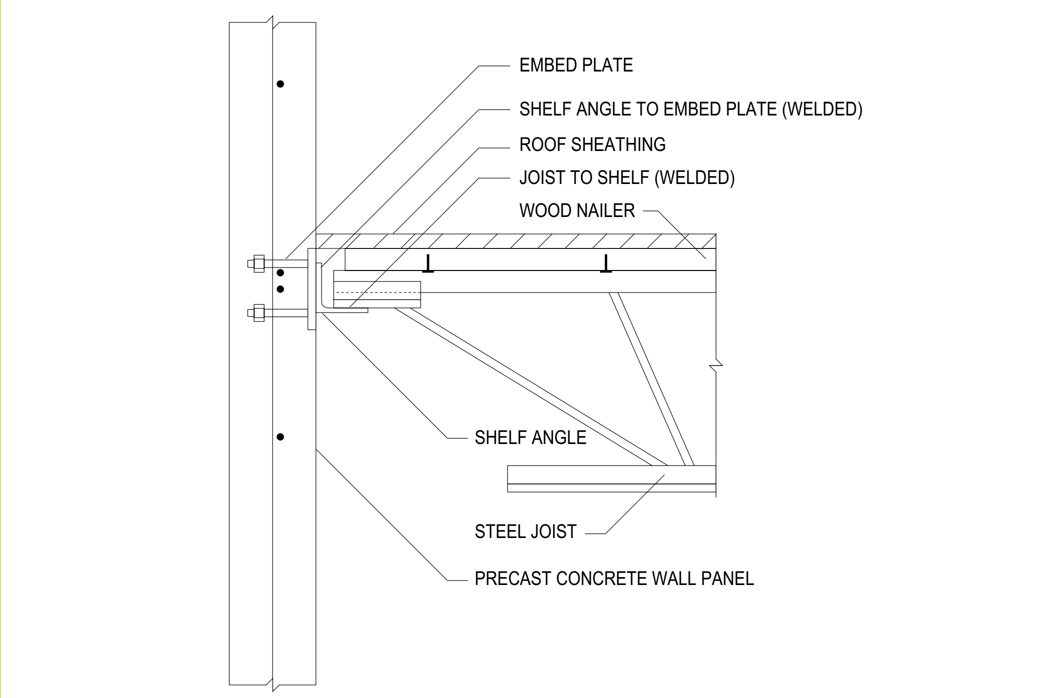
Figure 13: *Loading Diagram for Wall Anchorage Design*



$$F_p = 92.8\text{ psf }(37\text{ ft})\,\frac{37\text{ ft}}{2}\left(\frac{1}{32\text{ ft}}\right)(8\text{ ft}) = 15{,}880\text{ lbs}$$

Therefore, the wall anchorage connection force is $F_p = 15{,}880$ lbs and this is used to design the embedded concrete wall anchors, the steel joist (purlin) and all other connection elements (see Figure 14).

Figure 14: Typical Steel Joist to Wall Tie



Per ASCE 7-10 Section 12.11.2.2.2, steel elements of the structural wall anchorage system (SDC C and above) are required to be designed for these strength forces with an additional 1.4 multiplier. This material-specific multiplier is based on the poor performance of steel straps during the 1994 Northridge earthquake, where a lack of suitable overstrength and ductility was observed. This 1.4 multiplier is applied to all steel elements of the wall anchorage system including wall connectors, continuous ties and their connections. This 1.4 multiplier does not apply to wood bolting and nailing, concrete anchors or concrete reinforcing.

Note: The 8-foot connection point spacing exceeds 4 feet and thus the structural walls need to be designed to bend between connection points per ASCE 7-10 Section 12.11.2. This design detail is beyond the scope of this example.

5b. North/South Continuous Ties

In order for the wall anchorage forces to be adequately distributed into the roof diaphragm, continuous ties are provided as a force distributing strut. The continuous ties take the wall anchorage force and distribute the load uniformly across the diaphragm depth. This building example models the common hybrid panelized roof system where the wood deck diaphragm is supported by open-web steel joists and joist girders. The open-web steel joists are spaced at 8-feet o.c. and function as the gravity support purlins as well as the wall anchorage continuous ties.

The joist manufacturer typically provides engineering services for these joists based on load information from the building's design engineer. IBC Section 2207.2 requires the building's design engineer to provide axial wall tie and continuous tie forces to the manufacturer along with information stating which load factors if any have already been applied. For this building example, the building design engineer should report to the joist manufacturer that the unfactored wall tie axial force acting on the joist top chord (tension and compression) is F_p = 15,880 lbs increased by the steel overstrength factor 1.4 per ASCE Section 12.11.2.2.2 resulting in F_p = 15,880 lbs x 1.4 = 22,232 lbs. Because the joist manufacturer's specialty engineer is selecting load combinations during the design, it is necessary to indicate that this axial load is from seismic effects.

