

# Wind Design of Timber Panelized Roof Structures



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*2012 International Building Code (IBC) Edition*

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*Developed for WoodWorks by*

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# Free Design and Engineering Support for Non-residential and Multi-family Wood Buildings

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# Table of Contents

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<b>Part 1 – Overview</b> .....	4
<b>Part 2 – Main Wind Force Resisting System (MWFRS)</b> .....	8
2a) Design Procedure.....	8
2b) Diaphragm Loads .....	11
2c) Diaphragm Shears.....	12
2d) Diaphragm Penetrations .....	16
2e) Diaphragm Chord Design.....	19
<b>Part 3 – Diaphragm Deflection Check</b> .....	21
<b>Part 4 – Components and Cladding</b> .....	22
4a) Design Procedure .....	22
4b) Wall-Tie Loads .....	27
4c) Continuity Ties and Subdiaphragms .....	31
4d) Subpurlin Tie Connections.....	34
4e) Subdiaphragm Chord Check.....	35
4f) East/West Continuous Ties .....	35
<b>Part 5 – Uplift</b> .....	36
<b>Part 6 – Other Issues Of Importance</b> .....	37
6a) All-Wood Systems.....	37
6b) Metal Deck Diaphragms.....	37
6c) Moisture Condensation Issues.....	38
6d) Fire Sprinkler System Considerations.....	38
<b>Part 7 – Closure</b> .....	38

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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](mailto:info@woodworks.org).

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# Overview

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Panelized timber roof construction is the dominant form of roof structure along the West Coast and Southwest region 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 (purlins) and joist girders. 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 with substantial clear-height in a wind zone most commonly found in the United States.

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# Codes and Reference Documents Used

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This design example is not intended to give a thorough, or even complete, explanation of how to use the codes and standards referenced. It is assumed that the engineer has a good working knowledge of the following documents and their application:

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)*

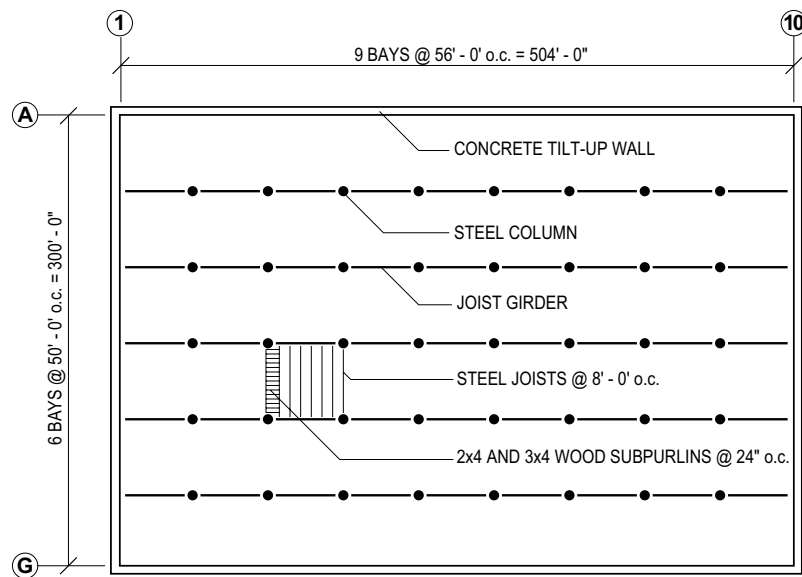
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## Given Information

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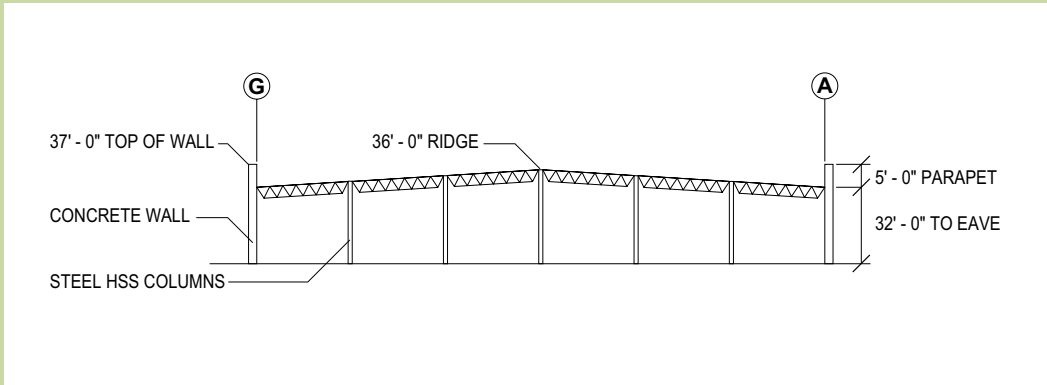
- BUILDING USE:** Warehouse storage and/or manufacturing
- ROOF STRUCTURE:** Panelized hybrid roof structure consisting of wood structural panel sheathing over wood subpurlins, supported by repetitive open web steel joists and joist girders
- ROOF SLOPE:** ¼ inch per foot minimum roof slope is typical. Panelized roof surfaces are often sloped toward the roof drains/scuppers in the adjacent bay and this may result in a roof slope of ½ inch per foot. In any case, the roof angle is less than 5 degrees.

**Figure 1. Roof Framing Plan**



**WALL STRUCTURE:** Concrete tilt-up wall panels (site cast precast)  
 Top of wall at 37 feet above finish floor  
 Top of sheathing varies from 32 to 36 feet

**Figure 2. Building Section**

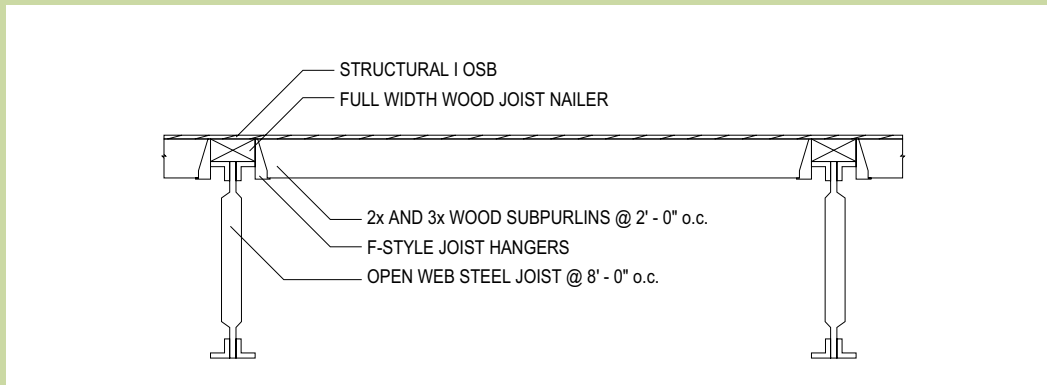


**ROOF WEIGHTS**

Roofing + re-roof	5.0 psf
Sheathing	2.0
Subpurlins	1.0
Steel Joists	2.0
Joist Girders	1.5
Sprinklers	2.0
Insulation	0.5
Miscellaneous	1.0
<hr/>	
Dead load	15.0 psf
Live load	20.0 psf (reducible)



**Figure 3. Enlarged Roof Section View**



Sawn Dimensional Lumber: Douglas-fir/Larch (DF/L), S-DRY  
 Sheathing: Structural I OSB in conformance with DOC PS-2 (15/32 inch typical)  
 Ultimate Design Wind Speed: 115 MPH, 3 second gust, Risk Category II  
 Exposure Category: C  
 Topographic Effects: None  
 Building fully enclosed (not partially enclosed)

### 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	Sprinkler Increase	Maximum
IBC Table 503	IBC 504.2	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 side yards and/or public ways.

Building Area = 300 ft x 504 ft = 151,200 ft<sup>2</sup> < Unlimited *Okay*

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## Part 2 – Main Wind Force Resisting System (MWFRS)

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### 2a. Design Procedure

Per ASCE 7-10 Section 26.1.2.1: The Envelope Procedure of Chapter 28, Part 1 is used for typical warehouse panelized roofs.

Per ASCE 7-10 Table 28.2-1 and design assumptions, the following can be calculated:

- Step 1. Risk Category II (given) ..... **ASCE 7-10 Table 1.5-1**
- Step 2. Ultimate Design Wind Speed  $V_{ult} = 115$  MPH (given) ..... **ASCE 7-10 Figure 26.5-1A**
- Step 3. Wind Load Parameters:
- a. Wind Directionality Factor  $K_d = 0.85$   
per Table 26.6-1 (Buildings)..... **ASCE 7-10 Section 26.6**
  - b. Exposure Category = C (given) ..... **ASCE 7-10 Section 26.7**
  - c. Topographic Factor  $K_{zt} = 1.0$  (given) ..... **ASCE 7-10 Section 26.8**
  - d. Enclosure Classification is Enclosed (given)..... **ASCE 7-10 Section 26.10**
  - e. Internal pressure coefficients  $GC_{pi} = \pm 0.18$   
per Table 26.11-1..... **ASCE 7-10 Section 26.11**
- Step 4.  $K_h$  is determined using the mean roof height  $h$ . For roof slopes < 10 degrees, this is evaluated at the roof eave height. For north/south (N/S) loads, this is 32 feet. For east/west (E/W) loads, the roof "eave" (top of sheathing at perimeter walls) varies so the average of 34 feet is used:
- a.  $K_h = 2.01 (z/z_g)^{(2/\alpha)}$  ..... **ASCE 7-10 Table 28.3-1**
    - i.  $z = h = 32.0$  (N/S) &  $34.0$  (E/W)
    - ii.  $\alpha$  is 9.5 &  $z_g$  is 900 feet for Exposure C ..... **ASCE 7-10 Table 26.9-1**
- $K_h$  (N/S) = 1.00
- $K_h$  (E/W) = 1.01 ..... **ASCE 7-10 Section 28.3.1**





Step 5. Velocity pressure at  $z = h$  is

$$q_h = 0.00256K_zK_{zt}K_dV^2 \dots\dots\dots \text{ASCE 7-10 Eq. 28.3-1}$$

$$q_h \text{ (N/S)} = 28.78 \text{ psf}$$

$$q_h \text{ (E/W)} = 29.07 \text{ psf} \dots\dots\dots \text{ASCE 7-10 Section 28.3.2}$$

Step 6: External pressure coefficient  $GC_{pf}$  is given in Figure 28.4-1 for low-rise buildings. Load Case A applies to loads in the same direction as the purlins (N/S). Load Case B applies to load in the E/W direction:

Load Case A:

Surface	1	2	3	4	1E	2E	3E	4E
$GC_{pf}$	0.4	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43

Load Case B:

Surface	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
$GC_{pf}$	-0.45	-0.69	-0.37	-0.45	0.4	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43

+ = toward surface

- = away from surface

Surfaces 1E-6E occur at a distance  $2a$  from the end of a wall.

$a$  = 10% of shortest plan dimension or 40%  $h$  (whichever is smaller), but not less than 4% of shortest plan dimension or 3 ft

$a$  = 12.8 ft,  $2a$  = 25.6 ft (N/S)

$a$  = 13.6 ft,  $2a$  = 27.2 ft (E/W) ..... **ASCE 7-10 Table 28.4-1**

Step 7: Design wind load  $p = q_h[(GC_{pf}) - (GC_{pi})]$  (lb/ft<sup>2</sup>)..... **ASCE 7-10 Eq. 28.4-1, Section 28.4.1**

The effects of internal pressure cancel each other out, so we are left with the effect of surfaces:

N/S: 1 + 4 (typical), and 1E + 4E (End Zone), Load Case A

E/W: 5 + 6 (typical), and 5E + 6E (End Zone), Load Case B

Expanding our  $GC_{pf}$  table from Step 6 we get:

N/S:

Surface	1	2	3	4	1E	2E	3E	4E
$GC_{pf}$	0.40	-0.69	-0.37	-0.29	0.61	-1.07	-0.53	-0.43
$p$ (no $GC_{pi}$ )	11.51	-19.86	-10.65	-8.35	17.56	-30.79	-15.25	-12.38

$$p \text{ (N/S) total} = 11.51 \text{ psf} + 8.35 \text{ psf} = 19.86 \text{ psf typical}$$

$$p \text{ (N/S) total} = 17.56 \text{ psf} + 12.38 \text{ psf} = 29.94 \text{ psf End Zone}$$

E/W:

Surface	1	2	3	4	5	6	1E	2E	3E	4E	5E	6E
$GC_{pf}$	-0.45	-0.69	-0.37	-0.45	0.40	-0.29	-0.48	-1.07	-0.53	-0.48	0.61	-0.43
$p$ (no $GC_{pi}$ )	-13.08	-20.06	-10.76	-13.08	11.63	-8.43	-13.95	-31.10	-15.41	-13.95	17.73	-12.50

$$p \text{ (E/W) total} = 11.63 \text{ psf} + 8.43 \text{ psf} = 20.06 \text{ psf typical}$$

$$p \text{ (E/W) total} = 17.73 \text{ psf} + 12.50 \text{ psf} = 30.23 \text{ psf End Zone}$$

Note that for low slope roofs, the  $GC_{pf}$  factors are the same for Load Case A and Load Case B. Therefore the only difference in pressure is due to the difference in  $K_h$ .

