Diaphragms and Shear Walls

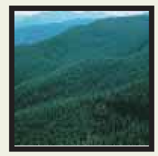
DESIGN/CONSTRUCTION GUIDE

APA
THE ENGINEERED WOOD ASSOCIATION

WOOD
The Natural Choice

Engineered wood products are a good choice for the environment. They are manufactured for years of trouble-free, dependable use. They help reduce waste by decreasing disposal costs and product damage. Wood is a renewable, recyclable, biodegradable resource that is easily manufactured into a variety of viable products.

A few facts about wood.

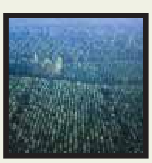


- ***We're growing more wood every day.*** Forests fully cover one-third of the United States' and one-half of Canada's land mass. American landowners plant more than two billion trees every year. In addition, millions of trees seed naturally. The forest products industry, which comprises about 15 percent of forestland ownership, is responsible for 41 percent of replanted forest acreage. That works out to more than one billion trees a year, or about three million trees planted every day. This high rate of replanting accounts for the fact that each year, 27 percent more timber is grown than is harvested. Canada's replanting record shows a fourfold increase in the number of trees planted between 1975 and 1990.

- ***Life Cycle Assessment shows wood is the greenest building product.*** A 2004 Consortium for Research on Renewable Industrial Materials (CORRIM) study gave scientific validation to the strength of wood as a green building product. In examining building products' life cycles – from extraction of the raw material to demolition of the building at the end of its long lifespan – CORRIM found that wood was better for the environment than steel or concrete in terms of embodied energy, global warming potential, air emissions, water emissions and solid waste production. For the complete details of the report, visit www.CORRIM.org.

- ***Manufacturing wood is energy efficient.*** Wood products made up 47 percent of all industrial raw materials manufactured in the United States, yet consumed only 4 percent of the energy needed to manufacture all industrial raw materials, according to a 1987 study.

Material	Percent of Production	Percent of Energy Use
Wood	47	4
Steel	23	48
Aluminum	2	8



- ***Good news for a healthy planet.*** For every ton of wood grown, a young forest produces 1.07 tons of oxygen and absorbs 1.47 tons of carbon dioxide.

Wood: It's the natural choice for the environment, for design and for strong, lasting construction.

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RATED SHEATHING
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000
PS 1-07 C-D PRP-108

NOTICE:
The recommendations in this guide apply only to products that bear the APA trademark. Only products bearing the APA trademark are subject to the Association's quality auditing program.

When designing a building for lateral loads such as those generated by wind or earthquakes, a design engineer may have several alternatives. Lateral loads may be transferred to the foundation via braced frames or rigid frames, diagonal rods or "x" bracing, including let-in bracing in the case of wood frame construction, or other methods. Where wood structural panels are used for the roof, floors, or walls in a building, lateral loads can be accommodated through the use of these ordinary vertical load bearing elements. This type of construction is easily adaptable to conventional light frame construction typically used in residences, apartment buildings and offices. The same concept is equally adaptable to larger warehouses and similar industrial or commercial buildings.

Buildings can be designed to resist the horizontal loads introduced by the most violent wind or earthquake through the application of a principle called "diaphragm design."

This guide from APA – The Engineered Wood Association defines diaphragms and shear walls and gives examples of how they can be incorporated into building design.

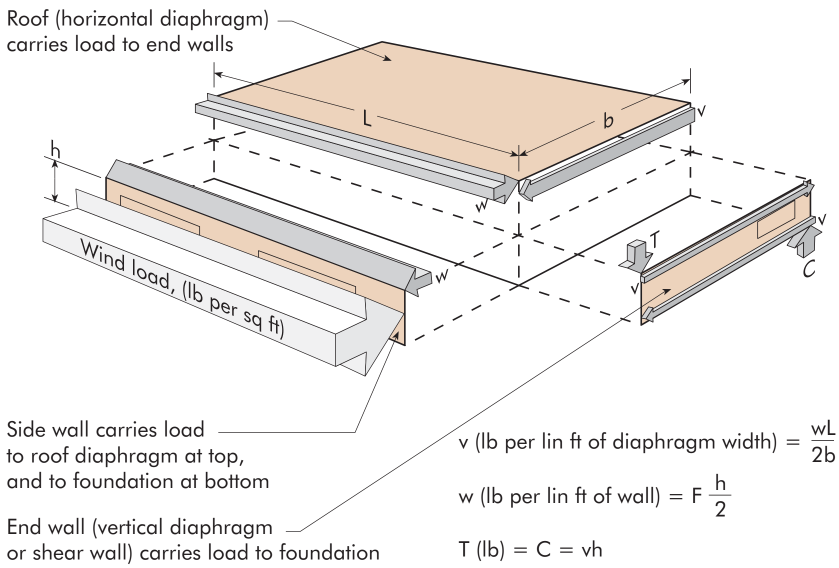
CONTENTS

DIAPHRAGMS AND SHEAR WALLS DEFINED

A diaphragm is a flat structural unit acting like a deep, thin beam. The term "diaphragm" is usually applied to roofs and floors. A shear wall, however, is a vertical, cantilevered diaphragm. A diaphragm structure results when a series of such vertical and horizontal diaphragms are properly tied together to form a structural unit. (See Figure 1.) When diaphragms and shear walls are used in the lateral design of a building, the structural system is termed a "box system." Shear walls provide reactions for the roof and floor diaphragms, and transmit the forces into the foundation.

FIGURE 1

DISTRIBUTION OF LATERAL LOADS ON BUILDING



$$v \text{ (lb per lin ft of diaphragm width)} = \frac{wL}{2b}$$

$$w \text{ (lb per lin ft of wall)} = F\frac{h}{2}$$

$$T \text{ (lb)} = C = vh$$

An accurate method for engineering diaphragms has evolved from analytic models and extensive testing, and will allow the engineer to supply his client with a building resistant to hurricanes or earthquakes at very little extra cost.

The structural design of buildings using diaphragms is a relatively simple, straightforward process if the engineer keeps in mind the over-all concept of structural diaphragm behavior. Actually, with ordinary good construction practice, any sheathed element in a building adds considerable strength to the structure. Thus, if the walls and roofs are sheathed with panels and are adequately tied together, and to the foundation, many of the requirements of a diaphragm structure are met. This fact explains the durability of panel-sheathed buildings in hurricane and earthquake conditions even when they have not been engineered as diaphragms. For full diaphragm design, it is necessary to also analyze chord stresses, connections and tie downs.

Panel diaphragms have been used extensively for roofs, walls, floors and partitions, for both new construction and rehabilitation of older buildings.

A diaphragm acts in a manner analogous to a deep beam or girder, where the panels act as a "web," resisting shear, while the diaphragm edge members perform the function of "flanges," resisting bending stresses. These edge members are commonly called chords in diaphragm design, and may be joists, ledgers, trusses, bond beams, studs, top plates, etc.

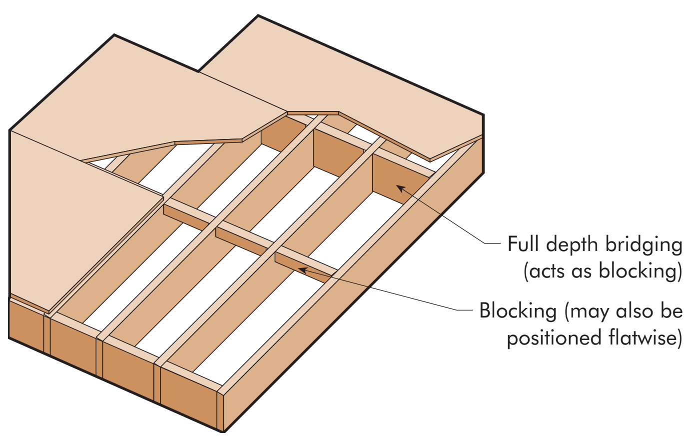
A shear wall is simply a cantilevered diaphragm to which load is applied at the top of the wall, and is transmitted out along the bottom of the wall. This creates a potential for overturning which must be accounted for, and any overturning force is typically resisted by hold-downs or tie-downs, at each end of the shear element.

Due to the great depth of most diaphragms and small span-to-depth ratios in the direction parallel to application of load, and to their means of assembly, their behavior differs slightly from that of the usual, relatively shallow, beam. Shear stresses have been proven essentially uniform across the depth of the diaphragm, rather than showing significant parabolic distribution as in the web of a beam. Similarly, chords in a diaphragm carry all "flange" stresses acting in a simple tension and compression, rather than sharing these stresses significantly with the web. As in any beam, consideration must be given to bearing stiffeners, continuity of webs and chords, and to web buckling, which is normally resisted by the framing members.

Diaphragms vary considerably in load-carrying capacity, depending on whether they are "blocked" or "unblocked." Blocking consists of lightweight nailers, usually 2x4s, framed between the joists or other primary structural supports for the specific purpose of connecting the edges of the panels. (See Figure 2.) Systems which provide support framing at all panel edges, such as panelized roofs, are also considered blocked. The reason for blocking in diaphragms is to allow connection of panels at all edges for better shear transfer. Another form of blocking for purposes of shear transfer is with a common piece of sheet metal stapled to adjacent panels to provide shear transfer between panels (see APA Technical Note: *Stapled Sheet Metal Blocking for APA Panel Diaphragms,* Form N370). Unblocked diaphragm loads are controlled by buckling of unsupported panel edges, with the result that such units reach a maximum load above which increased nailing will not increase capacity. For the same nail spacing, design loads on a blocked diaphragm are from 1-1/2 to 2 times design loads of its unblocked counterpart. In addition, the maximum loads for which a blocked diaphragm can be designed are many times greater than those for diaphragms without blocking.

FIGURE 2

BLOCKING



The three major parts of a diaphragm are the web, the chords, and the connections. Since the individual pieces of the web must be connected to form a unit; since the chord members in all probability are not single pieces; since web and chords must be held so that they act together; and since the loads must have a path to other elements or to the foundation, connections are critical to good diaphragm action. Their choice actually becomes a major part of the design procedure.