Joist axial load E = 15,880 lbs × 1.4 = 22,232 lbs (unfactored)

In some lower seismic regions, wind forces may control the axial design of the continuous ties. The load combinations of ASCE 7-10 Sections 2.3.1 and 2.4.1 contain very different formulas where wind W and seismic E are involved, and the design engineer cannot simply compare W and E to decide which load effect will govern. In these cases, the design engineer may have to provide both an axial load W and an axial load E to the joist manufacturer.

Because the steel joists terminate at each girder, the joist ends shall be spliced together to provide continuity across the diaphragm from chord to chord. This connection detail typically involves plates or bars and field connecting welds.

Part 6 – East/West Wall Anchorage   Code Reference

6a. East/West Wall Anchorage Forces

In this example building, the subpurlins (2x4s) are spaced 24 inches o.c. as is typical in panelized roof construction, and provide a convenient 2-foot anchorage module for the east and west walls. Typically, these anchor points are spread to 4 feet o.c. (every other subpurlin), and the subpurlins are enlarged to 3x or 4x framing members to accommodate the connection force. As was illustrated previously for the wall anchorage to the purlin, ASCE 7-10 Section 12.11.2.1 is applicable and anchorage forces are amplified due to diaphragm flexibility.

$F_p = 0.4S_{DS}k_aI_eW_p$ .... ***ASCE 7-10 EQ 12.11-1***

$S_{DS} = 1.0$

k_a = 1.0 + 300 ft/100 = 4.0 ... Use 2.0 max. .... ***ASCE 7-10 EQ 12.11-2***

$I_e = 1.0$

W_p = 116 psf (9¼-in-thick concrete)

$F_p = 0.4S_{DS}k_aI_eW_p$ = 0.8(116 psf) = 92.8 psf

The computed wall anchorage force needs to be checked against various wall anchorage force minimums to see if any control the design. Refer to ASCE 7-10 Sections 1.4.5 and 12.11.2.1.

$F_{p\ min}$ = 5 psf < 92.8 psf *Okay* .... ***ASCE 7-10 Section 1.4.5***

$F_{p\ min} = 0.2W_p$ = 23.2 psf < 92.8 psf *Okay* .... ***ASCE 7-10 Section 1.4.5***

$F_{p\ min} = 0.2k_aI_eW_p$ = 46.4 psf < 92.8 psf *Okay* .... ***ASCE 7-10 Section 12.11.2.1***

Evaluating the wall's full height cross-section and performing simple statics to sum moments about the wall's base, the wall's anchorage force is computed at the 4-foot connection points. It is important to note that the elevation of this wall connection varies along the east and west walls due to the roof slope. Because the top-of-wall elevation is constant, the critical design location will be at the lower roof elevation where the parapet will be highest (see Figure 13).

Along the roof line, the distributed wall anchorage force is

$$F_p = 92.8 \text{ psf } (37 \text{ ft}) \frac{37 \text{ ft}}{2} \left(\frac{1}{32 \text{ ft}} \right) = 1{,}985 \text{ lbs/ft}$$

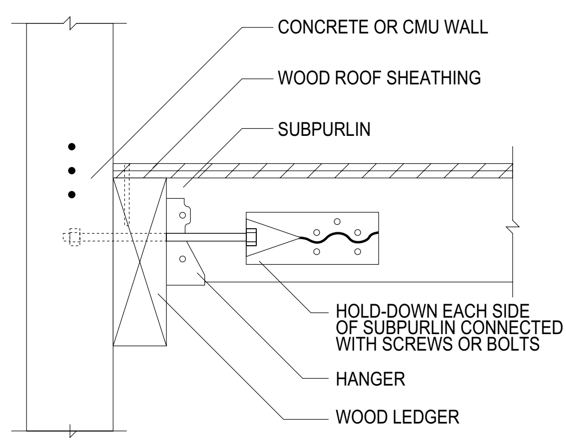
With the specific connection points spaced at 4 feet o.c., the anchorage force is

F_p = 1,985 plf × 4 ft = 7,940 lbs

Therefore, the design wall anchorage connection force is F_p = 7,940 lbs.

The design of the wall anchorage system here is best done by tracing the loads along their path from the heavy concrete wall, through all the various structural components, into the horizontal diaphragm. Figure 15 provides one configuration that is possible to anchor the concrete or masonry wall into the adjacent roof structure. Wood, steel and concrete elements are all a part of this wall anchorage load path.

Figure 15: *Typical Subpurlin to Wall Tie*



Concrete Elements of the Anchorage: The beginning of the wall anchorage load path starts at the anchor embedded in the concrete wall panel. Design for this concrete anchorage is per ACI 318-11 Appendix D and will not be shown here. It is important to note that the special ductility requirements and strength reductions of ACI 318 Sections D3.3.4, D3.3.5 and D.3.3.6 are not applicable based on IBC Section 1905.1.9 when ASCE Equation 12.11-1 is used as was done in this example above.