Parapets: The loads above apply to walls below the eave. ASCE 7-10 Section 28.4.2 gives the equation for the wind load to MWFRSs from parapets. ....**ASCE 7-10 Section 28.4.2**

$$p_p = q_p GC_{pn} \text{ .....ASCE 7-10 Eq. 28.4-2}$$

$q_p$  is evaluated at top of parapet (37 feet), so we must recalculate  $K_z$  (Step 4)

$$K_z = 1.03, \text{ which gives}$$

$$q_p = 29.64 \text{ psf}$$

$$GC_{pn} = 1.5 \text{ (windward parapet)}$$

$$GC_{pn} = -1.0 \text{ (leeward parapet)}$$

$$p_p = 44.46 \text{ psf (windward)}$$

$$p_p = -29.64 \text{ psf (leeward)}$$

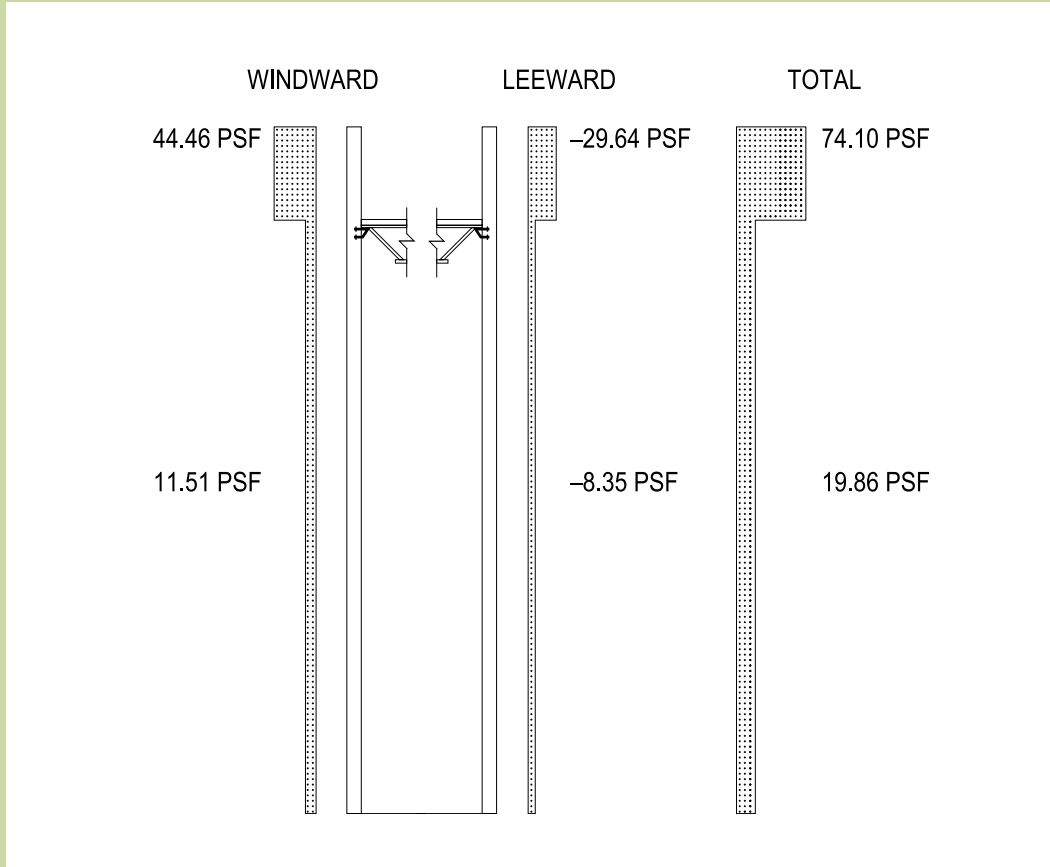
$$p_p = 44.46 \text{ psf} + 29.64 \text{ psf} = 74.10 \text{ psf (total)}$$

See Figure 4 for the load distribution for N/S loads in the typical zone.

It is worth noting that ASCE 7-10 Section 28.4.4 requires a minimum design wind load of 16 psf for the walls and 8 psf for the roof projected on a vertical plane. However, for this example, this will not control.



**Figure 4.** North/South MWFRS Loads



## 2b. Diaphragm Loads

These wind loads are resisted by the wood roof diaphragm. The roof diaphragm serves an important function, spanning horizontally like a flat beam from shear wall to shear wall, resisting horizontal wind forces. Summing moments about the base of the wall gives a linear load at the roof:

N/S

$$W = \frac{\left[ \frac{(19.86 \text{ psf}) \times (32 \text{ ft})^2}{2} + 74.10 \text{ psf} \times 5 \text{ ft} \times (32 \text{ ft} + 2.5 \text{ ft}) \right]}{32 \text{ ft}} = 717.21 \text{ plf typical}$$

Similarly:

$$W = 878.34 \text{ plf End Zone}$$

E/W

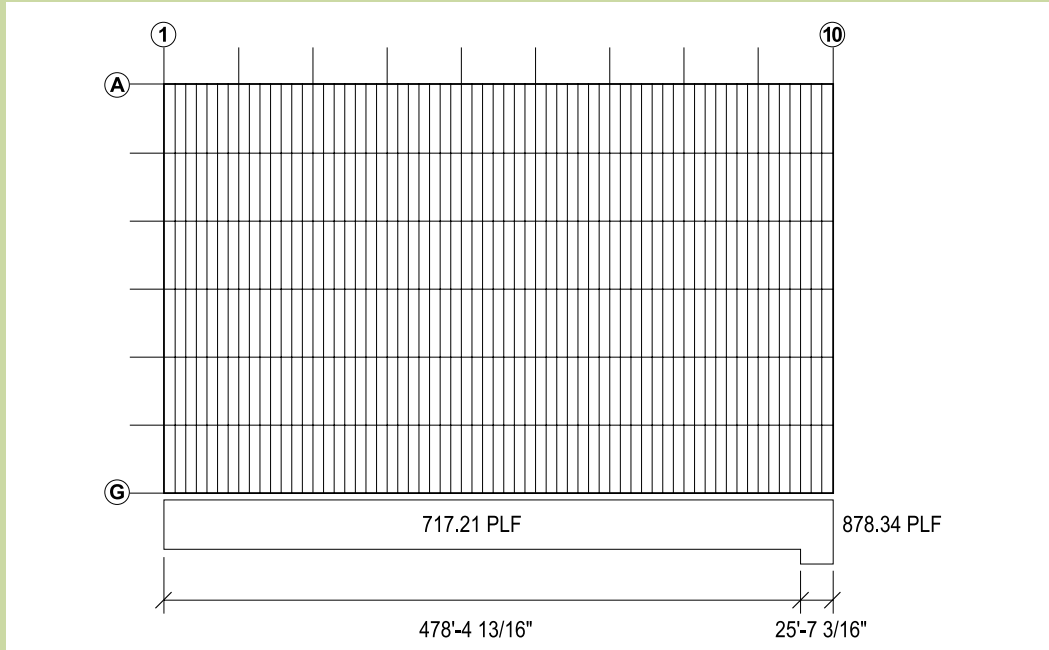
$$W = 573.10 \text{ plf typical}$$

$$W = 746.06 \text{ plf End Zone}$$

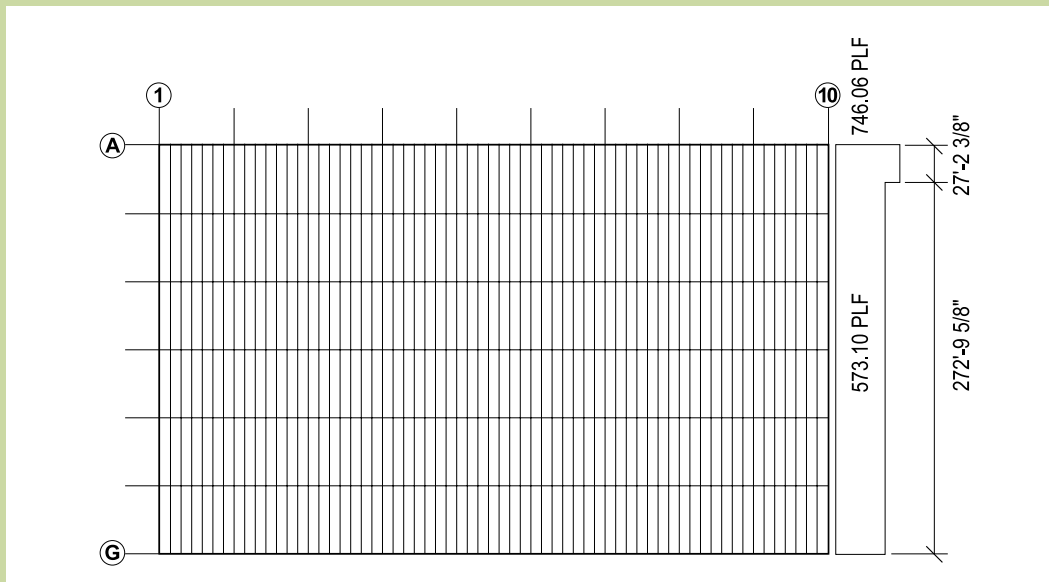
## 2c. Diaphragm Shears

Figure 5 shows loading in the transverse direction (N/S) and Figure 6 shows loading in the longitudinal direction (E/W). The highest diaphragm shear will occur at grid 10 (N/S) and grid A (E/W). We will then use the same diaphragm shears for grids 1 and G, respectively.

**Figure 5. North/South Applied Loads**



**Figure 6. East/West Applied Loads**



Using a flat-beam analogy, N/S wind forces are resisted by shear walls on grid lines 1 and 10, and E/W wind forces are resisted by shear walls on grid lines A and G.

We can calculate the total shear at grid 1:

$$V_1 = \frac{\left[ \frac{717.21 \text{ plf} \times (504 \text{ ft})^2}{2} + \frac{(878.34 \text{ plf} - 717.21 \text{ plf}) \times (25.6 \text{ ft})^2}{2} \right]}{(504 \text{ ft})} = 180.84 \text{ kips}$$

Similarly:

$$V_{10} = 184.76 \text{ kips}$$

$$V_A = 90.46 \text{ kips}$$

$$V_G = 86.18 \text{ kips}$$

Unit shear along grid 10 (N/S) and A (E/W) is worst case and calculated by taking the total shear and dividing by the total width of diaphragm:

$$v_{nS} = 184.76 \text{ kips}/300 \text{ ft} = 616 \text{ plf}$$

$$v_{eW} = 90.46 \text{ kips}/504 \text{ ft} = 179 \text{ plf}$$

The maximum diaphragm shears were determined for each orthogonal direction. The maximum unit shear demand is in the N/S direction with  $v_{nS} = 616$  plf. Note that we have been working with the ultimate wind speed. Wind loads shall be multiplied by 0.6 in all allowable stress load combinations per IBC 1605.3:

$$v_{nS} = 0.6 \times 616 \text{ plf} = 370 \text{ plf (ASD)}$$

$$v_{eW} = 0.6 \times 179 \text{ plf} = 107 \text{ plf (ASD)}$$

Allowable diaphragm shear capacities are provided in SDPWS-2008 Section 4.2 for a wide range of conditions. The first aspect to decide upon is the desired sheathing thickness, which ranges from 5/16 inch to 23/32 inch. A feature unique to panelized roof systems is that the wood structural panel (plywood or OSB) is oriented with the long direction parallel to the subpurlin supports (See Figure 7). This is actually spanning the weak direction for gravity loads, but is traditionally done to accommodate the efficiencies of the 8-foot panelized module and so that all panel edges are 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 where better protection against extended rain delays is desired. The panelized sheathing system is naturally fully blocked and typically follows layout Case 2 (E/W loading) and 4 (N/S loading) illustrated in SDPWS-2008 Tables 4.2A and B.