ADVANTAGES OF DIAPHRAGM DESIGN

Structural panel diaphragms take advantage of the capacity of wood to absorb impact loads. They maintain high strength in the design range and, if pushed to their ultimate capacity, yield gradually while continuing to carry load. In terms of engineering dynamics, they give high values of "work to ultimate" (will absorb a great deal of energy before failure). This action is illustrated by Figure 3, a load deformation test curve of a shear wall.

By considering the strength and stiffness of the skin of a building, the engineer can eliminate almost all of the expensive and inefficient diagonal bracing which might otherwise be required.

Diaphragms are easy to build and to connect to other portions of the structure. Primary components of the system are commercially available structural panels, structural lumber, nails and metal connectors.

Panel diaphragm design has been proven through some of the most harrowing hurricane and earthquake experiences imaginable.

And finally, diaphragm design enables the engineer to produce a building designed to resist high wind and seismic loads for little or no extra cost.

FIGURE 3

SHEAR WALL LOAD-DEFLECTION CURVE SUBJECTED TO CUREE CYCLIC LOADING

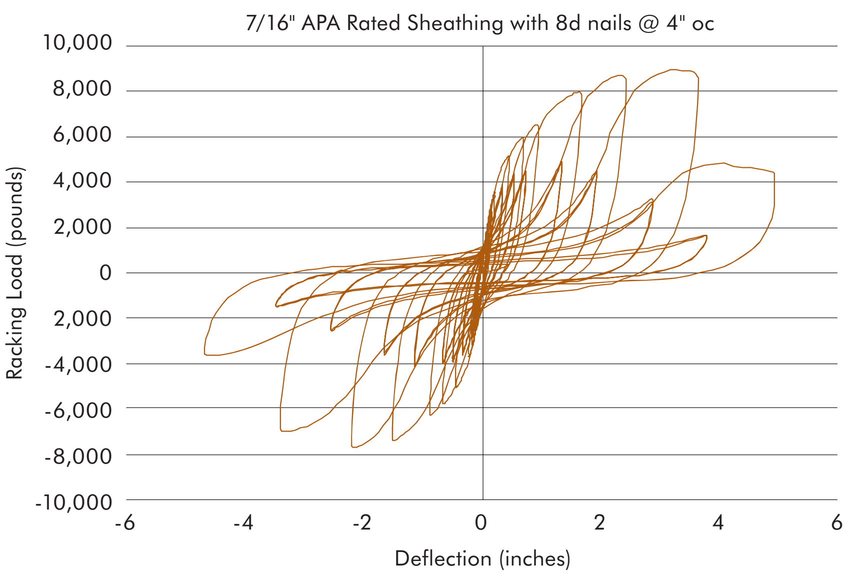


TABLE 1

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR APA PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR, LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b,h,i,j,k] (See also IBC Table 2306.4.1)

			Panels Applied Direct to Framing					Panels Applied Over 1/2" or 5/8" Gypsum Sheathing				
				Nail Spacing at Panel Edges (in.)					Nail Spacing at Panel Edges (in.)			
Panel Grade	**Minimum Nominal Panel Thickness (in.)**	**Minimum Nail Penetration in Framing (in.)**	**Nail Size (common or galvanized box)[k]**	**6**	**4**	**3**	**2[e]**	**Nail Size (common or galvanized box)**	**6**	**4**	**3**	**2[e]**
APA STRUCTURAL I grades	5/16	1-1/4	6d (0.113" dia.)	200	300	390	510	8d (0.131" dia.)	200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	230[d]	360[d]	460[d]	610[d]	10d (0.148" dia.)	280	430	550[f]	730
	7/16			255[d]	395[d]	505[d]	670[d]					
	15/32			280	430	550	730					
	15/32	1-1/2	10d (0.148" dia.)	340	510	665[f]	870		—	—	—	—
APA RATED SHEATHING; APA RATED SIDING[g] and other APA grades except Species Group 5	5/16 or 1/4[c]	1-1/4	6d (0.113" dia.)	180	270	350	450	8d (0.131 dia.)	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1-3/8	8d (0.131" dia.)	220[d]	320[d]	410[d]	530[d]	10d (0.148" dia.)	260	380	490[f]	640
	7/16			240[d]	350[d]	450[d]	585[d]					
	15/32			260	380	490	640					
	15/32	1-1/2	10d (0.148" dia.)	310	460	600[f]	770	—	—	—	—	—
	19/32			340	510	665[f]	870	—	—	—	—	—
APA RATED SIDING[g] and other APA grades except Species Group 5			**Nail Size (galvanized casing)**					**Nail Size (galvanized casing)**				
	5/16[c]	1-1/4	6d (0.113" dia.)	140	210	275	360	8d (0.131" dia.)	140	210	275	360
	3/8	1-3/8	8d (0.131" dia.)	160	240	310	410	10d (0.148" dia.)	160	240	310[f]	410

(a) For framing of other species: Find specific gravity for species of lumber in the AF&PA National Design Specification (NDS). Find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment shall not be greater than 1.

(b) Panel edges backed with 2 inch nominal or wider framing. Install panels either horizontally or vertically. Space fasteners maximum 6 inches on center along intermediate framing members for 3/8 inch and 7/16 inch panels installed on studs spaced 24 inches on center. For other conditions and panel thicknesses, space nails maximum 12 inches on center on intermediate supports.

(c) 3/8 inch panel thickness or siding with a span rating of 16 inches on center is the minimum recommended where applied direct to framing as exterior siding.

(d) Allowable shear values are permitted to be increased to values shown for 15/32 inch sheathing with same nailing provided (1) studs are spaced a maximum of 16 inch on center, or (2) panels are applied with long dimension across studs.

(e) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where nails are spaced 2 inch on center.

(f) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where both the following conditions are met: (1) 10d (3 inch x 0.148 inch) nails having penetration into framing of more than 1-1/2 inch and (2) nails are spaced 3 inch on center.

(g) Values apply to all-veneer plywood. Thickness at point of fastening on panel edges governs shear values.

(h) Where panels applied on both faces of a wall and nail spacing is less than 6 inches o.c. on either side, panel joints shall be offset to fall on different framing members, or framing shall be 3 inch nominal or thicker at adjoining panel edges and nails on each side shall be staggered.

(i) In Seismic Design Category D, E or F, where shear design values exceed 350 pounds per lineal foot, all framing members receiving edge nailing from abutting panels shall not be less than a single 3 inch nominal member, or two 2 inch nominal members fastened together in accordance with IBC Section 2306.1 to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered in all cases. See IBC Section 2305.3.11 for sill plate size and anchorage requirements.

(j) Galvanized nails shall be hot dipped or tumbled.

(k) For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

Typical Layout for Shear Walls

TABLE 2

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR HORIZONTAL APA PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR, LARCH OR SOUTHERN PINE[(a)] FOR WIND OR SEISMIC LOADING[(g)] (See also IBC Table 2306.3.1)

Panel Grade	Common Nail Size[(f)]	Minimum Nail Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Framing Member at Adjoining Panel Edges and Boundaries (in.)	Blocked Diaphragms: 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)[(b)]: 6 / Nail Spacing (in.) at other panel edges (Cases 1, 2, 3 & 4)[(b)]: 6	4 / 6	2-1/2[(c)] / 4	2[(c)] / 3	Unblocked Diaphragms: Nails Spaced 6" max. at Supported Edges[(b)]: Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 & 6)
APA STRUCTURAL I grades	6d[(e)] (0.113" dia.)	1-1/4	5/16	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d (0.131" dia.)	1-3/8	3/8	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[(d)] (0.148" dia.)	1-1/2	15/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240
APA RATED SHEATHING; APA RATED STURD-I-FLOOR and other APA grades except Species Group 5	6d[(e)] (0.113" dia.)	1-1/4	5/16	2	170	225	335	380	150	110
				3	190	250	380	430	170	125
			3/8	2	185	250	375	420	165	125
				3	210	280	420	475	185	140
	8d (0.131" dia.)	1-3/8	3/8	2	240	320	480	545	215	160
				3	270	360	540	610	240	180
			7/16	2	255	340	505	575	230	170
				3	285	380	570	645	255	190
			15/32	2	270	360	530	600	240	180
				3	300	400	600	675	265	200
	10d[(d)] (0.148" dia.)	1-1/2	15/32	2	290	385	575	655	255	190
				3	325	430	650	735	290	215
			19/32	2	320	425	640	730	285	215
				3	360	480	720	820	320	240

(a) For framing of other species: Find specific gravity for species of lumber in the AF&PA NDS. Find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment shall not be greater than 1.

(b) Space fasteners maximum 12 inches o.c. along intermediate framing members (6 inches o.c. when supports are spaced 48 inches o.c. or greater).

(c) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches o.c. or 2-1/2 inches o.c.

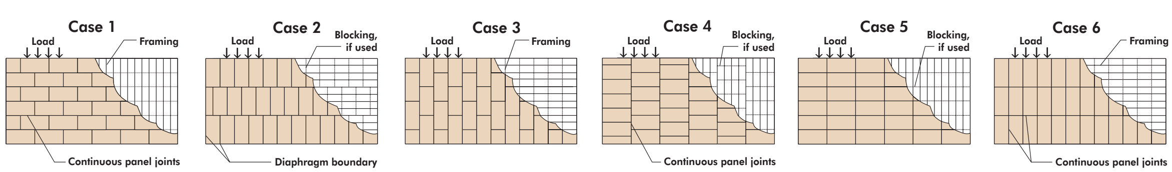
(d) Framing at adjoining panel edges shall be 3 inch nominal or wider, and nails shall be staggered where both of the following conditions are met: (1) 10d nails having penetration into framing of more than 1-1/2 inches and (2) nails are spaced 3 inches o.c. or less.

(e) 8d is recommended minimum for roofs due to negative pressures of high winds.