Another issue to recognize in the concrete design is that the 4-foot connection point spacing does not exceed the 4-foot threshold of ASCE 7-10 Section 12.11.2, and thus the structural concrete walls are not required to be specifically designed for bending between connection points.

Masonry Elements of the Anchorage: When masonry walls occur instead of concrete walls, the anchorage design is per TMS 402-11/ACI 530-11/ASCE 5-11 and will not be shown here. Similar to the concrete wall anchorage, the frequency of the connection point spacing does not exceed the 4-foot threshold of ASCE 7-10 Section 12.11.2, and thus the structural masonry walls are not required to be specifically designed for bending between connection points.

Steel Elements of the Anchorage: As the load transitions from the wall to the wood roof structure, steel connecting elements are often utilized to bridge between concrete and wood, as well as wood to wood. Similar to the north and south wall anchorage, ASCE 7-10 Section 12.11.2.2.2 requires the steel elements of the structural wall anchorage system (with SDC C and above) to be designed for the strength forces with an additional 1.4 multiplier, but this 1.4 multiplier does not apply to wood bolting and nailing, concrete anchors or concrete reinforcing.

Wood Elements of the Anchorage: While following the load path from the concrete (or masonry) wall, the anchorage force transfers to a wood roof framing member at the subpurlin location. As stated earlier, these subpurlins at the anchorage point are enlarged to accommodate large connection forces. In this example, the typical 2x4 subpurlin is replaced with a 4x6 subpurlin at the anchorage point, and will be checked for adequacy in Part 6b.

In this example, the wall anchorage load is transferred to the 4x6 through a prefabricated steel tie-down or hold-down with a series of wood screws or bolts into the 4x6.

The design of the prefabricated tie-down hardware is not a part of this example; however, allowable load information is typically obtained by consulting the hardware manufacturer's ICC-ES Evaluation Report where various allowable load capacities are listed for different configurations. This allowable load information typically includes the design check for the wood bolting for different wood member thicknesses, but does not consider the allowable tension capacity of the wood member's net area at the screws or bolt locations.

As indicated in Figure 15, the prefabricated tie-down hardware is mounted each side of the subpurlin, thus providing double shear wood bolt capacities which are generally double or more than the single shear capacity, thus the designer may safely double the ICC-ES Evaluation Report capacity for the double shear configuration, unless specifically noted otherwise in the report.

6b. Design of Subpurlin at Anchorage

In this example, a 4x6 subpurlin strut using DF/L No. 2 will replace the standard 2x4 subpurlin at the anchorage locations. As stated previously, the hardware connection brackets are each side of the subpurlin resulting in double shear. An important aspect of the double shear configuration is that this removes the problem that eccentricities may create on the wood subpurlin member under a single shear (one-sided) configuration. ASCE 7-10 Section 12.11.2.2.6 specifically states that eccentricities must be addressed in these single-sided configurations, which results in a weak-axis induced bending moment in the subpurlin.

The 4x6 subpurlin experiences a combination of wall anchorage tension from seismic and bending from the roof dead load gravity. These two loads are combined using the load combinations of IBC Section 1605.3.1 for allowable stress design. The maximum effect on the subpurlin is achieved with Equation 16-12.

$D + H + F + 0.7E$ .... ***IBC Equation 16-12***

This load combination may be simplified by substituting the definition of E into the equation per ASCE 7-10.

$E = E_h + E_v$

where:

$E_h = \rho Q_E = Q_E$ (ρ=1.0 for wall anchorage)

$E_v = 0.2S_{DS}D = 0.2(1.0)D = 0.2D$ (S_{DS} = 1.0 from Part 2a)

Thus in this case:

$E = Q_E + 0.2D$

And for ASD

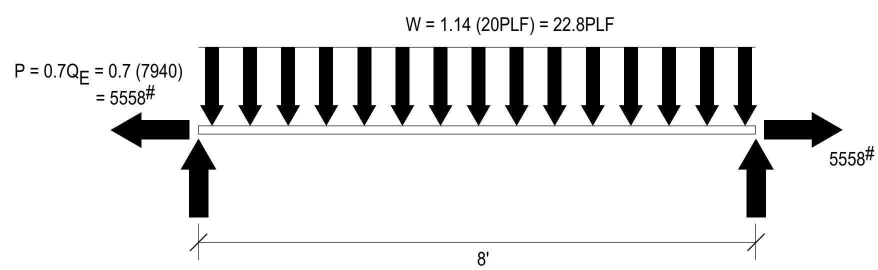
$0.7E = 0.7Q_E + 0.14D$

Substituting and recognizing that H=0 and F=0, the applicable load combination for the roof subpurlin is:

$(1.14)D + 0.7Q_E$

The applied dead load can be estimated from the given unit weights of the roof system at the beginning of this example. The total roof weight was estimated at 15 psf, but removing the self-weight of the joists, girders, and sprinklers which are not typically supported from subpurlins, a 10 psf dead load is estimated on the subpurlins. The 2-foot subpurlin spacing thus results in an unfactored 20 plf applied dead load. The following loading diagram (ASD) illustrates the subpurlin.