A wood diaphragm resisting lateral forces must have its aspect ratio checked against the limitations in 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 N/S loading direction is the critical  $L/W$  ratio.

$$L/W = 504 \text{ ft}/300 \text{ ft} = 1.68 < 4 \quad \text{Okay}$$

Looking at SDPWS-2008 Table 4.2A (shown on next page), using 15/32-inch Structural 1 sheathing with 10d common nails having 1 1/2 inch minimum penetration and 2x minimum nominal framing members, we see that the minimum blocked diaphragm nominal unit shear value is 895 plf for wind loads (column B). Per footnote 1, all values must be modified for LRFD or ASD use according to SDPWS-2008 Section 4.2.3. For ASD we divide the values by 2.0:

$$v_{all} = 895 \text{ plf} \div 2 = 447.5 \text{ plf (ASD)}$$

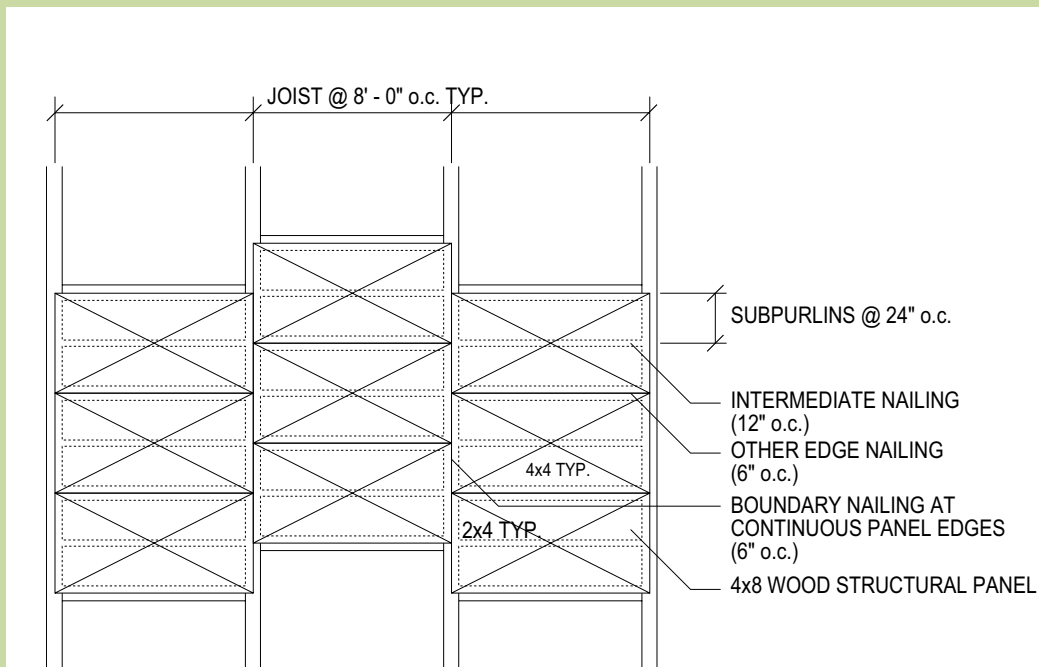
This is greater than our maximum required shear in either direction.

$$v_{ns} = 370 \text{ plf} < 447.5 \text{ plf}$$

$$v_{ew} = 107 \text{ plf} < 447.5 \text{ plf}$$

The diaphragm nailing can be a minimum 6 inches on center (o.c.) at diaphragm boundaries and (ledgers) continuous panel joints (purlins at 8 feet), and all other panel edges (every other 2x). Depending on the shape of the building, it is common to have diaphragm shears much higher than the minimum. Wood panelized roofs can resist diaphragm shears due to wind up to 2190 plf (ASD). In these cases, it is appropriate to vary the nailing zones across the diaphragm for greater efficiency. See the WoodWorks design example, *Seismic Design of Timber Panelized Roof Structures*, for an excellent explanation of this process.

**Figure 7. Sheathing Layout and Nailing**



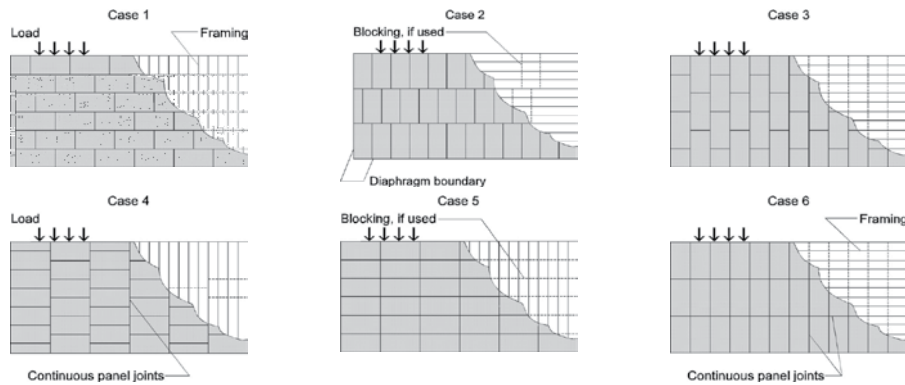
**Table 4.2A Nominal Unit Shear Capacities for Wood-Frame Diaphragms**  
**Blocked Wood Structural Panel Diaphragms<sup>1,2,3,4</sup>**

Sheathing Grade	Common Nail Size	Minimum Fastener Penetration in Framing Member or Blocking (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Nailed Face at Adjoining Panel Edges and Boundaries (in.)
Structural I	6d	1-1/4	5/16	2 3
	8d	1-3/8	3/8	2 3
	10d	1-1/2	15/32	2 3
Sheathing and Single-Floor	6d	1-1/4	5/16	2 3
			3/8	2 3
	8d	1-3/8	3/8	2 3
			7/16	2 3
			15/32	2 3
	10d	1-1/2	15/32	2 3
			19/32	2 3

- Nominal unit shear capacities shall be adjusted in accordance with 4.2.3 to determine ASD allowable unit shear capacity and LRFD factored unit resistance. For general construction requirements see 4.2.6. For specific requirements, see 4.2.7.1 for wood structural panel diaphragms. See Appendix A for common nail dimensions.
- For species and grades of framing other than Douglas-Fir-Larch or Southern Pine, reduced nominal unit shear capacities shall be determined by multiplying the tabulated nominal unit shear capacity by the Specific Gravity Adjustment Factor =  $[1 - (0.5 - G)]$ , where  $G$  = Specific Gravity of the framing lumber from the *NDS* (Table 11.3.2A). The Specific Gravity Adjustment Factor shall not be greater than 1.
- Apparent shear stiffness values,  $G_a$ , are based on nail slip in framing with moisture content less than or equal to 19% at time of fabrication and panel stiffness values for diaphragms constructed with either OSB or 3-ply plywood panels. When 4-ply or 5-ply plywood panels or composite panels are used,  $G_a$  values shall be permitted to be increased by 1.2.
- Where moisture content of the framing is greater than 19% at time of fabrication,  $G_a$  values shall be multiplied by 0.5.

A SEISMIC									
Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)									
6		4		2-1/2		2			
Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)									
6		6		4		3			
$v_s$ (plf)	$G_a$ (kips/in.)	$v_s$ (plf)	$G_a$ (kips/in.)	$v_s$ (plf)	$G_a$ (kips/in.)	$v_s$ (plf)	$G_a$ (kips/in.)		
	OSB PLY	OSB PLY	OSB PLY	OSB PLY	OSB PLY	OSB PLY	OSB PLY		
370	15 12	500	8.5 7.5	750	12 10	840	20 15		
420	12 9.5	560	7.0 6.0	840	9.5 8.5	950	17 13		
540	14 11	720	9.0 7.5	1060	13 10	1200	21 15		
600	12 10	800	7.5 6.5	1200	10 9.0	1350	18 13		
640	24 17	850	15 12	1280	20 15	1460	31 21		
720	20 15	960	12 9.5	1440	16 13	1640	26 18		
340	15 10	450	9.0 7.0	670	13 9.5	760	21 13		
380	12 9.0	500	7.0 6.0	760	10 8.0	860	17 12		
370	13 9.5	500	7.0 6.0	750	10 8.0	840	18 12		
420	10 8.0	560	5.5 5.0	840	8.5 7.0	950	14 10		
480	15 11	640	9.5 7.5	960	13 9.5	1090	21 13		
540	12 9.5	720	7.5 6.0	1080	11 8.5	1220	18 12		
510	14 10	680	8.5 7.0	1010	12 9.5	1150	20 13		
570	11 9.0	760	7.0 6.0	1140	10 8.0	1290	17 12		
540	13 9.5	720	7.5 6.5	1060	11 8.5	1200	19 13		
600	10 8.5	800	6.0 5.5	1200	9.0 7.5	1350	15 11		
580	25 15	770	15 11	1150	21 14	1310	33 18		
650	21 14	860	12 9.5	1300	17 12	1470	28 16		
640	21 14	850	13 9.5	1280	18 12	1460	28 17		
720	17 12	960	10 8.0	1440	14 11	1640	24 15		

B WIND			
Nail Spacing (in.) at diaphragm boundaries (all cases), at continuous panel edges parallel to load (Cases 3 & 4), and at all panel edges (Cases 5 & 6)			
6	4	2-1/2	2
Nail Spacing (in.) at other panel edges (Cases 1, 2, 3, & 4)			
6	6	4	3
$v_w$ (plf)	$v_w$ (plf)	$v_w$ (plf)	$v_w$ (plf)
520	700	1050	1175
590	785	1175	1330
755	1010	1485	1680
840	1120	1680	1890
895	1190	1790	2045
1010	1345	2015	2295
475	630	940	1065
530	700	1065	1205
520	700	1050	1175
590	785	1175	1330
670	895	1345	1525
755	1010	1510	1710
715	950	1415	1610
800	1065	1595	1805
755	1010	1485	1680
840	1120	1680	1890
810	1080	1610	1835
910	1205	1820	2060
895	1190	1790	2045
1010	1345	2015	2295

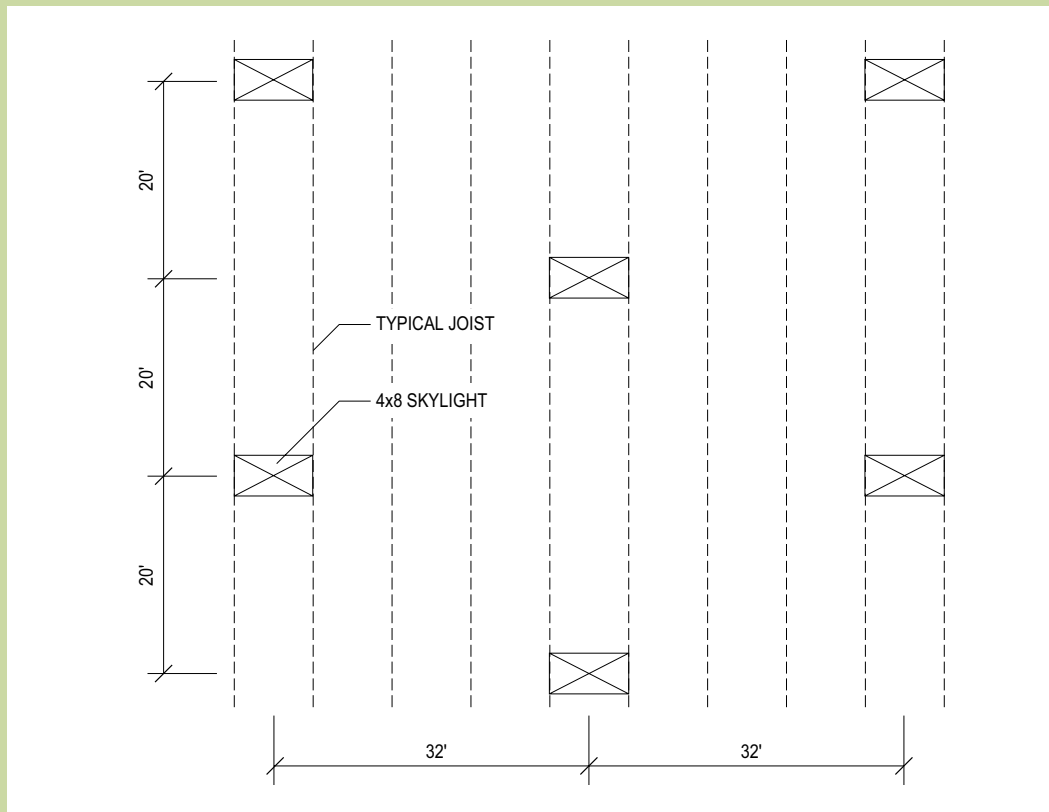


## 2d. Diaphragm Penetrations

Warehouse and manufacturing buildings 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. These penetrations reduce the effective diaphragm cross-sectional width; the unit shears will correspondingly increase at these locations.