(f) The minimum nominal width of frraming members not located at boundaries or adjoining panel edges shall be 2 inches

(g) For shear loads of normal or permanent load duration as defined by AF&PA NDS, the values in the table above shall be multiplied by 0.63 and 0.56, respectively.

Note: Design for diaphragm stresses depends on direction of continuous panel joints with reference to load, not on direction of long dimension or strength axis of sheet. Continuous framing may be in either direction for blocked diaphragms.



Design Examples

The following design examples are based on provisions of the 2006 International Building Code (IBC) unless otherwise stated.

EXAMPLE 1:
DETERMINE SHEAR WALL DESIGN FOR WIND LOADING

Given:

•Commercial building
•Wind loading
•Wall requires 5/8-inch gypsum sheathing applied under wood structural panel for one-hour fire rating
•Required shear wall capacity is 670 plf

Find:

Panel thickness, nail size and nailing schedule

Solution:

Using Table 1, check the "Panels Applied Over 1/2-inch or 5/8-inch Gypsum Sheathing" area of table. Check "APA RATED SHEATHING…" rows first since Structural I may not be readily available in all areas. From Table 1, note that 10d nails with 3-inch nail spacing at panel edges and 12-inch nail spacing at intermediate framing for a sheathing thickness of 3/8, 7/16 or 15/32 inch will provide a capacity of 490 plf (pounds per lineal foot) provided that the framing at adjoining panel edges is 3 inch nominal or wider (Footnote f). Per IBC Section 2306.4.1, the allowable shear capacity of the shear wall can be increased by 40% for wind design, therefore the allowable capacity of this wall is 490 plf x 1.4 = 686 plf. Since 686 plf > 670 plf, this selection is OK for use.

Commentary:

See Example 3 for sizing the hold down and checking the chord forces. All wall segments must meet the shear wall aspect ratio requirements of IBC Table 2305.3.4, which for wind design is 3.5:1. Note that the 2006 IBC Section 2305.3.9 allows, for wind conditions, the shear capacity of the gypsum sheathing to be added to the wood structural panel capacity. Depending on the fasteners used, fastening schedule and gypsum product type chosen, this can add from 75 to 200 plf to the allowable design capacity of the shear wall.

EXAMPLE 2:
DETERMINE SHEAR WALL DESIGN FOR SEISMIC LOADING

Given:

- Residential building
- Seismic loading and Seismic Design Category C
- Typical wall sheathing thickness of 7/16 inch
- Typical nail size of 8d common
- Wall stud spacing of 24 inches o.c.
- Required shear wall capacity is 435 plf

Find:

Required nail spacing

Solution:

Using Table 1, check the "Panels Applied Direct to Framing" area of table. Check "APA RATED SHEATHING…" rows first because Structural I may not be readily available in all areas. From the table, note that 7/16-inch APA Rated Sheathing panels with 8d nails spaced at 3 inches at the panel edges and 6 inches at intermediate framing (Footnote b) will provide a capacity of 450 plf. Since 450 plf > 435 plf, this selection is OK for use.

Commentary:

See Example 3 for sizing the hold down and checking the chord forces. All wall segments must meet the shear wall aspect ratio requirements of IBC Table 2305.3.4, which for seismic design is 2:1 without penalty. For seismic design, shear wall aspect ratios greater than 2:1 but not exceeding 3.5:1 are permitted provided the factored shear resistance values are multiplied by 2w/h, where w and h are equal to the width and height of the shear wall segment respectively. Note that if this wall were designed in Seismic Design Category D, E, or F then Footnote i of Table 1 would apply and require 3x lumber framing at adjoining panel edges and possibly at sill plates, as well, per IBC Section 2305.3.11.

EXAMPLE 3:
SHEAR WALL DESIGN (TRADITIONAL SEGMENTED) – WITH SPECIFIC GRAVITY FRAMING ADJUSTMENT

Given:

The ASD (allowable stress design) shear load on the wall from the diaphragm is 3,000 lbf. The controlling load is assumed to be from wind pressures.

Find:

The shear wall design for the wall shown in Figure 4.

Solution:

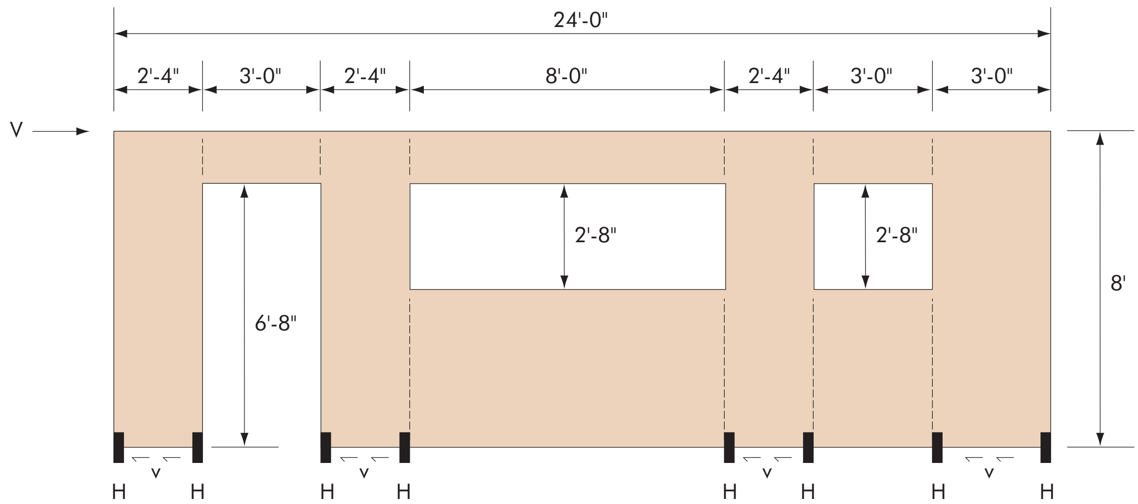
The total length of full height segments is 10 feet. Note per IBC Table 2503.3.4, that 3.5:1 is the minimum shear wall aspect ratio for wind loading. For seismic design, shear wall aspect ratios greater than 2:1 but not exceeding 3.5:1 are permitted provided the factored shear resistance values are multiplied by 2w/h, where w and h are equal to the width and height of the shear wall segment respectively.

1. The unit shear is:

$$v = V/L = 3000/10 = 300 \text{ plf}$$

FIGURE 4

BUILDING ELEVATION FOR SEGMENTED SHEAR WALL EXAMPLE



V = 3000 lbf, v = 300 plf, H = 2400 lbf

2. Assuming the framing will be spruce-pine-fir (with specific gravity, SG = 0.42), the shear values from the capacity table, Table 1, must be adjusted according to Footnote a. The specific gravity adjustment factor (SGAF) is:

$$SGAF = 1-(0.5\text{-}SG) = 1-(0.5-0.42) = 0.92$$

According to IBC Section 2306.4.1 of the 2006 IBC, for wind loads the allowable shear capacities are permitted to be increased by 40%.

From Table 1, 7/16-inch wood structural panels with 8d common nails at 6 inches o.c. on supported edges will provide an allowable capacity, $v_{allow.}$ of:

$$v_{allow.} = 260(0.92)1.4 = 335 \text{ plf} \quad 300 \text{ plf}$$

Note the increase to 15/32-inch panel design values is taken in accordance with Footnote d of Table 1, assuming studs will be placed 16 inches o.c. and the panel will be oriented with the 8-foot direction vertical. Also, as stated in Example 1, the shear wall capacity of the gypsum sheathing can be added to the wood structural panel shear wall capacity, but is not done in this example.

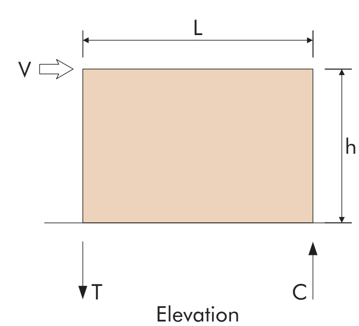
3. The hold downs must be located at the ends of each full height segment as shown in Figure 4 and designed to resist uplift tension, T, as shown in Figure 5. The compression, C, in the end studs, due to lateral loads acting on the shear wall is equal to the tension uplift:

$$T = C = V/L(h) = vh = 300(8) = 2400 \text{ lbf}$$

Where v = V/L = unit shear (lb/ft)

FIGURE 5

OVERTURNING FORCES



Note that no dead load is assumed to counter the hold-down uplift. Dead load would take away tension uplift forces but adds to the compression forces and also adds to the compression perpendicular to grain stress on the bottom plate. Due to the compression chord bearing on the bottom plate of the shear wall, the bottom plate should also be checked to ensure adequate compression-perpendicular-to-grain capacity.

4. The end studs to which the hold down is attached, sometimes called chords, must be capable of resisting the tension and compression forces due to the lateral forces in the wall as shown in Figure 5, in addition to the gravity-load forces. The required combination of lateral and gravity loads is provided in IBC Section 1605.

EXAMPLE 4:
SHEAR WALL DEFLECTION

Given:

Calculate the deflection of the shear wall in Example 3.

Solution:

The total shear-wall deflection will be considered to be a function of the deflection of the full-height segments. For this example, a weighted average based on wall rigidities will be used to calculate the total shear wall deflection. Wall rigidities will be assumed to be relative to wall length assuming consistent framing and nailing patterns. Another possible approach to this problem would be to assume the deflection of one segment represents the wall deflection.

Shear wall deflection analysis usually involves engineering judgment. In this example the walls are narrow, with an approximate aspect ratio of 3.5:1. The accuracy of the shear wall deflection equation at aspect ratios greater than 2:1 is questionable. In the absence of any guidelines for narrow shear wall deflection, however, the 4-term equation from IBC Section 2305.3.2 will be used. Other factors that would reasonably be expected to influence the accuracy of wall deflection calculations (by stiffening the wall) are the presence of sheathing above and below openings, and wall finish materials (such as siding, stucco, and gypsum). No guidelines currently exist to account for these aspects either, but testing indicates these aspects add significant stiffness to the walls (Cobeen et al. 2004).