***Figure 16:** Subpurlin Loading Diagram*



The actual tensile stress f_t and allowable tensile stress F'_t are determined and compared:

$$f_t = \frac{P}{A} = \frac{5{,}558 \text{ lbs}}{19.25 \text{ in}^2} = 289 \text{ psi}$$

$F'_t = F_t C_D C_M C_t C_F C_i$ .... ***NDS Table 4.3.1***

where:

F_t = 575 psi (DF/L No. 2) .... ***NDS Supplement Table 4A***

C_D = 1.6 .... ***NDS Table 2.3.2***

C_F = 1.3 .... ***NDS Supplement Table 4A***

C_M & C_t & C_i = 1.0

F'_t = 575 psi (1.6)(1.3) = 1,196 psi

$$\frac{f_t}{F'_t} = \frac{289 \text{ psi}}{1{,}196 \text{ psi}} = 0.24 \leq 1.0$$

Thus, tensile stress okay.

The actual bending stress f_b and allowable bending stress F'_b are determined and compared:

$$M = \frac{wL^2}{8} = \frac{22.8 \text{ plf } (8 \text{ ft})^2 (12 \text{ in/ft})}{8} = 2{,}189 \text{ lbs-in}$$

$$f_b = \frac{M}{S} = \frac{2{,}189 \text{ lbs}}{17.65 \text{ in}^3} = 124 \text{ psi}$$

$$F'_b = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r$$

where:

F_b = 900 psi (DF/L No. 2) .... ***NDS Supplement Table 4A***

C_D = 1.6 .... ***NDS Table 2.3.2***

C_F = 1.3 .... ***NDS Supplement Table 4A***

C_r = 1.15 .... ***NDS Section 4.3.9***

C_M & C_t & C_i & C_L & C_{fu} = 1.0

F'_b = 900 psi (1.6)(1.3)(1.15) = 2,153 psi

$$\frac{f_b}{F'_b} = \frac{124 \text{ psi}}{2{,}153 \text{ psi}} = 0.06 \leq 1.0$$

Thus, bending stress is okay, but the combined effect of the bending and tensile stresses needs to be investigated:

$$\frac{f_t}{F'_t} + \frac{f_b}{F^*_b} = 0.24 + 0.06 = 0.30 \leq 1.0$$ .... ***NDS Section 3.9.1***

Thus the combined stress check is acceptable, and the 4x6 subpurlin strut is adequate for the gross section check. Additionally, the net section at the fastener(s) needs to be checked for adequacy. The wall anchorage hardware often connects to the subpurlins with either screws or bolts. In this example, ¾-inch diameter bolts will be assumed to occur in the connector. Considering the bolt hole, the net section needs to be checked for tension alone (bending moment is near zero here). The hole diameter considering the acceptable 1/16-inch oversize is 13/16-inch diameter.

Net Area = (3.5 in)(5.5 in – 13/16 in)=16.41 in^2

$$f_t = \frac{P}{A} = \frac{5{,}558 \text{ lbs}}{16.41 \text{ in}^2} = 339 \text{ psi} \leq 1{,}196 \text{ psi}$$

Thus the net area tensile stress is acceptable and the proposed 4x6 DF/L No. 2 subpurlin at the anchorage is acceptable. It is noted that the 4x6 appears to be significantly oversized based on the stress demand, but sometimes it is desirable to have the extra member depth to facilitate construction tolerances in the field for attachment of the hold-down anchor.

Use 4x6 Subpurlin Strut DF/L No. 2

While the above procedure has checked whether there is adequate strength in the wall anchorage assembly, the designer should be careful not to design an assembly that will have excessive deformation under load. Currently, neither the IBC nor ASCE 7-10 have prescriptive deformation limits of the wall anchorage system, but deformation compatibility is a general requirement to consider. Wall anchorage systems with too much elongation under load will inadvertently load the wood structural panels' edge nailing, causing the nails to either pull through the panel edges or place the wood ledgers in cross-grain tension or bending. Manufacturers of prefabricated tie-down devices have specific limits in their ICC Acceptance Criteria as to how much deformation their devices may contribute to the assembly's overall stretch; however, that is only one component to consider. The type of connectors, length of anchor rods, and installation practices (oversized holes) can be significant sources of additional deformation or stretch.

6c. The Subdiaphragm Size

It is the intent of the building code to gather the wall anchorage force from the wall and then distribute it across the diaphragm from chord to chord using the continuous ties. For SDC C and higher, ASCE 7-10 Section 12.11.2.2.3 specifically states that the diaphragm sheathing cannot be used as the continuous ties. At first appearance, one solution would be to install ties across the diaphragm to line up with every wall anchorage location, but the 4-foot anchorage frequency would result in an excessive number of continuous ties and an excessive number of splice connections, occurring across every perpendicular joist. This is not a practical solution.