N/S: In this example, the architect has proposed the repetitive skylight layout as shown below, 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 E/W direction.

**Figure 8.** 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 shear loads in the N/S 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 as seen in Figure 9.



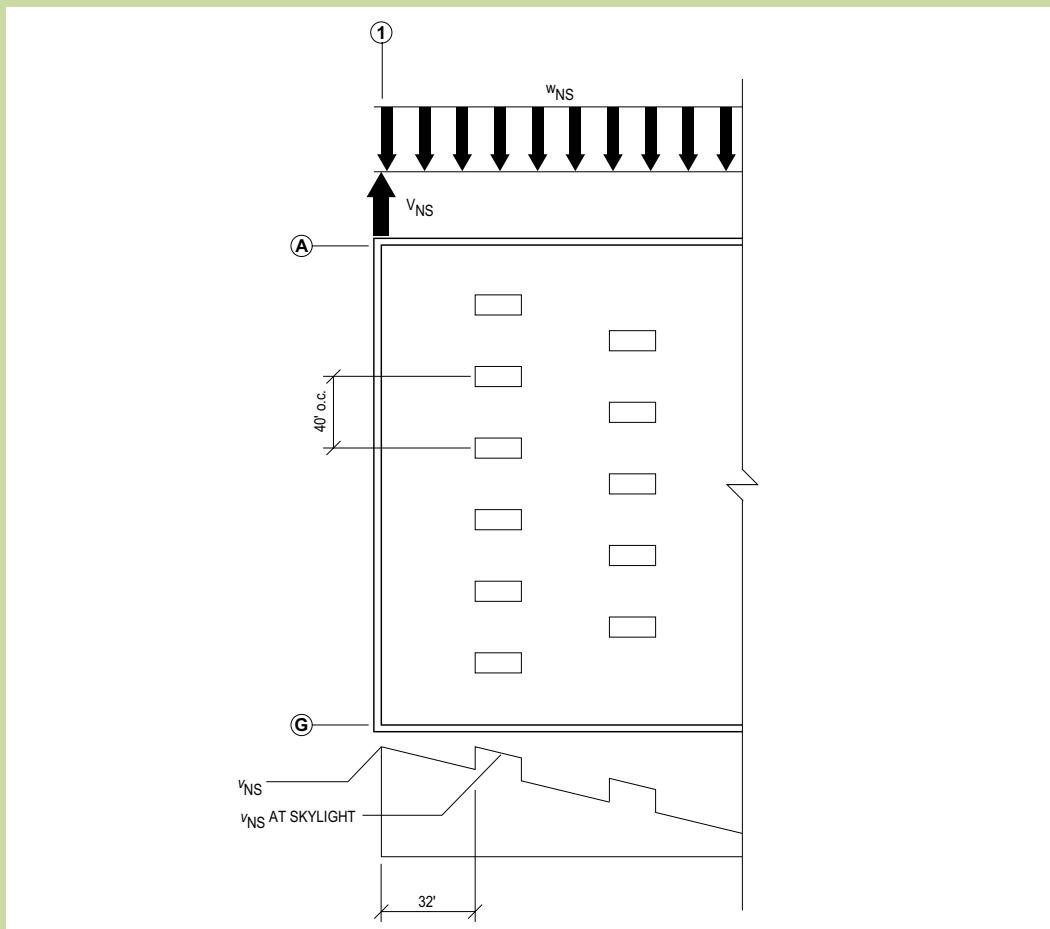


ASCE7-10 Section 12.10.1 requires that, at openings in diaphragms, the design shall assure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within the shear and tension capacity of the diaphragm. IBC Section 2305.1.1 requires that openings in shear panels that materially affect their strength shall be detailed on the plans, and shall have their edges adequately reinforced to transfer all shear stresses. Connections and blocking shall extend into the diaphragm a sufficient distance to develop the force being transferred into the diaphragm.

ATC-7 and Edward F. Diekmann, SE recommended that, at small openings, minimal reinforcing at the corners of the openings should extend a minimum distance equal to the depth or width of the opening in the direction under consideration. Diekmann's treatment of openings in diaphragms can be referenced in *Design of Wood Diaphragms, Journal of Materials Education*, University of Wisconsin, Madison, August 1982.

Skylight and smoke vent penetrations are typically confined between the 8-foot spaced purlins and thus the purlins provide continuity past the penetrations to help dissipate and transfer diaphragm forces around the openings. In the orthogonal direction, localized diaphragm continuity can be provided at each corner with steel straps at the subpurlins. If a thorough analysis of the stresses around the opening is desired, see APA Report 138 (Tissell, Elliot), Form E315H, revision 2000.

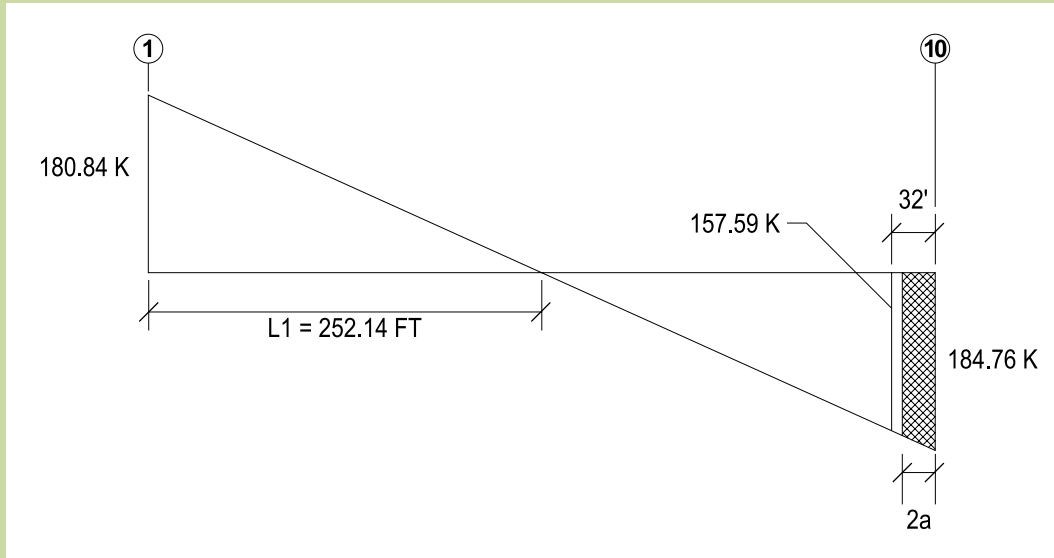
**Figure 9.** Diaphragm Shear Distribution at Skylights



The highest unit shear occurs at the edge of the first skylight, 32 feet from grid 10. The design shear at this location can be found by reducing the total shear from that calculated at grid 10 by subtracting the wind load within 32 feet of grid 10:

$$V = 184,760 \text{ lbs} - (26.2 \text{ ft}) (878.34 \text{ plf}) - (32 \text{ ft} - 26.2 \text{ ft})(717.21 \text{ plf}) = 157,587 \text{ lbs}$$

**Figure 10.** North/South Diaphragm Shear Diagrams



The unit shear in the diaphragm without the skylight effect becomes:

$$V_{ns \text{ at } 32 \text{ ft}} = 153,670 \text{ lbs} / 300 \text{ ft} = 512 \text{ plf}$$

The unit shear at this location including the skylight effect is:

$$V_{ns \text{ AT SKYLIGHTS}} = V_{NS} (1/0.90) = 512 \text{ plf} (1/0.90) \text{ plf} = 569 \text{ plf at skylight penetrations}$$

$$V_{ns \text{ AT SKYLIGHTS}} = 0.6 \times 569 \text{ plf} = 341 \text{ plf at skylight penetrations (ASD)}$$

This is less than our unit shear at grid 10 and therefore does not govern the diaphragm design. If it did, the nailing pattern would be adjusted to account for the higher shear requirement.

E/W: Similarly, in the E/W direction, the skylights will remove 8 feet of diaphragm 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.

$$\text{Intact diaphragm capacity} = 1 - (8 \text{ ft}/64 \text{ ft}) = 0.875 \text{ or } 87.5\%$$

If unit shears were higher, it would be prudent to reduce the shear based on the distance from grid A to the first skylight, as shown previously for the N/S direction. Because our unit shear is so low in this direction, this step will be skipped. Taking no reduction we get:

$$V_{ew \text{ AT SKYLIGHTS}} = V_{ew} (1/0.875) = 107 \text{ plf} (1/0.875) = 122 \text{ plf at skylight penetrations (ASD) Okay}$$

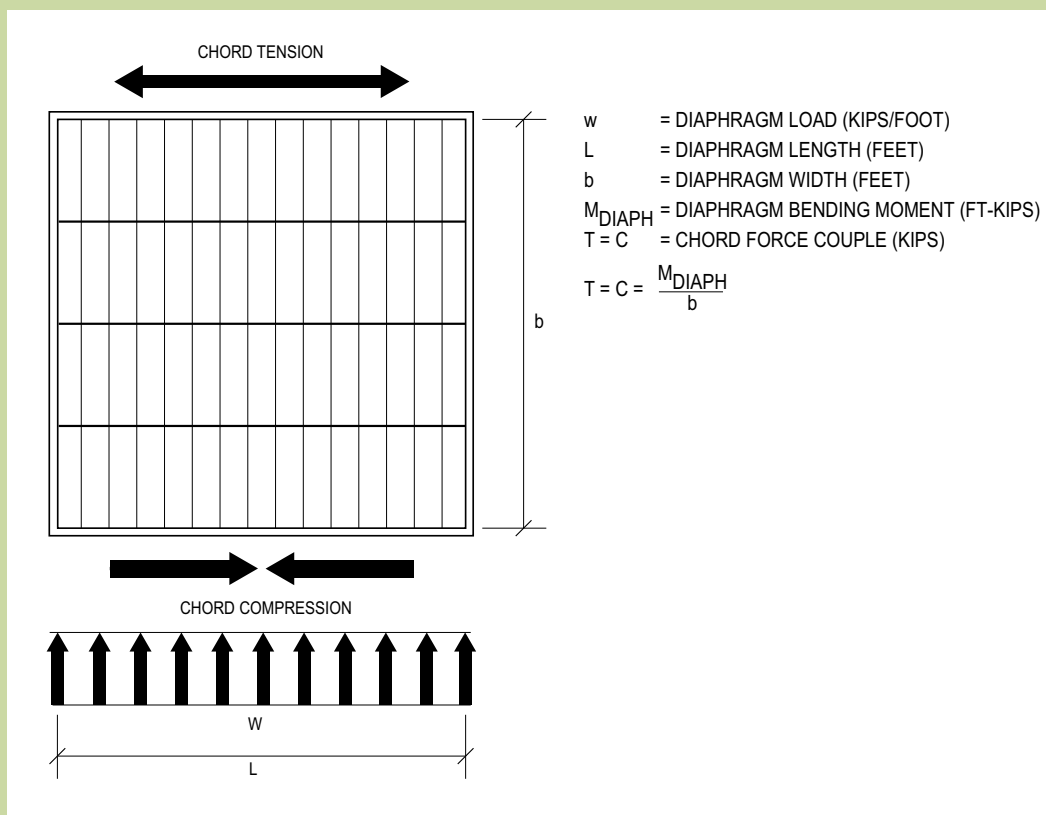


## 2e. Diaphragm Chord Design

Recall that a flexible diaphragm may be thought of as a flat 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 are often 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 near the center of the diaphragm's span where the maximum moment coincides.