The deflection of the 2.33-ft wall segment is calculated with the following IBC equation 23-2:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{hd_a}{b}$$

where,

v = 300 plf, unit load (given from example)
h = 8 feet, wall height (given from example)
E = 1,200,000 psi, for spruce-pine-fir studs – stud grade (from the NDS)
A = 10.5 in.^{2}, for two 2x4 vertical end studs
$Gt = G_v t_v$ = 83,500 lbf/in., for 7/16-inch (24/16 Span Rating) OSB (from Appendix Table-A-3)
b = 2.33 feet, wall width (given from example)
e_n = Nail slip (equation from Appendix Table A-2). First, the load per nail must be calculated:
Load per nail, v_{nail}:
$v_{nail} = v/(12/S) = 300/(12/6) = 150$ lbf/nail (where S = nail spacing in inches)
$e_n = 1.2(v_{nail}/616)^{3.018} = 1.2(150/616)^{3.018} = 0.017$ in. (the 1.2 is for non-structural I panels)

d_a = Hold-down slip, d_a = 0.033 inch, from hold-down manufacturer's catalog for lowslip hold down with an allowable tensile capacity of 3,375 lbf. in. spruce-pine-fir framing. Assuming hold-down slip to be linear, a reduction could be made since the design uplift force is 2,400 lbf (from Example 3), thus, the expected slip at the design lateral load would be (0.033/3,375)2,400 = 0.023 in.

The deflection of each component is:

$\Delta_{bending} = 8vh^3/(EAb) = 8(300)8^3/(1{,}200{,}000(10.5)2.33) = 0.042$ in.
$\Delta_{shear} = vh/(Gt) = 300(8)/83{,}500 = 0.029$ in.
$\Delta_{nail\ slip} = 0.75h(e_n) = 0.75(8)0.017 = 0.102$ in.
$\Delta_{hold\ down} = h(d_a)/b = 8(0.023)/2.33 = 0.079$ in.

The total shear wall deflection, Δ, is the summation of each component:

$\Delta = 0.042 + 0.029 + 0.102 + 0.079 = 0.252$ in.

The deflection for the 3-foot wall segment is calculated (not shown) as 0.225 inch. For information, Table 3 summarizes the wall segment relative rigidities, load, and deflection.

TABLE 3

RELATIVE SHEAR WALL RIGIDITIES FOR EXAMPLE 4

Wall Segment	Length	R(a)	R/ΣR	V(b) lbf	v lbf/ft	Δ (in.)
1	2.33	0.78	0.23	700	300	0.252
2	2.33	0.78	0.23	700	300	0.252
3	2.33	0.78	0.23	700	300	0.252
4	3	1.00	0.30	901	300	0.225
Σ	10.0	3.33		3000		

(a) R = relative rigidity based on wall length (length of wall segment ÷ length of longest wall segment).
(b) Shear distributed to wall segments in proportion to wall length.

The total wall deflection is calculated as a weighted average:

$$\Delta = 3\left(\frac{2.33}{10}\right)0.252 + \left(\frac{3}{10}\right)0.225 = 0.244 \text{ in.}$$

Commentary:

Shear wall deflection is important in seismic design for checking drift limitations, building separations, and in determining whether the diaphragm should be considered rigid or flexible. These same concepts could be used in wind design, but they are not part of the wind design requirements in the IBC.

The allowable story drift, Δ_a, for seismic design is given in ASCE 7-05 Section 12.12. For buildings in occupancy category I as defined in ASCE 7-05 Table 12.12-1, the allowable story drift is 0.025 x h, where, h = the story height. For an 8-ft story height, Δ_a = 2.4 inches.

The design story drift is determined in accordance with ASCE 7-05 Section 12.8.6. The design story drift requires the deflection determined by an elastic analysis to be increased by the deflection amplification factor, C_d (Table 12.2-1 in ASCE 7-05). C_d is 4.0 for light frame wood shear walls. If we assume, for example, the calculated deflection in the above was computed using code-specified elastic earthquake forces (instead of the given wind forces), then the design story drift would be: $\Delta \times C_d = 0.241(4.0) = 0.964$ in. < 2.4 in. ∴ OK

EXAMPLE 5:
SHEAR WALL DESIGNED WITH OPENINGS – PERFORATED SHEAR WALL DESIGN METHOD (IBC SECTION 2305.3.8.2)

Given:

The same wall section as shown in Example 3 will be redesigned as a perforated shear wall to highlight the differences between the two methods. In this empirical-based method, the entire wall, not just full height segments, is considered the shear wall and the openings are accounted for with a shear-resistance-adjustment factor, C_o. Hold downs are only required at the ends of the wall since the entire wall is treated as one shear wall with openings.

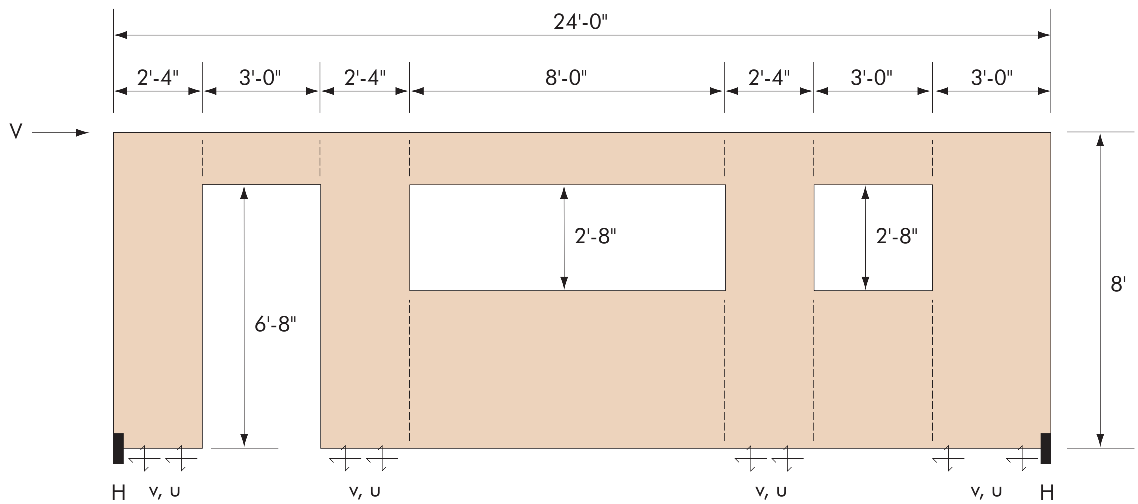
The shear load on the wall from the diaphragm, V, is 3,000 lbf from wind pressures. The length of the perforated shear wall is defined by hold-down location, H, as shown in Figure 6.

Find:

The shear wall design for the wall shown in Figure 6.

FIGURE 6

BUILDING ELEVATION FOR PERFORATED SHEAR-WALL EXAMPLE



V = 3000 lbf, v, u = 450 plf, H = 3600 lbf

Solution:

1. The unit shear in the wall's full-height segments is 300 plf (see Example 3).

2. Adjustment factors: The specific gravity adjustment factor (SGAF) is 0.92 (from Example 3).

The allowable shear capacities can be increased by 40% for wind loads, per 2006 IBC Section 2306.4.1.

The total length of full-height segments is 10 feet. Note that all the full-height segments in Figure 6 meet the minimum aspect ratio requirement for shear walls as specified in IBC Table 2305.3.4 (96"/3.5 = min. 27.4") and therefore may be counted when determining length of full-height segments. Full-height segments less than the minimum cannot be counted when determining length of full-height segment.

Two items are needed for finding the shear-resistance-adjustment factor, C_o:

- percent full-height sheathed and
- maximum opening height.

The percent full-height sheathed is the length of the full-height segments divided by the total length of wall = 10/24 = 0.42 (or 42%).

The maximum opening height is 6' 8".

From IBC Table 2305.3.8.2, the shear-resistance adjustment factor, C_o, is 0.53 (conservative using 40% full-height sheathed instead of interpolating).

3. From Table 1, 7/16-inch wood structural panels with 8d common nails spaced at 3 inches at panel edges will provide an adjusted allowable capacity, $v_{allow.}$ of:

$$v_{allow.} = 450(1.4)(0.92)(0.53) = 307 \text{ plf} \geq 300 \text{ plf} \therefore \text{OK}$$

4. The hold downs must be designed to resist:

$T=V(h)/(C_o(L_i)) = 3{,}000(8)/(0.53(10)) = 4{,}528$ lbf (Equation 23-3 of the IBC)

Where L_i = the sum of the aspect-ratio-qualifying full-height segments

5. The shear and uplift between hold downs, v and u, from Figure 6 must resist:

$v = u = V/(C_o(L_i)) = 3{,}000/(0.53(10)) = 566$ plf (Equation 23-4 of the IBC)

6. Provisions for calculating the total shear-wall deflection of a perforated shear wall (IBC Section 2305.3.8.2.9) state that the total deflection shall be based on the maximum deflection of any full-height segment divided by the shear-resistance adjustment factor, C_o. Using the deflection equation from Example 4 with all terms the same but with 3-inch o.c. edge nailing, and a higher capacity hold down (5,480-lbf capacity and 0.045-inch deflection), the deflection of the 2.33-foot shear-wall segment becomes 0.151 inch. The total perforated-shear-wall deflection is calculated as:

$$\Delta = 0.151/C_o = 0.151/0.53 = 0.285 \text{ in.}$$

EXAMPLE 6:

