Using Wood Structural Panels for Combined Uplift and Shear Resistance

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Home builders in high-wind areas are increasingly frustrated by the large number of metal hold down devices (hurricane straps) required between floors on exterior walls. In addition to the expense and additional labor required to install such devices, they often experience nailing interference problems with the wood structural panel sheathing and/or siding. This Technical Note provides guidance on the use of wood structural panels for combined shear and uplift. The design of anchor bolts, plate washers, and hold-down devices at the end of the wall or over opening is beyond the scope of this Technical Note.

Wood structural panels can be used to resist both shear and uplift. Shear wall design values are readily available from numerous sources, including the model building codes. Using the same wood structural panels (plywood or oriented strand board) to also resist uplift can be done as illustrated below using engineering principles in the National Design Specification for Wood Construction (NDS) and APA's *Panel Design Specification,* Form D510, with the limits described in Table 1.

Note that as long as the maximum uplift in pounds per lineal foot (plf) is not exceeded for any wood structural panel shear wall in any panel orientation (vertical or horizontal), the full design shear value of the panel may be used. It is left to the designer, as described below, to design the fastener spacing required to develop both the required shear and the uplift capacities.

Checking for uplift alone is relatively straightforward. Ensure that the framing, sheathing, and connections can all resist the uplift. The capacity of each of the elements can be calculated, and the element with the least capacity will determine the overall uplift capacity.

Recommendations for combining shear and uplift resistance in the wood structural panel:

• For analysis purposes, nails used for uplift are considered to resist uplift only and nails used for shear resistance are used for shear only.

• The nails calculated for uplift are provided **in addition** to the nails required for shear. Panel field nailing may be used for uplift resistance, as field nails contribute little to the shear nailing of the panel. In panel regions where shear nailing (panel perimeter) and uplift nailing are both required, designers should follow the minimum nail spacings listed in the shear wall tables for the combined nailing to prevent splitting of the framing members. Where splitting may be an issue and the framing is wide enough (e.g., at panel interior framing), staggered nailing is recommended.

TABLE 1

LIMITS FOR USING COMBINED SHEAR AND UPLIFT CALCULATION PROCEDURE[1][2]

Panel Thickness (in.)	Maximum Uplift (plf)
3/8	1,000
≥ 7/16	1,500

(1) Applies to vertical or horizontal panel orientation.