**Figure 11. Traditional Diaphragm Chord Forces**



N/S: Before using the equations derived in Figure 11, the maximum moment must be calculated. The simplest method is to calculate the area of the shear diagram triangle from grid 1.

The distance to theoretical zero shear as shown in Figure 10 is calculated as:

$$L_1 = \frac{V_1}{w_1} = \frac{180.84 \text{ kips}}{717.21 \text{ plf}} = 252.14 \text{ ft}$$

Next we calculate  $M = \frac{1}{2} V_1 L_1 = 22,799 \text{ kips} - \text{ft}$

Tensile chord force for our building example is determined:

$$T = \frac{M}{b} = \frac{22,799 \text{ kips} - \text{ft}}{300 \text{ ft}} = 76.0 \text{ kips}$$

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

$$A_s = \frac{\phi T}{F_y} = \frac{0.9(76.0) \text{ kips}}{60 \text{ ksi}} = 1.14 \text{ in}^2$$

Using #6 reinforcing bars ( $A_s = 0.44 \text{ in}^2$  each), the number of bars required is:

$$\text{Number of \#6's} = \frac{1.14 \text{ in}^2}{0.44 \frac{\text{in}^2}{\text{bar}}} = 2.59 \text{ bars}$$

Use (3) #6 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. The design of the rebar splices is not part of this example, but needs to be considered by the designer.



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## Part 3 – Diaphragm Deflection Check

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As any flexible diaphragm is laterally loaded, it will deform and deflect similarly 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 collapse. SDPWS-2008 4.2.1 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 collapse. For non-structural elements, the intent is to prevent failure of connections or self-integrity that could result in a localized falling hazard.

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 for computing deflections of wood structural panel diaphragms by considering flexural bending, shear deformation, nail slip and chord 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.

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{.25vL}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \dots\dots\dots \text{SDPWS-2008 Eq. 4.2-1}$$

This equation works well for a diaphragm with uniform nailing. If the diaphragm has multiple nailing zones, it is appropriate to use a modified shear deformation equation. An example of this can be found in the WoodWorks design example, *Seismic Design of Timber Panelized Roof Structures*.

Reviewing our diaphragm, the following parameters are determined below:

N/S:

- $v = 616$  plf ..... **Part 2c**
- $L = 504$  ft
- $W = 300$  ft
- $E = 29,000,000$  psi
- $A = 3$  #6 bars =  $3 \times 0.44 = 1.32$  in<sup>2</sup> ..... **Part 2e**
- $G_a = 24$  kips/in ..... **SDPWS-2008 Table 4.2A**
- $\Delta_c = 0$  (Welded chord connections have no slip)
- $\delta_{dia} = 4.29 + 3.23 + 0 = 7.52$  in at midspan

This is slightly conservative since the unit shear at grid 10 is more than the unit shear at grid 1. Neither the IBC nor ASCE 7-10 contains prescriptive or other specific procedures to evaluate deformation compatibility, and much is left to the designer’s rational judgment. Remember that the mean recurrence interval (MRI) used is 700 years. Per ASCE 7-10 Appendix C Commentary, this is excessively conservative for serviceability and “selection of the MRI for serviceability evaluation is a matter of engineering judgement that should be exercised in consultation with the building client.”

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 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.

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.

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## Part 4 – Components and Cladding

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The edge wall components of the roof must be designed to effectively transfer loads into the wood diaphragm. Windward loads applied to the walls are pushed directly into the diaphragm at the ledger elevation. However, leeward loads must be pulled into the diaphragm. In a panelized roof, this is accomplished through the use of wall-ties. Along the north and south walls, wall-ties will consist of the steel joists spaced at 8 feet o.c. Along the east and west walls, embedded straps will transfer these loads into the diaphragm. All walls must be designed to span from the ground to the joist elevation vertically, and between the joists horizontally for leeward wind loads.

### 4a. Design Procedure

Per ASCE 7-10 Section 26.1.2.2 and 30.1.1: Part 1 is used for typical warehouse panelized roofs.

Per ASCE 7-10 Table 30.4-1 and design assumptions, the following can be calculated. Note Steps 1-5 are unchanged from the MWFRS: ..... **ASCE 7-10 Section 30.4**

Step 1. Risk Category II (given) ..... **ASCE 7-10 Table 1.5-1**

Step 2. Ultimate Design Wind Speed  $V_{ult} = 115$  MPH (given)..... **ASCE 7-10 Figure 26.5-1A**

Step 3. Wind Load Parameters:

- a. Wind Directionality Factor  $K_d = 0.85$   
per Table 26.6-1 (Buildings) ..... **ASCE 7-10 Section 26.6**
- b. Exposure Category = C (given)..... **ASCE 7-10 Section 26.7**
- c. Topographic Factor  $K_{zt} = 1.0$  (given) ..... **ASCE 7-10 Section 26.8**
- d. Enclosure Classification is Enclosed (given)..... **ASCE 7-10 Section 26.10**
- e. Internal pressure coefficients  $G_{C_{pi}} = \pm 0.18$   
per Table 26.11-1 ..... **ASCE 7-10 Section 26.11**



Step 4.  $K_h$  is determined using the mean roof height  $h$ . For roof slopes < 10 degrees, this is evaluated at the roof eave height. For N/S loads, this is 32 feet. For E/W loads, the roof "eave" (top of sheathing at perimeter walls) varies so the average of 34 feet is used:

a.  $K_h = 2.01 (z/z_g)^{(2/\alpha)}$  ..... **ASCE 7-10 Table 30.3-1**

i.  $z = h = 32$  ft (N/S) & 34 ft (E/W)

ii.  $\alpha$  is 9.5 &  $z_g$  is 900 ft for Exposure C ..... **ASCE 7-10 Table 26.9-1**

$K_h$  (N/S) = 1.00

$K_h$  (E/W) = 1.01 ..... **ASCE 7-10 Section 30.3.1**

Step 5. Velocity pressure at  $z = h$  is

$q_h = 0.00256 K_z K_{zt} K_d V^2$  ..... **ASCE 7-10 Eq. 30.3-1**

$q_h$  (N/S) = 28.78 psf

$q_h$  (E/W) = 29.07 psf ..... **ASCE 7-10 Section 30.3.2**

Step 6: External pressure coefficient  $GC_p$  is given in Figure 30.4-1 for walls. Tributary area must be calculated to determine  $GC_p$ . The area can be calculated based on 32 feet to the joist, 37 feet to top of wall, and connections at 8 feet o.c.:

$$\text{Trib Area} = \frac{(37 \text{ ft})^2}{32 \text{ ft}} \times 8 \text{ ft} = 171 \text{ ft}^2$$

Since we are only concerned with leeward wind, we need only find the negative  $GC_p$  values from ASCE 7-10 Figure 30.4-1. We'll find the positive pressure values when considering the parapet, below.

Surface 4: -0.90

Surface 5: -0.98

Note surface 5 occurs for a distance  $a$  from the building corners instead of the  $2a$  used for MWFRS End Zones.

$a = 10\%$  of shortest plan dimension or  $0.4h$  (whichever is smaller), but not less than 4% of shortest plan dimension or 3 ft.

$a = 12.8$  ft (N/S)

$a = 13.6$  ft (E/W)

Use  $a = 14$  ft, typical

ASCE 7-10 Figure 30.4-1 note 5 calls for values to be reduced 10% for walls when  $\theta \leq 10^\circ$ .

Surface 4:  $GC_p = (-0.90)(0.9) = -0.81$

Surface 5:  $GC_p = (-0.98)(0.9) = -0.88$  ..... **ASCE 7-10 Figure 30.4-1**

Step 7: Design wind load  $p = q_h [(GC_p) - (GC_{pi})]$  (lb/ft<sup>2</sup>)..... **ASCE 7-10 Eq. 30.4-1, Section 30.4.2**

N/S

$$p_{surface\ 4} = 28.78\ \text{psf} [(-.81) - (0.18)] = -28.49\ \text{psf}$$

$$p_{surface\ 5} = 28.78\ \text{psf} [(-.88) - (0.18)] = -30.51\ \text{psf}$$

E/W

$$p_{surface\ 4} = 29.07\ \text{psf} [(-.81) - (0.18)] = -28.78\ \text{psf}$$

$$p_{surface\ 5} = 29.07\ \text{psf} [(-.88) - (0.18)] = -30.81\ \text{psf}$$

Parapets: The loads above apply to walls below the eave. For components and cladding, parapets are covered in ASCE 7-10 Chapter 30, Part 6. .... **ASCE 7-10 Section 30.9**

$$p = q_p [(GC_p) - (GC_{pi})] \dots\dots\dots \text{ASCE 7-10 Eq. 30.9-1}$$

$q_p$  is evaluated at top of parapet (37 ft), so we must recalculate  $K_z$  (step 4)

$K_z = 1.03$ , which gives..... **ASCE 7-10 Table 30.3-1**

$$q_p = 29.64\ \text{psf} \text{ (Same as MWFRS parapet)}$$

Negative (leeward) values for  $GC_p$  are the same as step 6.

$$\text{Surface 4: } GC_p = -0.90(0.9) = -0.81$$

$$\text{Surface 5: } GC_p = -0.98(0.9) = -0.88 \dots\dots\dots \text{ASCE 7-10 Figure 30.4-1}$$

Positive (windward) values:

$$\text{Surface 4 \& 5: } GC_p = 0.79(0.9) = 0.71 \dots\dots\dots \text{ASCE 7-10 Figure 30.4-1}$$

Since our parapet is solid concrete, no internal pressure can develop. Therefore:

$$GC_{pi} = 0$$

See ASCE 7-10 Figure 30.9-1 for the two load cases to be considered: Case A where positive pressure is applied to the face of the wall and negative pressure on the back of the parapet, and Case B where negative pressure is applied to the face and positive pressure on the back of the parapet. Recall that positive pressures are pushed directly into the diaphragm so we only need to consider Case B loads.

$$p_{(leeward)} = q_p [(GC_{p3}) - (GC_{p4})] \dots\dots\dots \text{ASCE 7-10 Eq. 30.9-1}$$