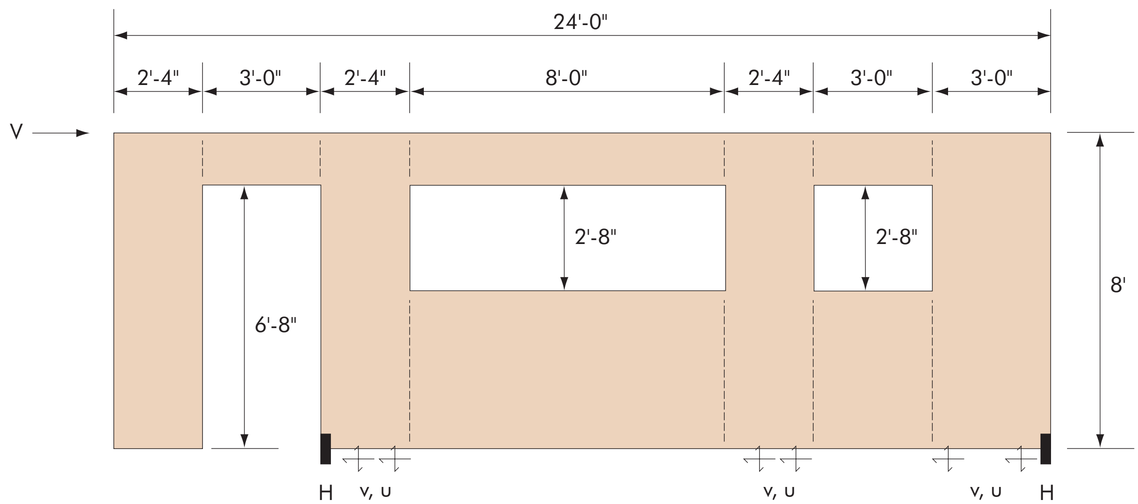
SHEAR WALL DESIGNED WITH OPENINGS – PERFORATED SHEAR WALL (IBC SECTION 2305.3.8.2)

Given:

Repeat Example 5, but in this example the perforated shear wall will be defined with hold downs as shown in Figure 7. The length of the perforated shear wall is 18.67 feet. The length of full-height segments, L_i, is 7.67 feet.

FIGURE 7

BUILDING ELEVATION FOR PERFORATED SHEAR-WALL EXAMPLE WITH ALTERNATE WALL LENGTH DEFINITION



V = 3000 lbf, v, υ = 350 plf, H = 2800 lbf

Solution:

1. The unit shear in the full-height wall segments is:

$v = V/7.67 = 3{,}000/7.67 = 391$ plf

2. Adjustment factors:

The specific gravity adjustment factor (SGAF) is still 0.92 (from Example 3).

The allowable shear capacities can be increased by 40% for wind loads, per IBC Section 2306.4.1.

The percent full-height sheathed is the length of the full-height segments divided by the total length of wall = 7.67/18.67 = 0.41 (or 41%).

The maximum opening height is now 2' 8".

From IBC Table 2305.3.8.2, the shear-resistance adjustment factor, C_o, is 1.0.

3. From Table 1, 7/16-inch wood structural panels with 8d-common nails spaced 4 inches o.c. at supported panel edges will provide an adjusted allowable capacity, $v_{allow.}$ of:

$v_{allow.} = 350(1.4)(0.92)(1.0) = 451 \text{ plf} \geq 391 \text{ plf} \therefore$ OK

4. The hold downs must be designed to resist:

$T = V(h)/(C_o(L_i)) = 3000(8)/(1.0(7.67)) = 3129$ lbf (Equation 23-3 of the IBC)

Where L_i = sum of aspect ratio qualifying full-height segments

5. The shear and uplift between hold downs, v and u, from Figure 7 must resist:

$v = u = V/(C_o(L_i)) = 3000/(1.0(7.67)) = 391$ plf (Equation 23-4 of the IBC)

Commentary:

Examples 5 and 6 show that by redefining the perforated shear wall boundaries to eliminate the door opening, the shear-resistance adjustment factor, C_o, becomes smaller and as a result, fewer nails may be required, as well as smaller shear, uplift and hold-down forces. Note that this places one or more hold downs away from the corners of the building.

EXAMPLE 7:
DIAPHRAGM DESIGN FOR WIND LOADING

Given:

•Residential roof diaphragm
•Wind loading
•Trussed roof
•Unblocked diaphragm required
•Required diaphragm capacity is 180 plf
•Panel orientation is unknown

Find:

Panel thickness, nail size and nailing schedule

Solution:

Using Table 2, refer to the "Unblocked Diaphragms" area of the table. As panel orientation is unknown, use the "All other configurations…" column since these values will be conservative. Check "APA RATED SHEATHING…" rows first since Structural I may not be readily available in all areas. Similarly, check only rows with 2-inch-minimum nominal framing width as the framing is made up of trusses. From Table 2, note that 8d nails with 15/32-inch sheathing over 2x_ framing yields a capacity of 180 plf with the noted 6- and 12-inch nail spacing. As 180 plf is equal to the required 180 plf capacity, this selection is OK for use.

Commentary:

Note that since this is for wind loading, the allowable diaphragm design capacity can be increased by 40% per IBC Section 2306.3.2. Also, check with the truss manufacturer to insure that the Douglas-fir or southern pine framing species assumption made above is correct (see Footnote a and Example 3 for a case using other lumber framing). The diaphragm chords will also have to be checked. See Example 9 for determining the chord forces. Also, the diaphragm must be connected to the supporting shear walls sufficiently to transfer the maximum shear (180 plf).

EXAMPLE 8:
DETERMINE DIAPHRAGM DESIGN FOR SEISMIC LOADING

Given:

•Commercial roof diaphragm
•Seismic loading
•Trussed roof
•Required diaphragm capacity is 350 plf
•Case 1 panel orientation (see diagrams in Table 2, page 8)

Find:

Panel thickness, nail size and nailing schedule

Solution:

Using Table 2, refer to the "Unblocked Diaphragms" area of the table first. Note that no solution is possible. Next, check the "Blocked Diaphragms" area of the table. Check "APA RATED SHEATHING…" rows first. Similarly, check only rows with 2-inch-minimum nominal framing width as the framing is made up of trusses. From Table 2, note that 8d nails with 15/32-inch APA Rated Sheathing over 2x_ framing yields a capacity of 360 plf. Nails must be placed 4 inches o.c. at all diaphragm boundaries and 6 inches o.c. at all other panel edges. As 360 plf is greater than 350 plf, this selection is OK for use.

Commentary:

Also, check with the truss manufacturer to insure that the Douglas-fir or southern pine framing species assumption made above is correct (see Footnote a and Example 3 for a case using other lumber framing). The diaphragm chords will also have to be checked. See Example 9 for determining the chord forces. Also, the diaphragm must be connected to the supporting shear walls adequately to transfer the maximum shear (350 plf).

EXAMPLE 9:

DESIGN OF WOOD STRUCTURAL PANEL ROOF DIAPHRAGM – ALL FRAMING DOUGLAS-FIR – PANEL JOINTS BLOCKED – SEISMIC DESIGN

Given:

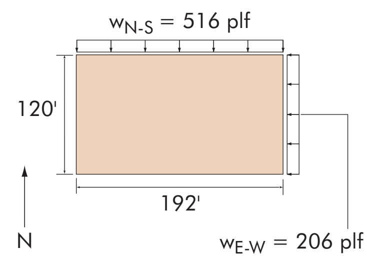
The panelized roof has purlins spaced 8 feet o.c. with sub-purlins spaced 2 feet o.c., thus achieving blocking at all panel edges. Load Case 4 is appropriate for load in the N-S direction and Case 2 for the E-W direction. The uniformly distributed load on the diaphragm is 516 plf in the N-S direction and 206 plf in the E-W direction. The building dimensions are 192 ft x 120 ft as shown in Figure 8. The loads are from seismic forces.

Find:

The diaphragm design for the panelized roof system shown in Figure 8.

FIGURE 8

BUILDING PLAN VIEW FOR BLOCKED DIAPHRAGM EXAMPLE



Solution:

1. The maximum diaphragm shear is:

$$V_{N\text{-}S} = \frac{wl}{2B} = \frac{516(192)}{2(120)} = 413 \text{ plf}$$

$$V_{E\text{-}W} = \frac{wl}{2B} = \frac{206(120)}{2(192)} = 64 \text{ plf}$$

2. From Table 2, 15/32 inch APA Rated Sheathing wood structural panels with 8d-common nails spaced at 2.5 inches o.c. at the diaphragm boundary; 2.5 inches o.c. on all N-S panel edges and 4 inches o.c. at all E-W panel edges will provide an allowable capacity, $v_{allow.}$ of:

$$v_{allow.} = 530 \text{ plf} \geq 413 \text{ plf} \therefore \text{OK*}$$

Examining the shear in the diaphragm along the length, as shown in Figure 9, provides an opportunity to reduce the nail density.

Table 4 summarizes the edge nail schedule requirements for different selected zones in the diaphragm. All field nailing should be at 12 inches o.c. in accordance with Footnote b in Table 2.

By inspection, the 6 inches o.c. nailing on all edges is adequate for the E-W load (max = 64 plf).

3. The maximum chord force, T (tension) or C (compression), is obtained by resolving the maximum diaphragm moment into a couple by dividing the maximum moment by the depth:

$$T_{N\text{-}S} = C_{N\text{-}S} = \frac{wl^2}{8B} = \frac{516(192)^2}{8(120)} = 19{,}814 \text{ lbf}$$

$$T_{E\text{-}W} = C_{E\text{-}W} = \frac{wl^2}{8B} = \frac{206(120)^2}{8(192)} = 1{,}931 \text{ lbf}$$

The chord force can be calculated at any distance along the length by using the moment equation as a function of length and then dividing the moment by the diaphragm depth. Typically, the ledger will carry the chord force, which is often either steel or wood. The ledger design is not shown in this example.

*Note that as this is a seismic design, the 40% increase to allowable diaphragm values is not appropriate.