Alternatively, chords may be added in the diaphragm's interior region to form smaller subdiaphragms which are a unique analytical tool used to redirect the wall anchorage load conveniently to the girder lines where significant continuity may already exist. The girders may function as the main diaphragm's continuous ties to distribute the heavy wall anchorage forces uniformly into the diaphragm depth, while the smaller subdiaphragms transfer the individual anchor loads to these main diaphragm continuous ties. This concept was first introduced into the *1976 Uniform Building Code* and is currently found in ASCE 7-10 Section 12.11.2.2.1. Consequently, because subdiaphragms, continuous ties and their connections are a part of the wall anchorage system, the elevated wall anchorage forces found in Part 6a are used for their design.

To prevent subdiaphragms from becoming too flexible compared to the rest of the main diaphragm, ASCE 7-10 Section 12.11.2.2.1 places a limit on the length-to-width ratio of the subdiaphragm at 2.5 to 1. This aspect ratio is assumed to provide sufficient stiffness that the independent deflections between the subdiaphragm and the main diaphragm may be ignored.

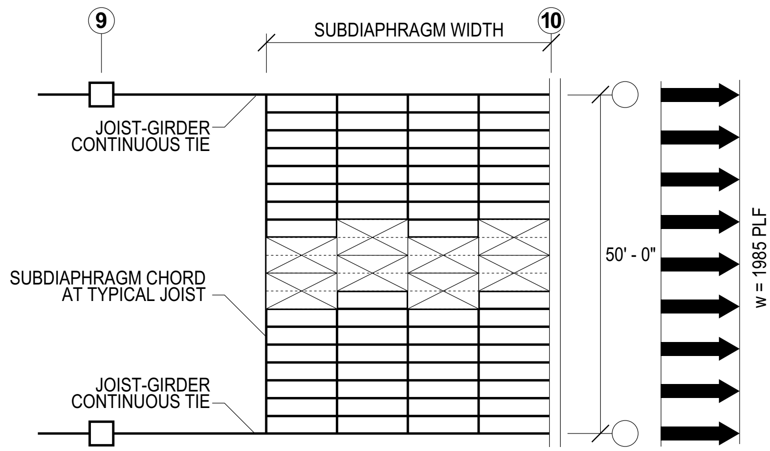
In this example, it is desirable to use the east/west girder lines as the continuous ties for distributing the wall anchorage forces into the main diaphragm. Given the 50-foot spacing between the continuous ties (the subdiaphragm length), the minimum subdiaphragm width is 50/2.5 = 20 feet; however, it is necessary to utilize an existing purlin line as the subdiaphragm's chord. Because these purlins are on an 8-foot module, the 20-foot diaphragm width is increased to 24 feet.

Proposed subdiaphragm length-to-width ratio = 50 ft/24 ft = 2.08 < 2.5 *Okay*

However, the width is more often controlled by the available subdiaphragm's shear capacity. The necessary subdiaphragm width for the shear demand will now be investigated.

The subdiaphragm gathers the wall anchorage load between the main girder lines as illustrated in Figure 17. Similar to a typical flexible diaphragm analysis, the subdiaphragm behaves as a simply supported flat beam with a uniformly distributed load.

__Figure 17:__ Typical Subdiaphragm



The typical subdiaphragm's shear reaction to the continuous ties along the girder lines:

$$\text{Subdiaphragm reaction} = 1{,}985 \text{ plf } \frac{50 \text{ ft}}{2} = 49{,}625 \text{ lbs}$$

Consulting Figure 11, the main diaphragm is utilizing Zone D nailing for the first 72 feet from the east and west walls, and Zone D's diaphragm shear capacity is 1,395 plf (ASD). The minimum subdiaphragm width based on shear demand adjusted to allowable stress design (2012 IBC Section 1605.3.2) is as follows:

$$\text{Minimum subdiaphragm width} = \frac{0.7(49{,}625 \text{ lbs})}{1{,}395 \text{ plf}} = 24.9 \text{ ft} \quad \text{Use 32 ft}$$

The subdiaphragm width is being governed by shear stress instead of the aspect ratio limit, and thus needs to be increased to the next purlin joist on the 8-foot module, or a 32-foot subdiaphragm width.

Note that the subdiaphragm shears being investigated are not combined with the main diaphragm shears near the boundaries for the same load direction. This is because the main diaphragm shears are determined from the overall building's base shear equation, while a different philosophy is used for the wall anchorage forces loading the subdiaphragm. The wall anchorage forces involve an amplified force coefficient based maximum expected roof accelerations for designing and detailing a component of the structure. The main diaphragm involves the overall building and is designed for a reduced load. It is inappropriate to directly combine the shears of these two concepts.

6d. Subpurlin Tie Connections

As a practical matter, the wall anchorage at 4-foot spacing needs continuous ties only 32-feet-deep into the diaphragm (the full subdiaphragm width). With the steel joists spaced at 8 feet apart, the continuous ties will be divided into fourths. See Figure 17 for the subdiaphragm configuration with the applied wall anchorage load of 1,985 plf from Part 6a.