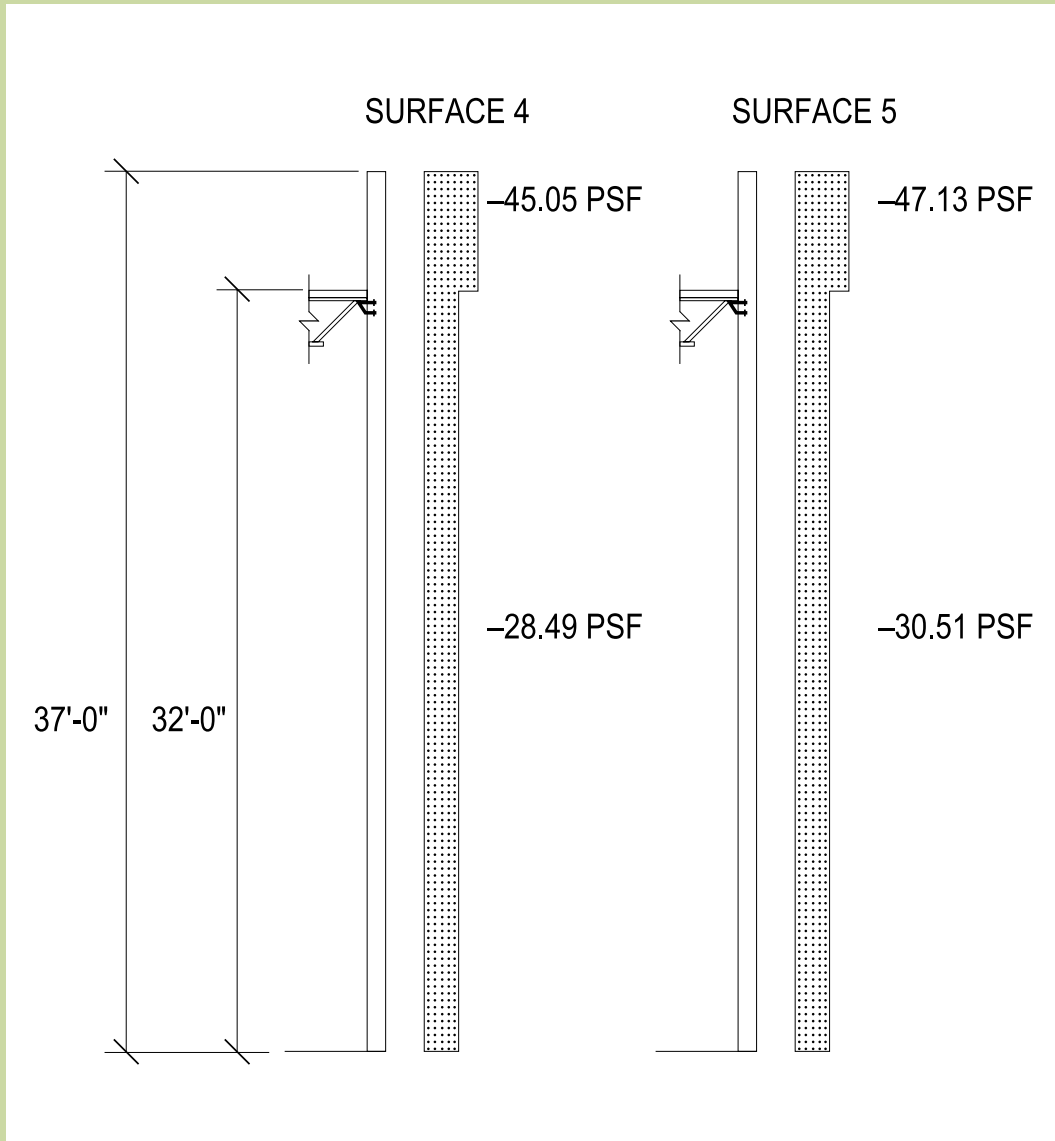
$$p_{(surface\ 4)} = 29.64\ \text{psf} [(0.71) - (-0.81)] = 45.05\ \text{psf}$$

$$p_{(surface\ 5)} = 29.64\ \text{psf} [(0.71) - (-0.88)] = 47.13\ \text{psf}$$

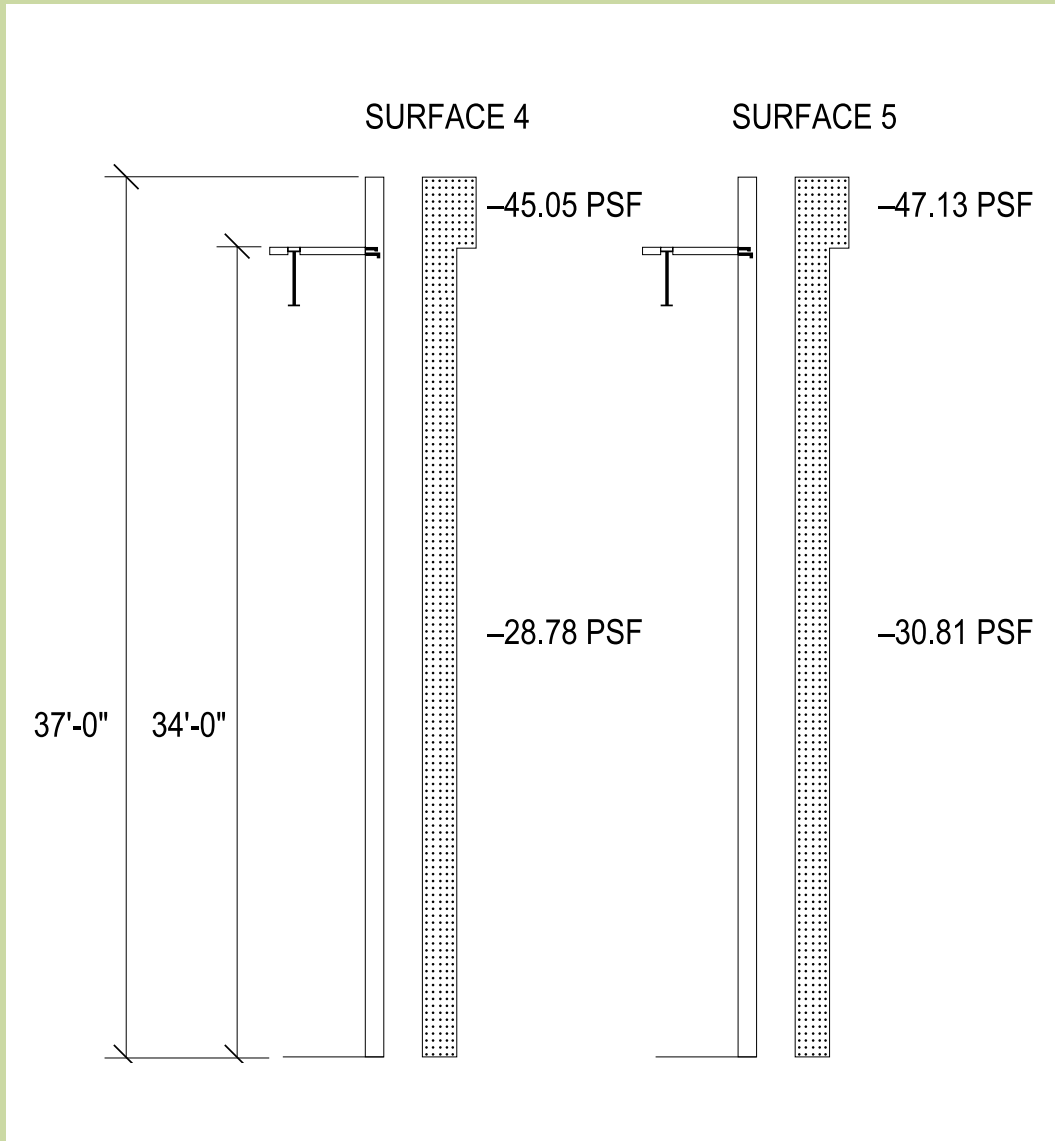




**Figure 12.** North/South Wall-Tie Loads



**Figure 13.** East/West Wall-Tie Loads



#### 4b. Wall-Tie Loads

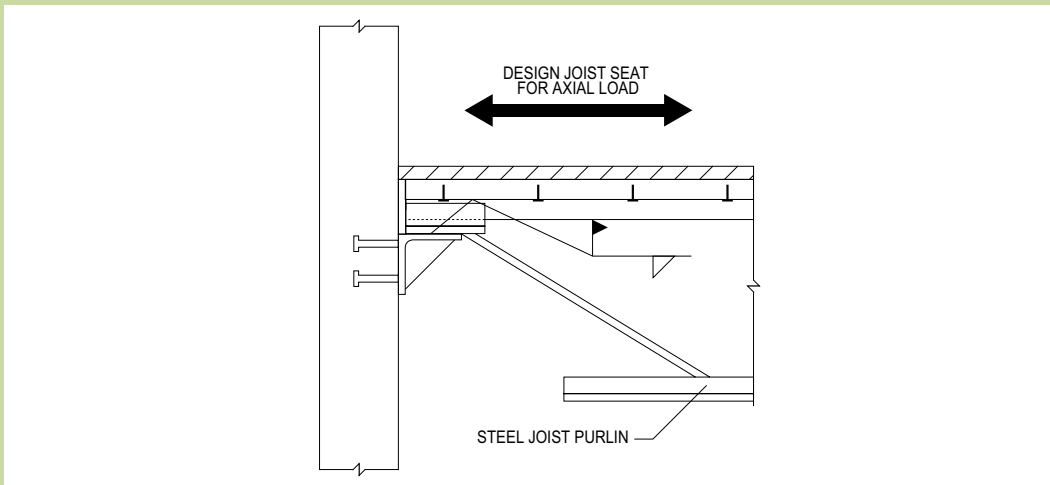
See Figures 12 and 13 for load distributions at surfaces 4 and 5 for N/S and E/W loads. Note that the wind pressures for E/W loads are slightly higher than those for N/S loads, but N/S loads will result in a higher total force to the wall-tie due to the larger distance between the roof and top of parapet. In reality, wall-ties occur the entire length of the east and west walls, having roof elevations from 32 feet to 36 feet. This does not seem conservative enough when designing the wall-ties that occur below the 34-foot elevation. Therefore, the low-eave loads (N/S in this example) will be used when calculating wall-tie loads.

The total force (T) can be calculated by summing the moments about the base of the wall and multiplying by the 8-foot purlin spacing:

$$T_{\text{surface 4}} = \frac{\left[ \frac{(28.49 \text{ psf}) \times (32\text{ft})^2}{2} + 45.05 \text{ psf} \times 5 \text{ ft} \times (32 \text{ ft} + 2.5 \text{ ft}) \right]}{32 \text{ ft}} \times 8 \text{ ft} = 5,590 \text{ lbs}$$

$$T_{\text{surface 5}} = 5,976 \text{ lbs}$$

**Figure 14.** Typical Steel Joist to Wall



N/S:

N/S loads are resisted by the steel joist purlins. 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. Buildings that are classified seismically as Seismic Design Category (SDC) C or higher will have continuity ties in conformance with ASCE 7 Section 12.11.2.2.1, and these may also be used to uniformly distribute the wind forces into the diaphragm. Buildings with SDC A or B should still have a rational load path for distributing wind forces into the diaphragm. In these roof systems, the load path of the wind load is through the joist seat, into the top chord, through the screws attaching the wood nailer to the top chord, and into the wood diaphragm sheathing through the diaphragm nails. 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 forces to the manufacturer. Given in Allowable Stress Design load they are:

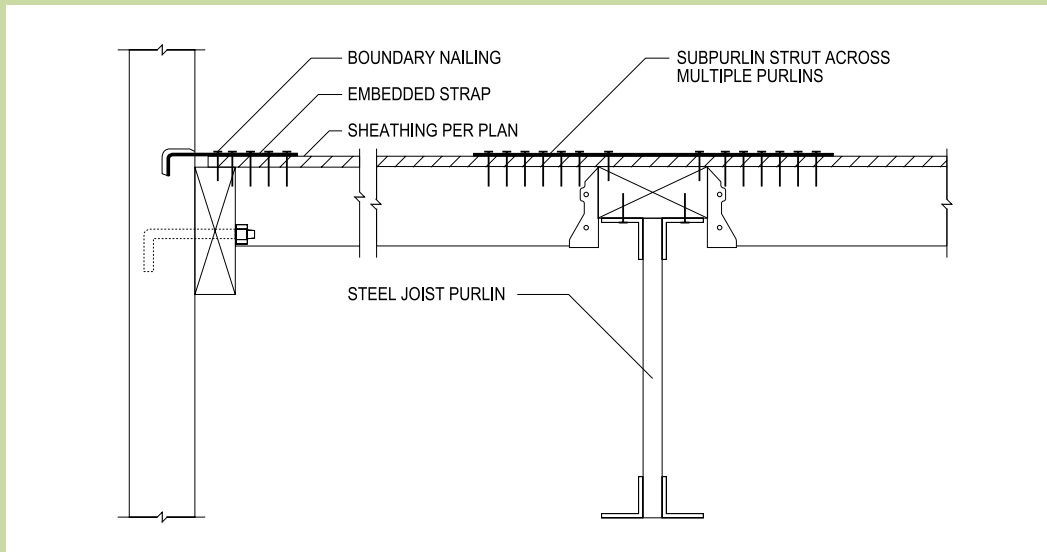
$$T_{surface\ 4} = 0.6 \times 5,590 \text{ lbs} = 3,354 \text{ lbs (ASD)}$$

$$T_{surface\ 5} = 0.6 \times 5,976 \text{ lbs} = 3,586 \text{ lbs (ASD)}$$

Joist Axial Wind Load = 3,354 lbs typical & 3,586 lbs for joists within 16 ft of building corners

In some regions, seismic forces may control the axial design of the continuity ties. The load combinations of ASCE 7-10 Sections 2.3.2 and 2.4.1 contain different load factors 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 an axial load  $W$  and an axial load  $E$  to the joist manufacturer.