FIGURE 9

MAIN DIAPHRAGM

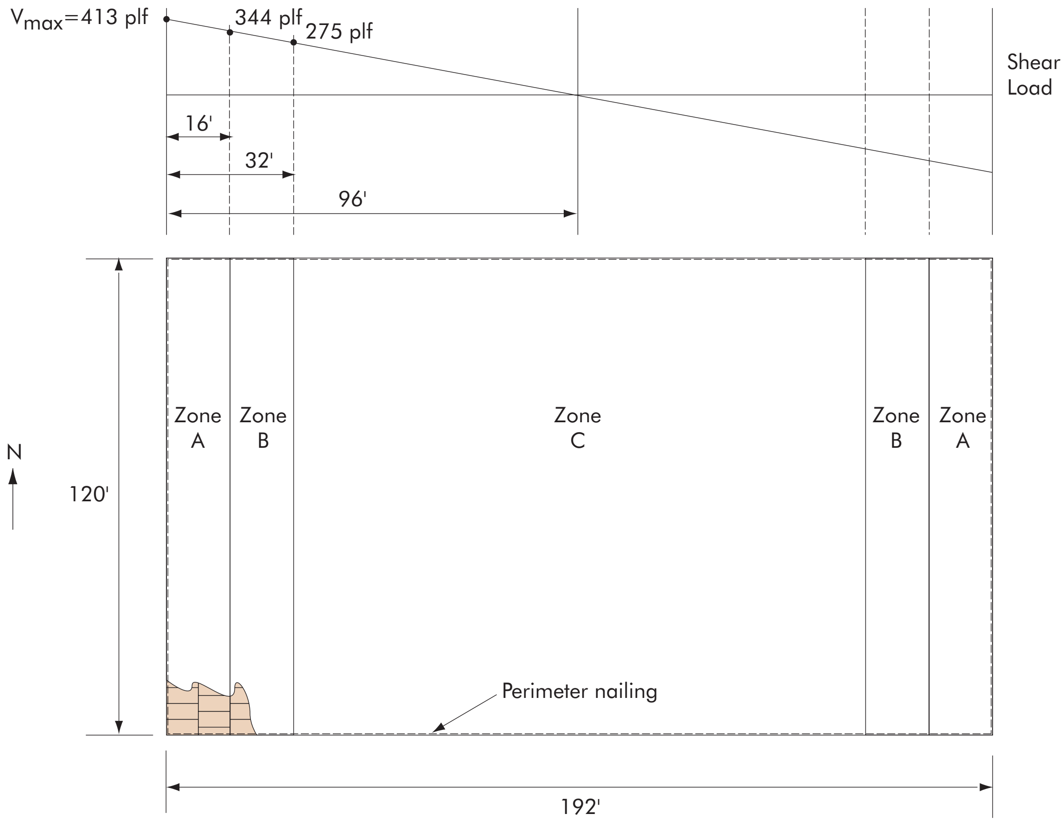


TABLE 4

15/32 WOOD STRUCTURAL PANEL WITH 8D COMMON (0.131" x 2-1/2") NAILS

N-S	Zone	N-S Continuous Edge Nailing (in. o.c.)*	Nailing at Other Edges (in. o.c.)	Allowable Shear (plf)	Load: ASD Shear (plf)
Case 4	A	2.5	4	530	413
	B	4	6	360	344
	C	6	6	270	275

*Framing at adjoining panel edges shall be 3-inch nominal or wider.

EXAMPLE 10:
DESIGN OF WOOD STRUCTURAL PANEL ROOF DIAPHRAGM – UNBLOCKED WITH SPRUCE-PINE-FIR FRAMING – WIND

Given:

Design the diaphragm for a roof system consisting of light-frame wood trusses spaced at 24 inches o.c., without blocking. Panel orientation is assumed as Load Case 1 for load in the N-S direction and Load Case 3 for the E-W direction. The specific gravity of spruce-pine-fir is 0.42. A wind pressure of 19 psf is assumed to act uniformly in both the east-west and north-south directions as shown in Figure 10. The building dimensions are 72 ft x 42 ft.

Solution:

1. The uniformly distributed load acting on the roof diaphragm by tributary area is:

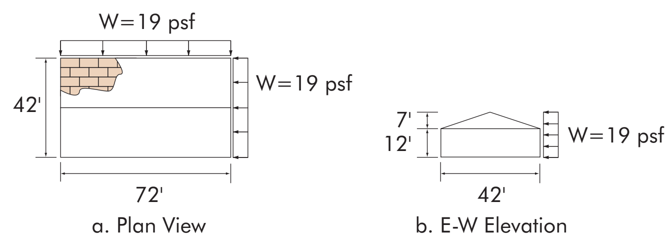
$$w_{N\text{-}S} = 19\left(7 + \frac{12}{2}\right) = 247 \text{ plf}$$

$$w_{E\text{-}W} = 19\left(\frac{7}{2} + \frac{12}{2}\right) = 180 \text{ plf}$$

Note that where the wind is normal to the gable-end wall, 1/2 the gable-end-wall height is used to represent the actual area.

FIGURE 10

BUILDING DIMENSIONS FOR UNBLOCKED DIAPHRAGM EXAMPLE



2. The maximum diaphragm shear in the roof diaphragm is:

$$v_{N\text{-}S} = \frac{wl}{2B} = \frac{247(72)}{2(42)} = 212 \text{ plf}$$

$$v_{E\text{-}W} = \frac{wl}{2B} = \frac{180(42)}{2(72)} = 52 \text{ plf}$$

3. Diaphragm nailing capacity. Since the framing will be spruce-pine-fir with SG = 0.42, the shear values from Table 2 must be adjusted according to Footnote a. The specific-gravity adjustment factor (SGAF) is:

$$\text{SGAF} = 1\text{-}(0.5\text{-SG}) = 1\text{-}(0.5 - 0.42) = 0.92$$

From Table 2, 7/16-inch APA wood structural panels with 8d-common nails spaced at 6 inches on the supported edges will provide an adjusted allowable shear capacity, $v_{allow.}$, of:

$$v_{allow., Case 1} = 230(0.92) = 212 \text{ plf} \geq 212 \text{ plf} \therefore \text{OK (for the N-S direction)}$$

$$v_{allow., Case 3} = 170(0.92) = 156 \text{ plf} \geq 52 \text{ plf} \therefore \text{OK (for the E-W direction)}$$

4. The maximum chord force in tension (T) and compression (C) is:

$$T_{N\text{-}S} = C_{N\text{-}S} = \frac{wl^2}{8B} = \frac{247(72)^2}{8(42)} = 3810 \text{ lbf}$$

$$T_{E\text{-}W} = C_{E\text{-}W} = \frac{wl^2}{8B} = \frac{180(42)^2}{8(72)} = 551 \text{ lbf}$$

A double top plate, spliced together, will carry the chord force along the length. The splice design is not shown in this example. Recall from Example 10 that the chord force can be calculated at any distance along the length by using the moment equation as a function of length and then dividing the moment by the diaphragm depth.

Note that since this is for wind loading the allowable diaphragm design capacity can be increased by 40% per IBC Section 2306.3.2.

EXAMPLE 11:
CALCULATE DEFLECTION OF AN UNBLOCKED DIAPHRAGM

Given:

Same diaphragm as in Example 10

Solution:

Research by APA, as discussed in Appendix C, has shown that unblocked diaphragms deflect about two-and-a-half times that of blocked diaphragms, and for diaphragm framing spaced greater than 24 inches o.c. this difference increases to about three. The deflection of a blocked diaphragm is calculated by the following equation (Equation 23-1 in the IBC):

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

where,

v = 212 plf, maximum unit shear in diaphragm (given from example)
L = 72 feet, diaphragm length (given from example)
E = 1,200,000 psi, for spruce-pine-fir studs – stud grade (from the NDS)
A = 16.5 in.^{2}, for 2, 2x6 vertical end studs
$G_v t_v = Gt$ = 83,500 lbf/in., for 7/16-inch OSB (from Appendix Table A-3)
b = 42 feet, diaphragm depth (given from example)
e_n = nail slip (from Appendix Table A-2)
Δ_c = chord-splice slip (in.)
X = distance from chord splice to closest supporting shear wall

Load per nail, v_{nail} (needed for nail-slip calculation):

$v_{nail} = v/(12/S) = 212/(12/6) = 106$ lbf/nail (where S = nail spacing in inches)
$e_n = 1.2(v_{nail}/616)^{3.018} = 1.2(106/616)^{3.018} = 0.006$ in. (the 1.2 is for non-Structural I panels)

Chord-splice slip, Δ_c, will be assumed to be 0.03 inch in the tension chord splices and 0.005 inch in the compression-chord splices. These values are based on a review of diaphragm tests from APA (Research Report 138). APA test values for tension-chord slip range from 0.011 to 0.156 inch for the different configurations tested, with a value of about 0.03 inch being an "estimated average." In addition, APA research shows that compression-chord slip is about 1/6 of the tension-chord slip. Selecting values for chord-splice slip involve considerable engineering judgment. Alternate assumptions and techniques can be found in other sources (Breyer et. al., 2006; SEAOC, 2000), but no values appear to be definitive since many variables can be involved.

Chord splices will be located every 8 feet.

The deflection of each component is:

$\Delta_{bending} = 5vL^3/(8EAb) = 5(212)72^3/((8)(1{,}200{,}000)(16.5)(42)) = 0.060$ in.
$\Delta_{shear} = vL/(4Gt) = 212(72)/(4(83{,}500)) = 0.046$ in.
$\Delta_{nail\ slip} = 0.188L(e_n) = 0.188(72)0.006 = 0.081$ in.
$\Delta_{chord\ splice} = \Sigma(\Delta_c X)/(2b)$
$\Delta_c X_{tension\ chord} = 2[0.03(8) + 0.03(16) + 0.03(24) + 0.03(32)] = 4.8$ in.-ft
$\Delta_c X_{compression\ chord} = 2[0.005(8) + 0.005(16) + 0.005(24) + 0.005(32)] = 0.8$ in.-ft
$\Delta_{chord\ splice} = (4.8+0.8)/(2(42)) = 0.067$ in.