With the subdiaphragm depth being proposed as 32 feet (4 purlin bays), three-fourths of the wall anchorage load will be carried by the connection across the *first* purlin from the wall, one-half of the load across the *second* purlin, and one-fourth of the load across the *third* purlin.

Anchorage force at the wall and force in the first subpurlin strut:

F_p = 7,940 lbs

Connection force across the first purlin and force in the second subpurlin strut:

F_{p1} = 3/4 x (7,940 lbs) = 5,955 lbs

Connection force across the second purlin and force in the third subpurlin strut:

F_{p2} = 1/2 x (7,940 lbs) = 3,970 lbs

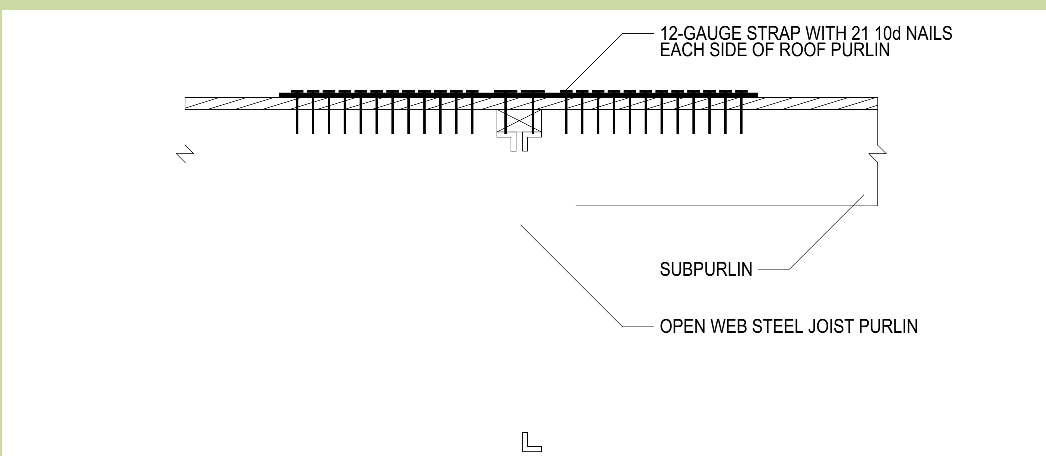
Connection force across the third purlin and force in the fourth subpurlin strut:

F_{p2} = 1/4 x (7,940 lbs) = 1,985 lbs

The design of the second, third and fourth subpurlin struts is similar to Part 6b. Usually the size of these struts is more a function of making a good connection than the actual combined stresses. Thicker framing members will provide for a stronger connection, reduce the chance of nail splitting, and provide greater allowance for field tolerances. Subpurlin struts in the subdiaphragm should be 3x framing or thicker.

The connection that bridges across the perpendicular purlin joists to create a subpurlin strut splice is typically accomplished with a nailed steel strap placed above the diaphragm sheathing (see Figure 18). This steel strap is commonly a pre-engineered, prefabricated strap with a valid ICC-ES Evaluation Report which lists the allowable load capacity. Otherwise, the building's design engineer may elect to develop their own design by checking nail capacity and steel strap capacity separately.

Figure 18: *Typical Subpurlin Continuous Tie Connection*



In this example, a 12-gauge steel strap with 10d common nails will be investigated. It is often beneficial to utilize a strap nail size that matches the diaphragm nail size for consistency. The number of nails necessary into the DF/L framing at the first purlin connection is as follows using allowable stress design:

$$\text{Number of 10d nails} = \frac{0.7(5{,}955 \text{ lbs})}{(127 \text{ lbs/nail})(1.6)} = 20.5 \quad \text{Use 21 nails minimum}$$

The allowable capacity of the 10d nails is obtained from NDS Tables 11P and multiplied by 1.6, the Seismic Load Duration Factor from Section 2.3.2. The design of the 12-gauge steel strap is not shown.

It is important to recognize that the steel stress check must be evaluated against seismic forces that have been increased by 1.4 in accordance with ASCE 7-10 Section 12.11.2.2.2. This 1.4 force increase for steel is a result of early strap failures observed in the 1994 Northridge earthquake in these types of buildings. It was determined that many of the steel straps lacked sufficient overstrength and ductility to accommodate seismic overloads, and thus a 1.4 force increase was deemed sufficient to forgo the reliance on ductility and simply accommodate the maximum expected wall anchorage force in the strap's available elastic range with some strain hardening.

When selecting a pre-engineered, prefabricated steel strap, the building's design engineer should compare the published strap capacities with the seismic forces increased by 1.4, unless the manufacturer can provide the governing capacities of the wood nailing and steel strap separately.

As the subpurlin strut force decreases away from the wall, the strap design may also be decreased. In this example, the second purlin strap connection could use this same strap for consistency or could contain a lighter strap using the same design process above.