**Figure 15.** Subpurlin at Wall-Tie



E/W:

E/W loads are resisted by embedded straps. 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 vary from 4 feet o.c. (every other subpurlin) to 8 feet o.c. (every fourth subpurlin) depending on the magnitude of the load. Recall that an 8-foot width was used for calculating tributary area needed for  $GC_p$  (step 6). If the spacing of embedded straps is 4 feet or 6 feet o.c.,  $GC_p$  must be recalculated, resulting in a higher wind pressure. Prefabricated embedded steel hardware with nails to the subpurlin is commonly used for wall-ties (see figure 15). Allowable load information can be obtained by consulting the hardware manufacturer's Evaluation Report.

In this example, a 4x4 subpurlin strut using DF/L No. 2 will replace the standard 2x4 subpurlin at the anchorage locations. The strap anchor is located above the wood structural panels and nailed to the top of the 4x4 subpurlin. This configuration will result in an eccentric load to the subpurlin, but this will be discussed later.



The 4x4 subpurlin sees a combination of wall anchorage tension from wind and gravity bending from the roof dead and live loads. These three loads are combined using the load combinations of IBC Section 1605.3.1, load combinations for allowable stress design. The maximum effect on the subpurlin is achieved with Equation 16-13.

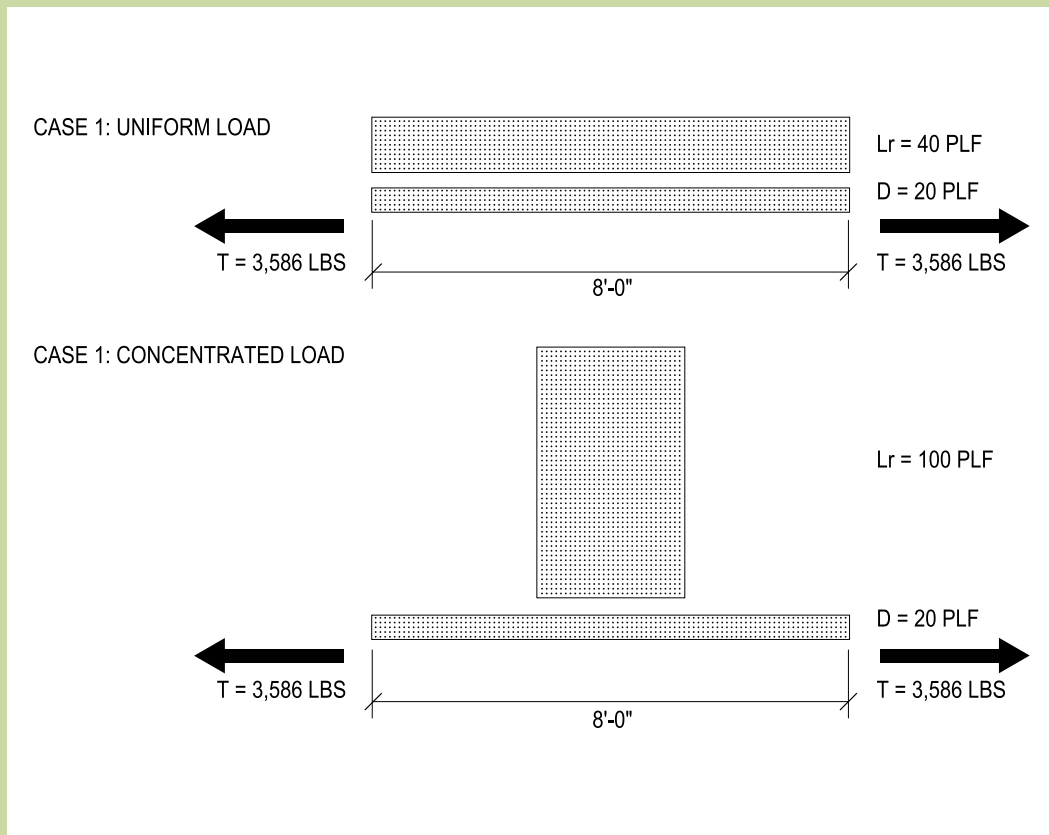
$$D + H + F + 0.75(0.6W) + 0.75L + 0.75(Lr \text{ or } S \text{ or } R) \dots \text{IBC Eq. 16-13}$$

Removing the non-applicable terms, this becomes:

$$D + 0.75(0.6W) + 0.75Lr$$

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 subpurlin load is estimated. The 2-foot subpurlin spacing thus results in a 20 plf applied dead load. Roof live load can be found in IBC table 1607.1, item 26. It is 20 psf uniform load or 300 lbs concentrated load over a 2.5-foot x 2.5-foot area, per IBC 1607.4. The 300 lbs concentrated load becomes 48 psf over the 2.5-foot x 2.5-foot area. Centering that on the 4x4 wall-tie member, we distribute to the subpurlins on either side resulting in a 100 plf load over 2.5 feet. The following loading diagram illustrates the subpurlin load combinations.

**Figure 16.** Subpurlin Loading Diagram



The actual tensile stress  $f_t$  (ASD) and allowable tensile stress  $F'_t$  are determined and compared:

$$f_t = \frac{P}{A} = \frac{0.75 \times 3,586 \text{ lbs}}{12.25 \text{ in}^2} = 220 \text{ psi}$$

$$F'_t = F_t C_D C_M C_t C_F C_j \dots \dots \dots \text{NDS Table 4.3.1}$$

where:

$$F_t = 575 \text{ psi (DF/L No. 2)} \dots \dots \dots \text{NDS Supplement Table 4A}$$

$$C_D = 1.6 \dots \dots \dots \text{NDS Table 2.3.2}$$

$$C_F = 1.5 \dots \dots \dots \text{NDS Supplement Table 4A}$$

$$C_M \text{ \& } C_t \text{ \& } C_j = 1.0$$

$$F'_t = 575 \text{ psi (1.6)(1.5)} = 1,380 \text{ psi}$$

$$\frac{f_t}{F'_t} = \frac{220 \text{ psi}}{1,380 \text{ psi}} = 0.16 \leq 1.0$$

Thus, tensile stress okay.

The actual bending stress  $f_b$  (ASD) and allowable bending stress  $F'_b$  are determined and compared:

$$M_D = \frac{wL^2}{8} = \frac{20 \text{ plf (8 ft)}^2(12 \text{ in})}{8} = 2,198 \text{ lbs-in dead load}$$

$$0.75M_{L1} = 0.75 \frac{wL^2}{8} = 0.75 \frac{40 \text{ plf (8 ft)}^2(12 \text{ in})}{8} = 3,297 \text{ lbs-in uniform live load}$$

$$0.75M_{L2} = 0.75 \left( \frac{wb}{2} \right) \left( a + \frac{b}{2} \right) = 0.75 \left( \frac{100 \text{ plf (2.5 ft)}}{2} \right) \left( 2.75 \text{ ft} + \frac{2.5 \text{ ft}}{2} \right) (12 \text{ in}) = 4,500 \text{ lbs-in concentrated live load}$$

The concentrated load case governs the live load.

$$f_b = \frac{M}{S} = \frac{2,198 \text{ lbs-in} + 4,500 \text{ lbs-in}}{7.146 \text{ in}^3} = 937 \text{ psi}$$

$$F'_b = F_b C_D C_M C_t C_L C_F C_{fu} C_i C_r \dots \dots \dots \text{NDS Table 4.3.1}$$

where:

$$F_b = 900 \text{ psi (DF/L No. 2)} \dots \dots \dots \text{NDS Supplement Table 4A}$$

$$C_D = 1.6 \dots \dots \dots \text{NDS Table 2.3.2}$$

$$C_F = 1.5 \dots \dots \dots \text{NDS Supplement Table 4A}$$

$$C_r = 1.15 \dots \dots \dots \text{NDS Section 4.3.9}$$

$$C_M \text{ \& } C_t \text{ \& } C_i \text{ \& } C_L \text{ \& } C_{fu} = 1.0$$

$$F'_b = 900 \text{ psi (1.6)(1.5)(1.15)} = 2,484 \text{ psi}$$

$$\frac{f_b}{F'_b} = \frac{937 \text{ psi}}{2,484 \text{ psi}} = 0.38 \leq 1.0$$

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

$$\frac{f_t}{F'_t} + \frac{f_b}{F'_b} = 0.16 + 0.38 = 0.54 \leq 1.0 \dots \dots \dots \text{NDS Section 3.9.1}$$



Thus the combined stress check is acceptable, and the 4x4 DF/L No. 2 subpurlin strut is adequate for the wall-tie load. Note that the eccentricity caused by the anchor strap attaching to the top of the 4x4 creates a moment that counteracts gravity load stresses, therefore it is conservative to ignore it. If the strap is attached to one side of the subpurlin, it can potentially subject the member to weak axis bending and must be checked.

Use 4x4 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 deformation 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 elongation; however, that is only one component to consider. The type of connectors, length of anchor rods and installation practices (oversized bolt holes) can be significant sources of additional deformation or stretch.

#### **4c. Continuity Ties and Subdiaphragms**

In order for the wall anchorage forces to be adequately distributed into the roof diaphragm, continuity ties are provided as a force distributing strut from chord to chord. The continuity ties take the wall anchorage force and distribute the load uniformly across the diaphragm depth. Buildings that are classified seismically as SDC C or higher must have continuity ties in conformance with ASCE 7 Section 12.11.2.2.1, and these may also provide a load path to uniformly distribute the wind forces into the diaphragm. Buildings with SDC A or B should still have a rational load path for distributing wind forces into the diaphragm, and the use of continuity ties is recommended.

N/S: 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 connecting welds.

E/W: At first appearance, one solution to uniformly distribute the wall-tie force into the diaphragm would be to install ties across the diaphragm to line up with every wall anchorage as we did in the N/S direction, but this would require splices at each perpendicular purlin across the entire length of the building, resulting in an excessive number of continuous ties and their splice connections. 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 analytic 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. The concept of subdiaphragms is recognized in ASCE 7 Section 12.11.2.2.1 for seismic wall-tie forces, but is equally suited for wind loads.

To prevent subdiaphragms from becoming too flexible compared to the rest of the main diaphragm, ASCE 7-10 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 deflection between the subdiaphragm and the main diaphragm may be ignored.

In this example, it is desirable to use the E/W girder lines as the continuous ties for distributing the wall anchorage force into the main diaphragm. Given the 50-foot spacing between the continuous ties (the subdiaphragm length), the minimum subdiaphragm width is 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 often controlled by the available subdiaphragm's shear capacity. The necessary subdiaphragm width for the shear demand will now be investigated.