The total deflection, Δ_t, is a summation of the terms above multiplied by 2.5 to account for the unblocked diaphragm construction:

$\Delta_t = (0.060 + 0.046 + 0.081 + 0.067)2.5 = 0.635$ in.

EXAMPLE 12:
DESIGN OF A SUBDIAPHRAGM

Given:

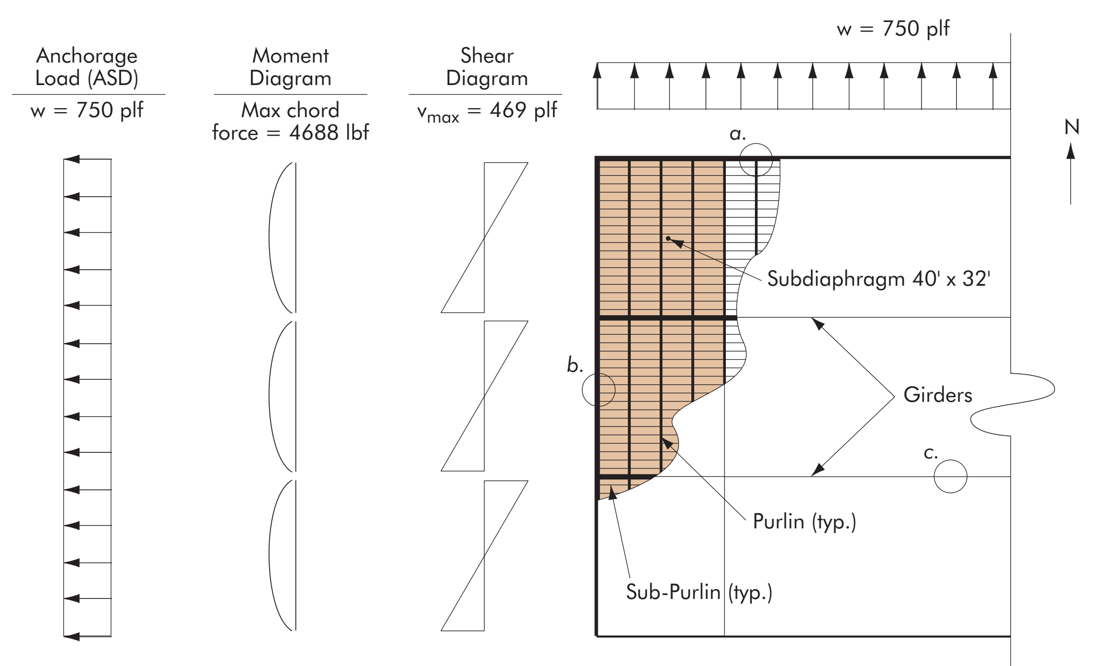
A common problem observed after large seismic events is roof-to-wall separation, particularly for high-mass walls such as concrete or masonry. In recent building codes, more attention has been given to this critical connection with increased connection-force requirements. Continuous tension ties from one main diaphragm chord to the other opposite chord are required. The following design example is based on the provisions of ASCE 7-05. (See ASCE 7-05 Section 12.14.7.5.1.)

Subdiaphragms are useful for concentrating the forces and connections needed to provide a continuous tension cross-tie path from one diaphragm support to the other. In this example, the east and west wall are required to have a continuous cross-tie connection. (For a more detailed description of subdiaphragms, see EWS Data File: *Lateral Load Connections for Low Slope Roof Diaphragms,* Form Z350.) This can be achieved in two ways: 1) by directly connecting all, or enough, of the subpurlins together from east to west so that continuity is achieved, which requires many small connections, or 2) by using a subdiaphragm to concentrate the wall connection force into the main girders, which requires fewer but larger connections. Continuity between the north and south walls can be achieved by purlin connections (Location *a.* in Figure 11).

Considering the diaphragm designed in Example 9, design a subdiaphragm for the main diaphragm. The assumed design anchorage force, F_p, for the wall-to-diaphragm connection is 750 plf as shown in Figure 11.

FIGURE 11

SUBDIAPHRAGM AND ANCHORAGE CONNECTION FORCES



*Wood structural panels not shown for clarity.

Solution:

Assume the width of the building is divided into 3 bays of 40 feet each as shown in Figure 11.

The maximum length-to-width ratio of the structural subdiaphragm is 2.5:1 (per ASCE 7-05 Section 12.14.7.5.1). Thus, the minimum subdiaphragm depth is: 40-ft/2.5 = 16 ft

1. After several iterations a subdiaphragm depth of 32 feet was selected for load compatibility with the existing main diaphragm nailing pattern. The maximum subdiaphragm shear for this depth is:

$$v = \frac{wl}{2B} = \frac{750(40)}{2(32)} = 469 \text{ plf}$$

Zone A nailing, as shown in Example 10, Figure 9, is adequate, though the area of Zone A must be increased to extend 32 feet from the east and west walls as shown in Figure 12.

2. The maximum chord force, T (tension) or C (compression), in the subdiaphragm is:

$$T = C = \frac{wl^2}{8B} = \frac{750(40)^2}{8(32)} = 4{,}688 \text{ lbf}$$

The steel-channel ledger and purlin act as subdiaphragm chords. Their design is not shown here.

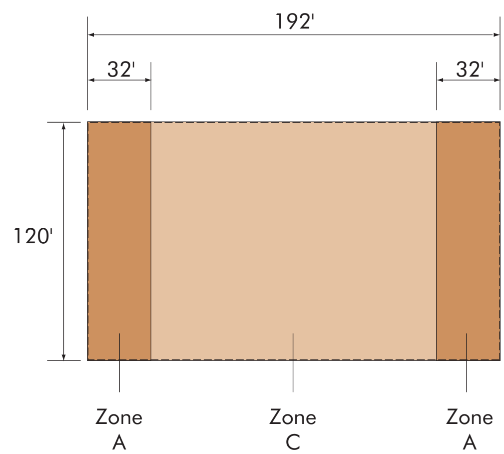
3. The three general connection forces are:

N-S walls (see location a in Figure 11), for 8-foot purlin spacing the anchorage force is:

$$F = 750(8) = 6{,}000 \text{ lbf}$$

FIGURE 12

FINAL DIAPHRAGM SHEATHING LAYOUT



E-W walls (see location b in Figure 11), for 2-foot subpurlin spacing the anchorage force is:

$$F = 750(2) = 1{,}500 \text{ lbf}$$

Main girder connection force is (see location c in Figure 11):

$$F = 750(40) = 30{,}000 \text{ lbf}$$

4. The final diaphragm sheathing design is shown in Figure 12, and the Zone nailing is specified in Example 9.

APPENDIX A – Reference Information

TABLE A-1

NOMINAL THICKNESS BY SPAN RATING.
*(The nominal thickness is given. The predominant thickness for each Span Rating is highlighted in **bold** type.)*

	Nominal Thickness (in.)										
Span Rating	**3/8**	**7/16**	**15/32**	**1/2**	**19/32**	**5/8**	**23/32**	**3/4**	**7/8**	**1**	**1-1/8**
APA Rated Sheathing											
24/0	**.375**	.437	.469	.500							
24/16		**.437**	.469	.500							
32/16			**.469**	.500	.594	.625					
40/20					**.594**	.625	.719	.750			
48/24							**.719**	.750	.875		
APA Rated Sturd-I-Floor											
16 oc					**.594**	.625					
20 oc					**.594**	.625					
24 oc							**.719**	.750			
32 oc									**.875**	1.000	
48 oc											**1.125**

Note: 1 inch = 25.4 mm

TABLE A-2

FASTENER SLIP EQUATIONS (See also IBC Table 2503.2.2(1))

Fastener	**Minimum Penetration (in.)**	**For Maximum Loads up to (lbf)(c)**	**Approximate Slip, e_n (in.)(a)(b)** Green/Dry	Dry/Dry
6d common nail (0.113" x 2")	1-1/4	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
8d common nail (0.131" x 2-1/2")	1-7/16	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
10d common nail (0.148" x 3")	1-5/8	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
14-ga staple	1 to 2	140	$(V_n/902)^{1.464}$	$(V_n/596)^{1.999}$
14-ga staple	2	170	$(V_n/674)^{1.873}$	$(V_n/461)^{2.776}$

(a) Fabricated green/tested dry (seasoned); fabricated dry/tested dry. V_n-=-fastener load.

(b) Values based on Structural-I panels fastened to Group-II lumber, specific gravity 0.50 or greater. Increase slip by 20% when panels are not Structural-I.

(c) ASD basis.

TABLE A-3

PANEL RIGIDITY THROUGH THE THICKNESS Gt (lbf/in. of panel depth) (See also IBC Table 2305.2.2(2))

Span Rating	Stress Parallel to or Perpendicular to Strength Axis – Plywood 3-ply	Plywood 4-ply	Plywood 5-ply	OSB
24/0	25,000	32,500	37,500	77,500
24/16	27,000	35,000	40,500	83,500
32/16	27,000	35,000	40,500	83,500
40/20	28,500	37,000	43,000	88,500
48/24	31,000	40,500	46,500	96,000
16 oc	27,000	35,000	40,500	83,500
20 oc	28,000	36,500	42,000	87,000
24 oc	30,000	39,000	45,000	93,000
32 oc	36,000	47,000	54,000	110,000
48 oc	50,500	65,500	76,000	155,000
Structural I Multiplier	1.3	1.3	1.1	1.0

APPENDIX B – High-Load Diaphragms

Tables 1 and 2 present allowable shears which apply to most shear wall and diaphragm designs. Occasionally, due to higher lateral loads or to building geometry or layout, higher allowable shears are required. Calculation by principles of mechanics using values of fastener strength and panel shear values (see *APA Research Report 138*) is one way to design for higher shears.

Another option is to use Table B-1 for high-load horizontal diaphragms (see also IBC Table 2306.3.2).