6e. Subdiaphragm Chord Check

The subdiaphragm depth of 32 feet was selected to match the 8-foot joist spacing in order that a joist may function as one of the subdiaphragm chords. The other chord is traditionally the main diaphragm's chord located at the perimeter wall and is not governed by subdiaphragm forces (see Part 3i).

Determine the chord force in the joist and compare with the axial force found in Part 5b associated with the north/south wall anchorage. Because the two joist forces are from different orthogonal directions, they do not need to be combined. The 1.4 force increase for steel elements still applies to the joist acting as a subdiaphragm chord.

Subdiaphragm chord force from east/west seismic wall anchorage forces:

$$\text{Chord force in joist} = \frac{1{,}985 \text{ plf } (50 \text{ ft})^2}{8\ (32 \text{ ft})} \times 1.4 = 27{,}139 \text{ lbs}$$

Continuous tie force from north/south seismic wall anchorage forces:

Continuity tie force in joist = 15,880 lbs × 1.4 = 22,232 lbs (Does not govern)

The subdiaphragm chord force induced into the joist is greater than the previously determined axial force for the north/south wall anchorage loads, thus this joist needs to be flagged to the steel joist manufacturer as having a different design criteria. Alternatively, the building's design engineer could extend the subdiaphragm width from 32 feet to 40 feet for the purpose of reducing the chord force below the north/south tie force, keeping the joist designs more uniform.

It is worthwhile to recognize that the chord force induced into the joist is compression as the wall is being resisted from falling away from the building, thus the joist manufacture does need to design the joist for axial loads in both tension and compression.

6f. East/West Continuous Ties

As has been discussed up to this point, a primary purpose of the main diaphragm's continuous ties is to transmit the heavy out-of-plane wall anchorage loads into the main diaphragm uniformly. This is required in Seismic Design Categories C and higher per ASCE 7-10 Section 12.11.2.2.1. The magnitude of this tie force is simply the sum of the reactions from the two adjacent subdiaphragms.

As found in Part 6c, the unfactored subdiaphragm reaction is 49,625 lbs, and the main diaphragm's continuous ties resist subdiaphragm reactions. In addition, because the joist girders are of steel material and a part of the wall anchorage system, an additional 1.4 force increase is required per ASCE 7-10 Section 12.11.2.2.2.

Continuous tie force in girder line = 49,625 lbs × 2 × 1.4 = 139 kips

ASCE 7-10 Sections 1.4.2, 12.1.3 and 12.1.4 contain provisions for the minimum interconnection forces of elements within a building. As a minimum, smaller portions of buildings must be connected to the larger portions with an interconnection force of $0.133S_{DS}W$, but not less than $0.05W$, where W represents the dead weight of the smaller portion of the building being connected. The dead weight W is maximized near the center of the building where the smaller and larger portions of the building have nearly the same weight.

In this building example, the largest interconnection force will be near the building center and is based on tributary dead weight to the continuous cross-tie.

Weight (W) = roof structure weight + wall weight

$$W = 15\text{ psf }(50\text{ ft})\left(\frac{504\text{ ft}}{2}\right) + 116\text{ psf }(50\text{ ft})\ \frac{(37\text{ ft})^2}{2(32\text{ ft})} = 313\text{ kips}$$

The interconnection force is based on $S_{DS} = 1.0$ in this example:

$F_p = 0.133S_{DS}W = 0.133(1.0)313\text{ kips} = 41.6\text{ kips} < 139\text{ kips}$

The 139 kip seismic continuous tie force accumulated from the subdiaphragm design governs the axial load used in design of the east/west joist girders. The joist girder's design engineer is typically a specialty engineer associated with the joist manufacturer and will rely upon information within the construction documents for the girder design loads. IBC Section 2207.2 places the responsibility on the building's design engineer to provide joist girder load information with an indication of which load factors, if any, have already been applied.

In this example, the 139 kip axial continuous tie force is placed on the construction drawings with the indication that this is an unfactored seismic load *E*. In situations where the wind force *W* control the design, depending upon the load combinations, it is possible that both need to be reported on the drawings. The load combinations of ASCE 7-10 Sections 2.3.1 (strength design) and 2.4.1 (allowable stress design), as well as 2012 IBC Section 1605.3.2 (alternative allowable stress design) all contain different formulas when considering seismic *E* and wind *W* loadings, and thus the design engineer cannot simply compare the unfactored *E* and *W* to determine which to report. The steel joist industry is soon to transition in large part to strength design, but many continue to design under allowable stress design.

Part 7 – Other Issues Of Importance

7a. All-Wood Systems

Up until the mid 1990s, panelized roof systems were almost exclusively all wood instead of the hybrid steel joist, wood diaphragm system commonly used today. The all-wood systems typically have large glulam girders while the purlins are either 4x sawn lumber, 2½-inch or 3⅛-inch glulam beams, or open-web wood trusses. The transition to the hybrid system from an all-wood system was encouraged by a spike in timber prices in the early 1990s making steel look more financially attractive.