Because the subdiaphragm, continuous ties, and their connections are a part of the components and cladding system, the wall anchorage forces found in Part 4a are used for their design. However, the subdiaphragm tributary area is larger, affecting  $G_{C_p}$ . Tributary Area for the subdiaphragm is:

$$\text{Trib Area} = \frac{\frac{(37 \text{ ft})^2}{2}}{32 \text{ ft}} \times 50 \text{ ft} = 1,069.5 \text{ ft}^2$$

Surface 4 & 5:  $G_{C_p} = -0.8(0.9) = -0.72$  ..... **ASCE 7-10 Figure 30.4-1**

Design wind load  $p = q_h [(G_{C_p}) - (G_{C_{pi}})]$  ..... **ASCE 7-10 Eq. 30.4-1, Section 30.4.2**

$$p_{\text{subdiaphragm}} = 28.78 \text{ psf} [(-.72) - (0.18)] = 25.90 \text{ psf}$$

Parapet:

Positive (windward) values:

Surface 4 & 5:  $G_{C_p} = 0.7(0.9) = 0.63$  ..... **ASCE 7-10 Figure 30.4-1**

$p_{\text{(leeward)}} = q_p [(G_{C_{p3}}) - (G_{C_{p4}})]$  ..... **ASCE 7-10 Figure 30.9-1**

$$P_{\text{(subdiaphragm)}} = 29.64 \text{ psf} [(0.63) - (-0.72)] = 40.01 \text{ psf}$$

Uniform load to the subdiaphragm:

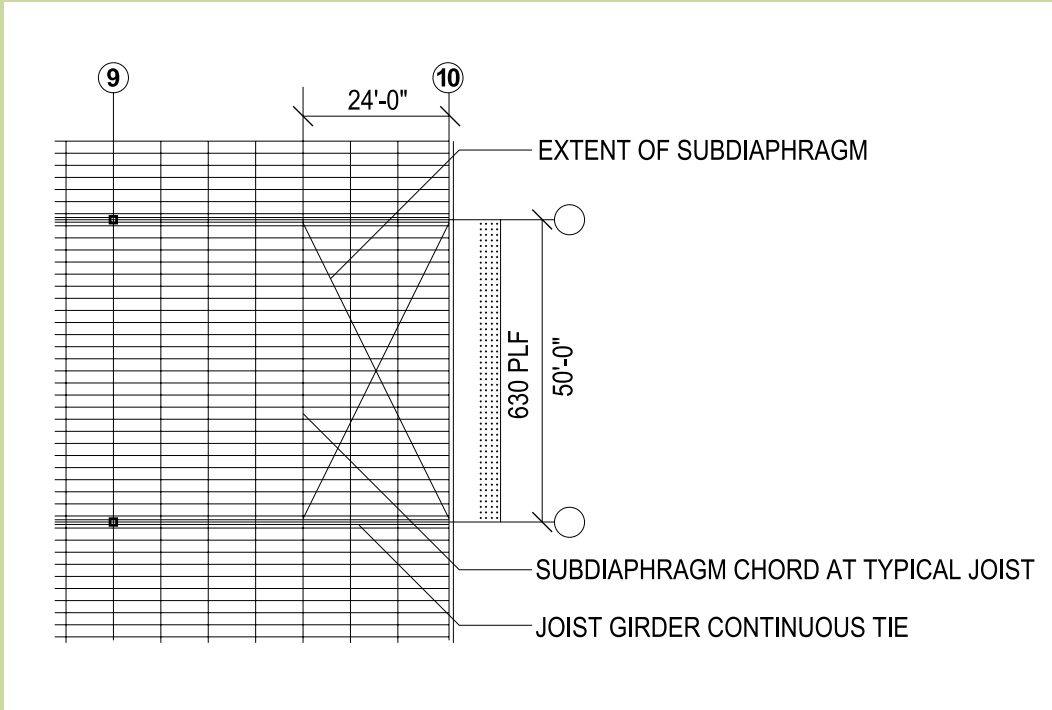
$$W_{\text{subdiaphragm}} = \frac{\left[ \frac{(25.90 \text{ psf}) \times (32 \text{ ft})^2}{2} + 40.01 \text{ psf} \times 5 \text{ ft} \times (32 \text{ ft} + 2.5 \text{ ft}) \right]}{32 \text{ ft}} = 630 \text{ plf}$$





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 beam with uniformly distributed load.

**Figure 17.** Typical Subdiaphragm



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

$$\text{Subdiaphragm Reaction} = 630 \text{ plf} \frac{50 \text{ ft}}{2} = 15,750 \text{ lbs}$$

The diaphragm has a diaphragm shear capacity of 447.5 plf (ASD) as shown in Part 2c. The minimum subdiaphragm width based on shear demand is as follows:

$$\text{Minimum Subdiaphragm Width} = \frac{0.6(15,750 \text{ lbs})}{447.5 \text{ plf}} = 21.1 \text{ ft} < 24 \text{ ft} \quad \text{Okay}$$

If the subdiaphragm width were to be governed by shear stress instead of the aspect ratio limit, it would simply need to be increased to the next purlin joist on the 8-foot module (i.e., 32-foot or 40-foot subdiaphragm width).

#### 4d. Subpurlin Tie Connections

As a practical matter, the wall anchorage at 8-foot spacing needs continuous ties only 24 feet deep into the diaphragm (the full subdiaphragm width). With the steel joists spaced at 8 feet apart, the continuous ties will be broken at one-third points. See Figure 17 for the subdiaphragm configuration with the applied wall anchorage load of 630 plf from Part 4c.

Since the subdiaphragm depth is 24 feet (3 purlin bays), two-thirds of the wall anchorage load will be carried by the connection across the first purlin from the wall, and one-third of the load across the second purlin.

This 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 15). 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.

The tributary area of the component we are designing (subpurlin strut) is the same as the wall-tie embedded strap in Section 4b. We will consider only the End Zone loads here in an effort to simplify the number of connections specified.

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

$$F_p = 5,976 \text{ lbs (End Zone)}$$

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

$$F_{p1} = 2/3 \times (5,976 \text{ lbs}) = 3,984 \text{ lbs}$$

$$F_{p1} = 0.6 (3,984 \text{ lbs}) = 2,390 \text{ lbs (ASD)}$$

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

$$F_{p2} = 1/3 \times (5,976 \text{ lbs}) = 1,992 \text{ lbs}$$

$$F_{p2} = 0.6 (1,992 \text{ lbs}) = 1,195 \text{ lbs (ASD)}$$

In the selection of pre-engineered, prefabricated steel strap assemblies, an allowance must be made for the width of the perpendicular purlin that separates the subpurlin struts. In most cases, 5 inches is adequate. If in doubt, consult your joist supplier for a preliminary width of the joists. Also take note of any special requirements the strap manufacturer has relative to the subpurlin size. Some straps are designed to be nailed into the edge of a 2x member, with a nailing pattern that reduces the potential for splitting. Others require a wider (4x) width member.

The design of the second and third subpurlin struts is similar to Part 4b.



#### 4e. Subdiaphragm Chord Check

The subdiaphragm depth of 24 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 2e).

Determine the chord force in the joist and compare with the axial force found in Part 4b associated with the N/S wall anchorage. Because the two joist forces are from different orthogonal directions, they do not need to be combined.

Subdiaphragm chord force from E/W wind wall anchorage forces:

$$\text{Chord Force in Joist} = \frac{630 \text{ plf} (50 \text{ ft})^2}{8 (24 \text{ ft})} = 8,203 \text{ lbs}$$

Continuous tie force from N/S wall anchorage forces:

$$\text{Continuous Tie Force in Joist} = 5,976 \text{ lbs} \dots\dots\dots \text{Part 4b}$$

The subdiaphragm chord force induced into the joist is greater than the previously determined axial force for the N/S 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 24 feet to 40 feet for the purpose of reducing the chord force below the N/S 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 their joist for axial loads in both tension and compression.

#### 4f. 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 out-of-plane wall anchorage loads into the main diaphragm uniformly. The magnitude of this continuous tie force is simply the sum of the reactions from the two adjacent subdiaphragms.

As found in Part 4c, the subdiaphragm reaction is 15,750 lbs, and the main diaphragm's continuous ties resists two subdiaphragm reactions.

$$\text{Continuous Tie Force in Girder Line} = 15,750 \text{ lbs} \times 2 = 31.5 \text{ kips}$$

$$\text{Continuous Tie Force in Girder Line} = 0.6(31.5 \text{ kips}) = 18.9 \text{ kips (ASD)}$$

The joist girder's 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 18.9 kip axial continuous tie force is placed on the construction drawings with the indication that this is an ASD Wind load *W*. In situations where the seismic force *E* may 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 IBC Section 1605.2 (strength design) and 1605.3.1 (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.

# Part 5 – Uplift

Roof members are subject to uplift forces as shown in ASCE 7-10 Figure 30.4-2A. The sheathing and subpurlins are not considered as individual members, but rather as an assembly. For a thorough discussion of these assemblies, see APA publication G310, *Wind-Rated Roofs: Designing Commercial Roofs to Withstand Wind Uplift Forces*.

The purlins and girders will need to be designed for the uplift forces. Purlin tributary area is 8 ft x 50 ft = 400 ft<sup>2</sup>.

- Surface 1:  $G_{C_p} = -0.90$  ..... **ASCE 7-10 Figure 30.4-2A**
- Surface 2 & 3:  $G_{C_p} = -1.1$  ..... **ASCE 7-10 Figure 30.4-2A**
- Design wind load  $p = q_h[(G_{C_p}) - (G_{C_{pi}})]$  (lb/ft<sup>2</sup>) ..... **ASCE 7-10 Eq. 30.4-1, Section 30.4.2**

Surface 1:  $p_{uplift} = 28.78 \text{ psf} [(-.90) - (0.18)] = -31.08 \text{ psf}$

Surface 2 & 3:  $p_{uplift} = 28.78 \text{ psf} [(-1.1) - (0.18)] = -36.84 \text{ psf}$

This is the gross uplift on the purlins. IBC Equation 16-15 can be used to calculate the net uplift.

$p = 0.6D + 0.6W + H$  ..... **IBC 16-15**

Recall that our total dead load is 15 psf. This includes 1.5 psf for joist girders, so our dead load to the purlins is 13.5 psf.

Surface 1:  $p_{net} = 0.6(13.5 \text{ psf}) + 0.6(-31.08 \text{ psf}) = -10.55 \text{ psf}$

Surface 2 & 3:  $p_{net} = 0.6(13.5 \text{ psf}) + 0.6(-36.84 \text{ psf}) = -14.00 \text{ psf}$

This information is placed on the construction drawings with the dimensions indicating the extent of surfaces 2 and 3 (within 14 feet of walls).

Girder tributary area is 50 ft x 50 ft = 2,500 ft<sup>2</sup>. Per ASCE 7-10 Section 30.2.3, we will design using the provisions of the MWFRS. From ASCE 7-10 Figure 28.4-1 we find values for  $G_{C_{pf}}$ . We are concerned with surfaces 2, 3, 2E, and 3E. Since our building is symmetrical, we need only consider the greater values between surfaces 2 and 3, and between surfaces 2E and 3E:

- Surface 2:  $G_{C_{pf}} = -0.69$  ..... **ASCE 7-10 Figure 28.4-1**
- Surface 2E:  $G_{C_p} = -1.07$  ..... **ASCE 7-10 Figure 28.4-1**
- Design wind load  $p = q_h[(G_{C_{pf}}) - (G_{C_{pi}})]$  ..... **ASCE 7-10 Eq. 28.4-1, Section 28.4.1**

Surface 2:  $p_{uplift} = 29.07 \text{ psf} [(-.69) - (0.18)] = -25.29 \text{ psf}$

Surface 2E:  $p_{uplift} = 29.07 \text{ psf} [(-1.07) - (0.18)] = -36.34 \text{ psf}$

The full 15 psf roof dead load is utilized for the girder dead load.

$p_{net} = 0.6D + 0.6W + H$  ..... **IBC 16-15**

Surface 2:  $p_{net} = 0.6(15 \text{ psf}) + 0.6(-25.29 \text{ psf}) = -6.17 \text{ psf}$

Surface 2E:  $p_{net} = 0.6(15 \text{ psf}) + 0.6(-36.34 \text{ psf}) = -12.80 \text{ psf}$

Note that the extent of surface 2E is 2a, or 28 feet from walls per ASCE 7-10 Figure 28.4-1. As with joist uplift, this information is placed on the construction drawings.



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## Part 6 – Other Issues Of Importance

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### 6a. 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 3⅞-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.

### 6b. 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.

## 6c. 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.

## 6d. 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 the 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<sup>2</sup>). 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<sup>2</sup>).

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.

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# Part 7 – Closure

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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. 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.







*WoodWorks is an initiative of the Wood Products Council.*