For high-load shear walls, structural panels may be applied to both faces of framing. Allowable shear for the wall may be taken as twice the tabulated shear for one side per IBC Section 2305.3.9. Where the shear capacities of each side are not equal, the allowable shear may be either the shear for the side with the higher capacity or twice the shear for the side with the lower capacity, whichever is greater. If nail spacing is less than 6 inches o.c. on either side, panel joints should be offset to fall on different framing members or framing should be 3-inch nominal or greater and nails on each side should be staggered.

TABLE B-1

ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL BLOCKED DIAPHRAGMS UTILIZING MULTIPLE ROWS OF FASTENERS (HIGH-LOAD DIAPHRAGMS) WITH FRAMING OF DOUGLAS-FIR-LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b,f,g,h]

						Blocked Diaphragms Cases 1 and 2[d] Fastener Spacing Per Line at Boundaries (in.)					
						4		2-1/2		2	
Panel Grade[c]	Common Nail Size or Staple Gage	Minimum Fastener Penetration in Framing (in.)	Minimum Nominal Panel Thickness (in.)	Minimum Nominal Width of Framing Member at Adjoining Panel Edges and Boundaries[e]	Lines of Fasteners	Fastener Spacing Per Line at Other Panel Edges (in.) 6	4	4	3	3	2
APA STRUCTURAL I grades	10d common nails (0.148" dia.)	1-1/2	15/32	3	2	605	815	875	1,150	–	–
				4	2	700	915	1,005	1,290	–	–
				4	3	875	1,220	1,285	1,395	–	–
			19/32	3	2	670	880	965	1,255	–	–
				4	2	780	990	1,110	1,440	–	–
				4	3	965	1,320	1,405	1,790	–	–
			23/32	3	2	730	955	1,050	1,365	–	–
				4	2	855	1,070	1,210	1,565	–	–
				4	3	1,050	1,430	1,525	1,800	–	–
	14 gage staples	2	15/32	3	2	600	600	860	960	1,060	1,200
				4	3	860	900	1,160	1,295	1,295	1,400
			19/32	3	2	600	600	875	960	1,075	1,200
				4	3	875	900	1,175	1,440	1,475	1,795
APA RATED SHEATHING, APA RATED STURD-I-FLOOR and other APA grades except Species Group 5	10d common nails (0.148" dia.)	1-1/2	15/32	3	2	525	725	765	1,010	–	–
				4	2	605	815	875	1,105	–	–
				4	3	765	1,085	1,130	1,195	–	–
			19/32	3	2	650	860	935	1,225	–	–
				4	2	755	965	1,080	1,370	–	–
				4	3	935	1,290	1,365	1,485	–	–
			23/32	3	2	710	935	1,020	1,335	–	–
				4	2	825	1,050	1,175	1,445	–	–
				4	3	1,020	1,400	1,480	1,565	–	–
	14 gage staples[f]	2	15/32	3	2	540	540	735	865	915	1,080
				4	3	735	810	1,005	1,105	1,105	1,195
			19/32	3	2	600	600	865	960	1,065	1,200
				4	3	865	900	1,130	1,430	1,370	1,485
			23/32	4	3	865	900	1,130	1,490	1,430	1,545

For **SI:** 1 inch = 25.4 mm, 1 plf = 14.6 N/m.

(a) For framing of the other species: (1) Find specific gravity for species of framing lumber in AF&PA NDS, (2) For staples find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. (3) For nails, find shear value from table above for nail size of actual grade and multiply value by the following adjustment factor: = [1 – (0.5 – SG)], where SG = Specific Gravity of the framing lumber. This adjustment factor shall not be greater than 1.

(b) Fastening along intermediate framing members: Space fasteners a maximum of 12 inches on center, except 6 inches on center for spans greater than 32 inches.

(c) Panels conforming to PS 1 or PS 2.

(d) This table gives shear values for Cases 1 and 2, as shown in IBC Table 2306.3.1. The values shown are applicable to Cases 3, 4, 5 and 6 as shown in IBC Table 2306.3.1, providing fasteners at all continuous panel edges are spaced in accordance with the boundary fastener spacing.

(e) The minimum nominal depth of framing members shall be 3 inches nominal. The minimum nominal width of framing members not located at boundaries or adjoining panel edges shall be 2 inches.

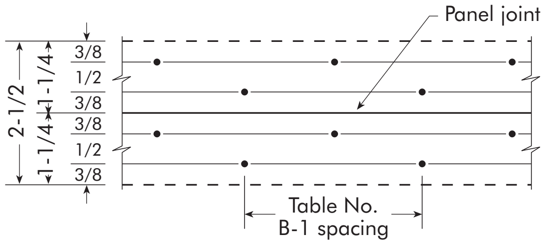
(f) Staples shall have a minimum crown width of 7/16 inch, and shall be installed with their crowns parallel to the long dimension of the framing members.

(g) High load diaphragms shall be subject to special inspection in accordance with IBC Section 1704.6.1.

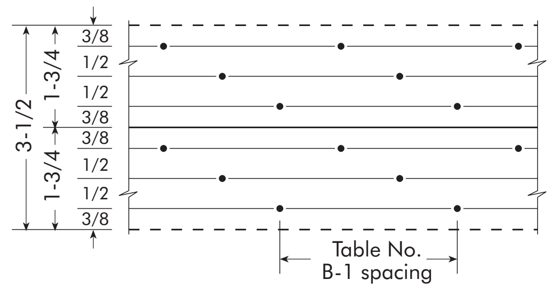
(h) For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

FIGURE B-1

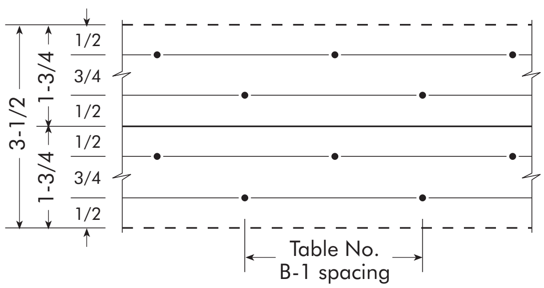
FASTENER PATTERNS FOR USE WITH TABLE B-1



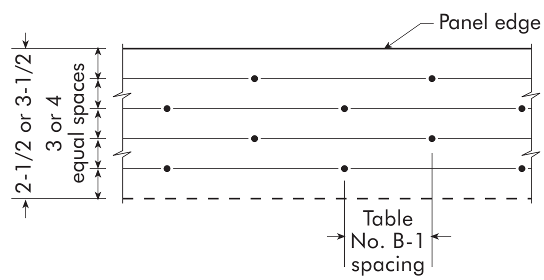
3" Nominal – Two Lines



4" Nominal – Three Lines



4" Nominal – Two Lines



Typical Boundary Fastening
(Illustration of two lines, staggered)

Note: Space panel end and edge joints 1/8 inch. Reduce spacing between lines of nails as necessary to maintain minimum 3/8-inch fastener edge margins. Minimum spacing between lines is 3/8 inch.

APPENDIX C – DIAPHRAGM DEFLECTION ISSUES

The diaphragm deflection equation used in Example 11 is for blocked and uniformly nailed diaphragms. When diaphragms are unblocked and not uniformly nailed, the following is suggested:

Unblocked Diaphragm

"Limited testing of diaphragms [APA, 1952, 1954, 1955, 1967] suggests that the deflection of an unblocked diaphragm at its tabulated allowable shear capacity will be about 2.5 times the calculated deflection of a blocked diaphragm of similar construction and dimensions, at the same shear capacity. If diaphragm framing is spaced more than 24 inches o.c., testing indicates a further increase in deflection of about 20% for unblocked diaphragms (e.g., to 3 times the deflection of a comparable blocked diaphragm). This relationship can be used to develop an estimate of the deflection of unblocked diaphragms."

– SEAOC Blue Book, 1999, §805.3.2

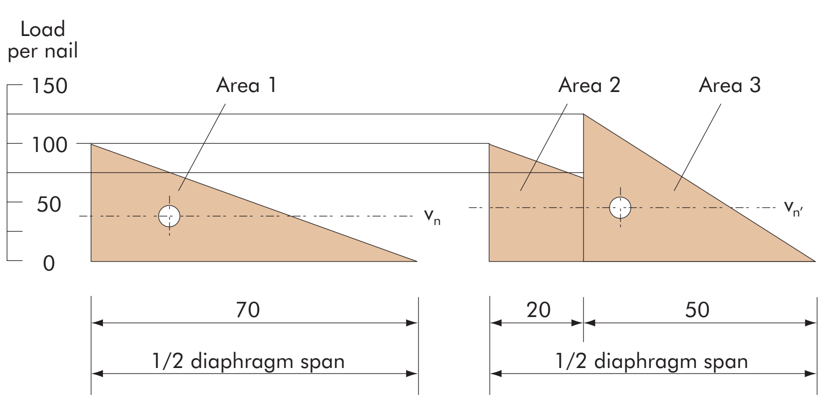
Non-Uniform Nailing

The 0.188 constant in the nail-slip deflection-contribution term is correct when panel edge nailing is the same for the entire length of the diaphragm. When nail spacing becomes less dense near the center of the diaphragm span, the 0.188 constant should increase in proportion to the average load on each nail with non-uniform nailing compared to the average load that would be present if a uniform nail schedule had been maintained (ATC 7, 1981). The new constant can be written as:

$$0.188\left(\frac{v_n'}{v_n}\right)$$

where v_n' is the average non-uniform load per nail and v_n is the average uniform load per nail. Graphically, this can be shown and calculated in terms of areas. In the figure below, area 1 is proportional to v_n and the sum of areas 2 and 3 is proportional to v_n'.

FIGURE C-1



To finish this graphic example, the increased constant would become:

$$0.188\left(\frac{\text{Area}_2 + \text{Area}_3}{\text{Area}_1}\right) = 0.188\left[\frac{0.5(100 + 75)20 + 0.5(125)50}{0.5(100)70}\right] = 0.188 \text{ x } 1.39 = 0.262$$

DIAPHRAGM/SHEAR WALL DESIGN REFERENCES

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