(2) While a number of variables are associated with computing the uplift load at a given location in any specific structure, note that the 1,000 lb maximum uplift capacity for 3/8"-thick sheathing is sufficient for a large number of residential-type structures subject to wind uplift loads. The actual uplift loads that can be imposed on residential-type structures can be seen in Table 2.2A of ANSI/AF&PA WFCM-2001 or calculated by a design professional.
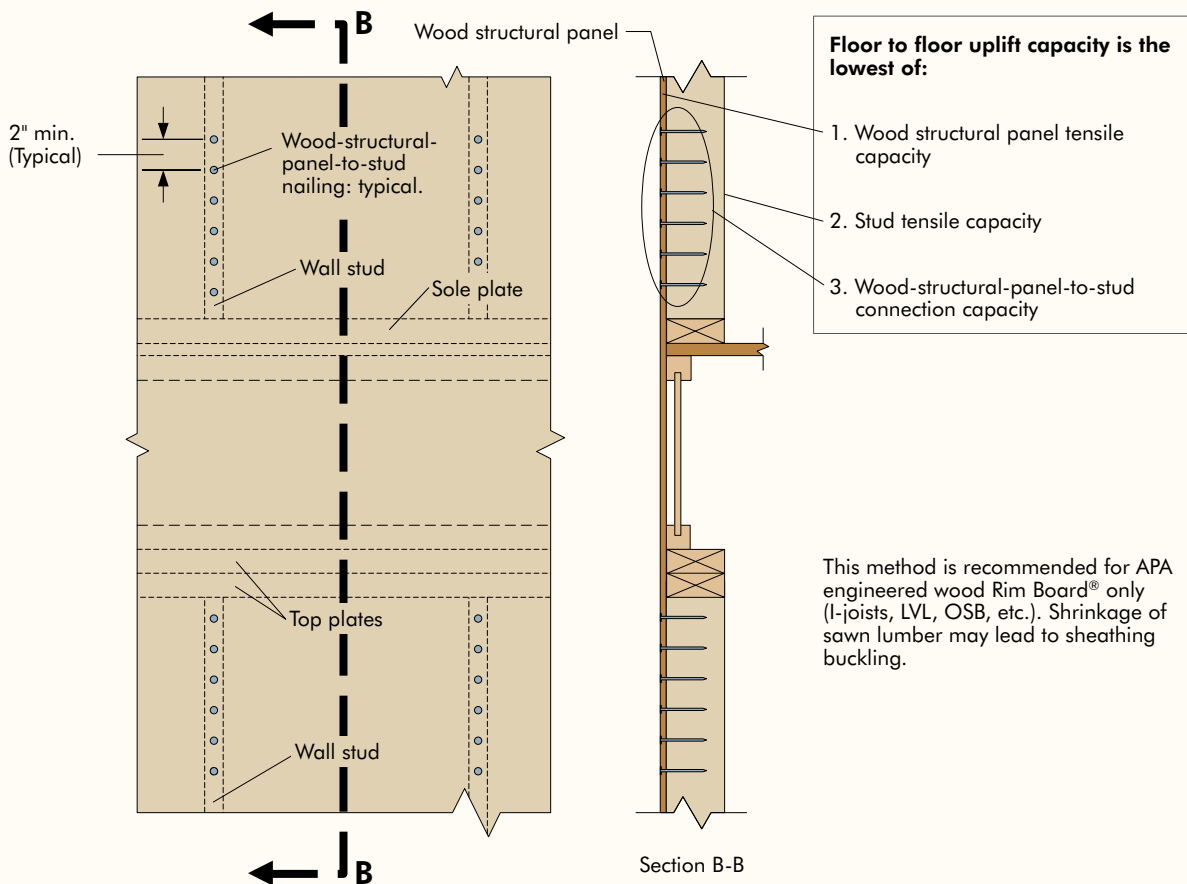
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• Wood structural panels designed for combined shear and uplift forces are subject to these stresses simultaneously. Because the actual shear stress in the sheathing that comprises the shear wall is typically very low (the shear in the shear wall is typically controlled by fastener capacity and not panel shear capacity), within certain limits, the combined stresses in the wood structural panel subjected to shear and uplift may be safely ignored. The limits for which this is true are those given in Table 1.

• Similar solutions are often sought for attachment of the sheathing to the foundation sill plate. The principles expressed above may be used as a part of an engineered solution that compensates for the cross-grain bending and/or tension-perpendicular-to-grain stresses developed by the uplift forces in the sill plate.

• For additional information on the development of the combined shear and uplift interaction equations, contact APA.

Example 1. Compute the maximum wind uplift capacity of the connection shown in Figure 1.

FIGURE 1

**A FLOOR-TO-FLOOR UPLIFT CONNECTION USING WOOD STRUCTURAL PANELS –
TENSION TRANSFERRED BY THE USE OF STUDS**

Floor to floor uplift capacity is the lowest of:

1. Wood structural panel tensile capacity

2. Stud tensile capacity

3. Wood-structural-panel-to-stud connection capacity

This method is recommended for APA engineered wood Rim Board® only (I-joists, LVL, OSB, etc.). Shrinkage of sawn lumber may lead to sheathing buckling.

Section B-B

Note: Only the uplift nailing is shown for clarity. A complete load path may include additional nails for shear transfer. The number of nails shown is for example only. The actual number of nails required will depend on design uplift, panel thickness, nail size, dead-load weight of overlying structure and lumber species.

Assume: Sheathing is 7/16-inch oriented strand board (OSB), APA Rated Sheathing (24/16 Span Rating), oriented such that the tensile stress in the panel is perpendicular to its strength axis; the shear wall must resist 500 lbf/ft of wind uplift force; 6-8d common nails per stud are used as shown; wall framing is Stud grade 2x4 Douglas-fir; and stud spacing is 16-inches o.c.