Despite today's popularity of the hybrid panelized roof system, all-wood systems are still used in certain situations. The following are some situations in which an all-wood panelized roof system may be preferable to a hybrid steel and wood system:

1. Insufficient material lead time: In some cases, acquiring steel joists and joist girders can be in the critical path of project completion. Joists and joist girders need significantly more time to be engineered and manufactured after the job is awarded to a contractor, and in vibrant economic times these joist manufacturers can be backlogged with production load. However, a design engineer who specifies steel joists clearly on the plans and a contractor familiar with their place in the project scheduling can minimize these problems.

2. Dust sensitive occupants: Buildings with food and drink processing or other operations where cleanliness is an issue may prefer an all-wood beam system over a hybrid system because the horizontal shelf surfaces within the steel joists and joist girder members collect dust and are too difficult to routinely clean. Using rectangular glulam beams and/or sawn purlins eliminates the horizontal shelf surfaces providing a cleaner interior environment.

3. Smaller scale structures: It is often desirable to minimize the number of different trades within a smaller job to simplify the project scheduling and coordination. Utilizing an all-wood system on a smaller job may have benefits in the consolidation of suppliers and subcontractors.

4. Aesthetics: The visual warmth of exposed wood surfaces is often desired by architects in certain occupancies with exposed structural framework. Architectural grade glulam wood beams in conjunction with a modified panelized roof system have been used in the past to create a craftsman style appearance desired by the architect and owner.

7b. Metal Deck Diaphragms

Wood and hybrid panelized roof systems occasionally compete with metal deck diaphragm systems. Metal deck systems are very common in the eastern United States, and less common in the western United States. When determining which system to use, the following factors may be considered:

1. Cost: Wood and hybrid panelized roof systems are more cost effective and erected faster than metal deck systems, especially where rigid insulation is not deemed necessary above the diaphragm.

2. Thermal Expansion: Wood has a significantly smaller amount of thermal expansion compared with steel. The thermal growth and contraction that steel decking undergoes creates a need for frequent thermal expansion joints within the building, unlike those with wood diaphragms.

7c. Moisture Condensation Issues

As in all construction, enclosed building spaces tend to trap moisture in the air which can lead to condensation issues if not properly addressed or controlled. When warm moist air contacts cooler steel building components such as sprinkler piping, steel joists, and subpurlin joist hangers, condensation may occur and lead to ongoing corrosion. Because low-sloped wood and hybrid panelized roof systems do not necessarily require rigid insulation above the roof, the underlying steel roof components are more susceptible to condensation if proper ventilation is not provided.

While most buildings with low-slope roofs have never had a performance issue with moisture condensation, a combination of factors can lead to an issue in some buildings. The publication *Moisture Control in Low Slope Roofs* by APA (APA Form No. R525) provides some guidance in mitigating this issue.

7d. Fire Sprinkler System Considerations

In buildings with high-pile storage or tall storage rack systems, the fire sprinkler system may have a dramatic impact on the structural design. Traditionally, large storage rack systems required fire sprinkler systems within the storage racks themselves to protect the product from fire. These "in-rack" fire sprinkler systems are expensive and limit the flexibility of the occupant to quickly move storage racking layouts. Today's state-of-the-art distribution facilities and warehouse structures utilize a fire suppression system called ESFR (Early Suppression, Fast Response). This fire suppression system utilizes special high pressure water lines below the roof with special high volume sprinkler heads. The use of this system allows building occupants to forgo the installation of in-rack fire sprinklers, but the ESFR system dictates some of structural design parameters. For example, an important part of the ESFR system is that the fire sprinkler water discharge is free to travel through the framing members with limited obstruction. The requirements of ESFR limit the maximum size of solid structural members in order to minimize the obstruction to the water spray pattern. Compliance with ESFR almost certainly requires the heavy use of open web trusses to minimize spray obstructions.

Another issue associated with the ESFR system is the designated area associated with each sprinkler head. Compliance requires that each head have between 80 and 100 square feet of unobstructed spray area, and with the normal 10-foot branch line pipe spacings, the 8-foot purlin spacings work perfectly into the head spacings (8 ft x 10 ft = 80 ft^2). In addition, the purlin spacings can increase up to 10 feet and still comply with the unobstructed spray area range (10 ft x 10 ft = 100 ft^2).

Designers of distribution facilities and warehouse occupancies who do not consider the ramifications of their designs with the ESFR fire sprinkler system will often find themselves redesigning the structure late in the process to accommodate the fire protection requirements.

Part 8 – Closure

Panelized roof systems grew out of a need for an efficient use of labor and materials resulting in rapid and safe erection. Today's panelized roof market has matured to take advantage of new state-of-the-art materials, design techniques, and construction processes. Current building codes have never been safer, evolving as more is learned from each earthquake, and this publication has provided an illustrative example and coordinated commentary to assist the designer. Acceptance of the panelized roof structure continues to grow as designers and contractors appreciate its efficiency and benefits in flat diaphragm construction.