From Table 1, for a 7/16-inch panel, the effects of combined tensile and shear stresses may be ignored providing the final uplift capacity does not exceed 1,500 lbf/ft.

1. Check the sheathing allowable tensile capacity:
   $F_t A' = F_t A \times C_D$
   = 1,300 lbf/ft (from APA D510, Table 4A) x 1.6 = 2,080 lbf/ft

2. Check the stud allowable tensile capacity:
   $T_{stud} = F_t \times A \times C_D \times C_F$

   Where: F_t = (from NDS Supplement Table 4A) = 450 psi
   A = 1.5 in. x 3.5 in. = 5.25 in.^{2}
   C_D = (load duration factor from NDS Table 2.3.2 for wind) = 1.6
   C_F = (size factor from NDS Table 4A adjustment factors) = 1.1

   T_{stud} = 450 psi x 5.25 in.^{2} x 1.6 x 1.1 = 4,158 lbf/stud (or 4,158 x 12/16 = 3,119 lbf/ft)

3. Check fastener capacity of OSB-to-stud connection:
   $Z' = Z \times C_D \times n$
   Z = (from NDS Table 11Q) = 73 lbf/nail
   C_D = (load duration factor from NDS Table 2.3.2 for wind) = 1.6
   n = number of nails per stud (Given) = 6 nails/stud

   Z' = 73 lbf/nail x 1.6 x 6 nails/stud = 700 lbf/stud (or 700 x 12/16 = 525 lbf/ft)

4. Check with Table 1 to see if combined shear and uplift must be considered:
   Design is currently controlled by the fastener capacity of 6 nails/stud = 525 lbf/ft
   Since 525 lbf/ft < 1,500 lbf/ft, combined shear and tension may be ignored.

   (To fully develop the uplift capacity of the OSB without having to consider combined shear and tension:
   1,500 lbf/ft x 16/12 = 2,000 lbf/stud
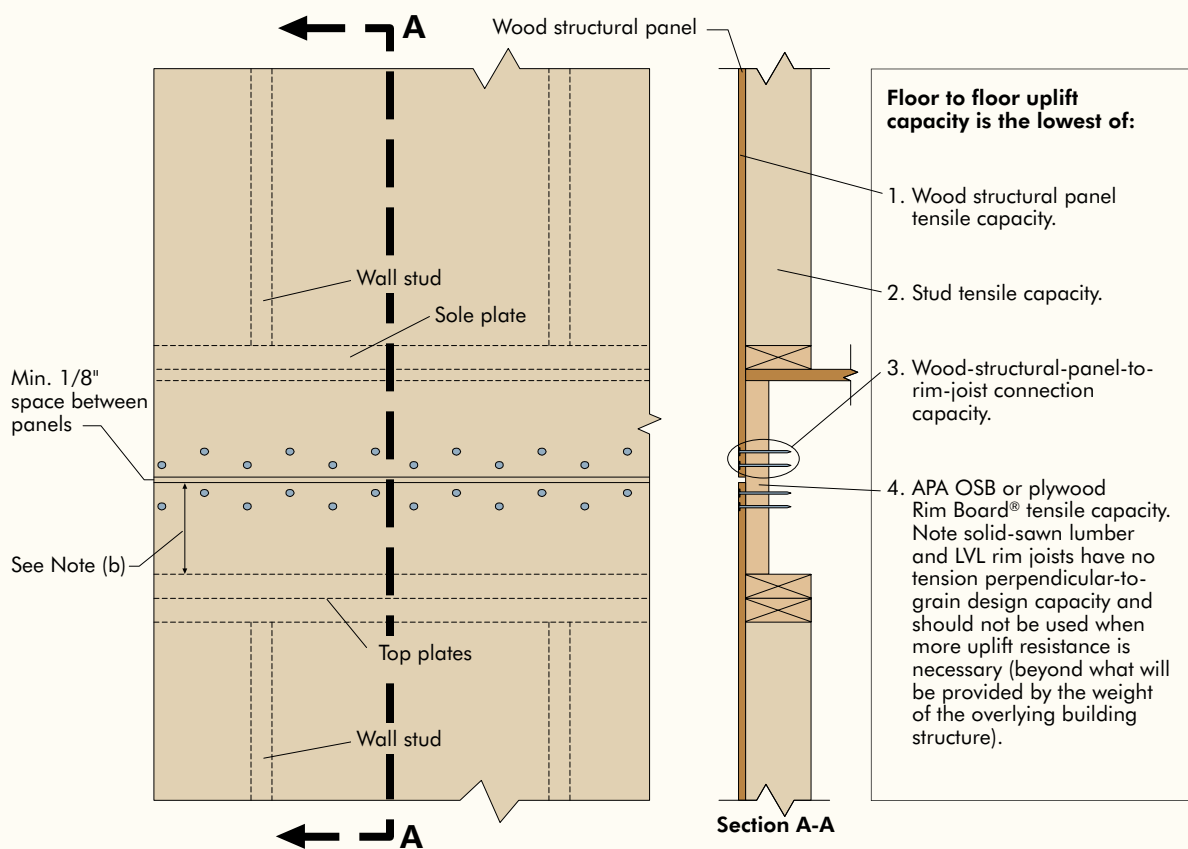   2,000 lbf/stud /(73 lbf/nail x 1.6) = 17.1 nails/stud, use 17 nails/stud)

Note that the calculation in this example considers **only** the nailing required for uplift. At studs located at the ends and edges of panels where nailing is required for shear, the uplift nails must be in addition to those required for shear. In this case, with a 300-plf-shear requirement and 7/16-inch thick wood structural panels fastened with 8d nails, a 4-inch-on-center nail spacing is required for the shear alone. For a stud upon which two shear panels abut, using a 3-inch-on-center spacing for 3 vertical feet above and below the framing interruptions will provide the required 6 additional nails for uplift $\frac{36"}{3"} - \frac{36"}{4"}$ x 2 = 6 extra nails/stud over 3 ft. As an alternative, additional framing members could be used to provide this uplift resistance.

As shear walls begin and end with a double stud, it is relatively simple to add additional nails at shear wall ends.

5. Figure 1 demonstrates an uplift connection for a continuous panel overlapping the rim joist. In some cases, a panel joint may coincide with the rim joist (Figures 2 and 3). Figure 2 is predicated on the rim joist having sufficient tensile capacity, otherwise the detail illustrated in Figure 3 is required.

6. If the tension "splice" is going to be made at the rim joist (as shown in Figure 3), check the following:

   a. Lumber or LVL rim joists should not be used to form a tension splice, as the resulting load on the joist will be in tension perpendicular to the grain. Lumber and LVL do not have design stresses for tension perpendicular to grain. As such, if a lumber or LVL rim joist is present, it should be recessed so that a wood structural panel splice plate can be placed between the sheathing and the recessed rim joist (see Figure 3). If the sheathing used for this splice plate is the same type, orientation, and thickness as that of the wall sheathing for sheathing type, fastener- and tension-capacity calculations of the splice plate are unnecessary. This is because, with the nail anchored in the lumber or LVL rim joist, the nails will have the same lateral capacity in each of the identical wood structural panel elements – the sheathing and the splice plate.

FIGURE 2

**A FLOOR-TO-FLOOR UPLIFT CONNECTION USING WOOD STRUCTURAL PANELS –
TENSION IS TRANSFERRED THROUGH OSB OR PLYWOOD RIM JOIST[a,b]**

Floor to floor uplift capacity is the lowest of:

1. Wood structural panel tensile capacity.

2. Stud tensile capacity.

3. Wood-structural-panel-to-rim-joist connection capacity.

4. APA OSB or plywood Rim Board® tensile capacity. Note solid-sawn lumber and LVL rim joists have no tension perpendicular-to-grain design capacity and should not be used when more uplift resistance is necessary (beyond what will be provided by the weight of the overlying building structure).

Section A-A

a. Only the uplift nailing at the rim joist is shown for clarity. A complete load path may include additional nails for shear transfer and uplift-resisting nailing in the wall studs. The actual number of nails required for uplift and shear resistance will depend on design uplift, panel thickness, nail size, dead-load weight of the overlying structure, and lumber species. This method can be used when uplift nailing is away from shear wall nails that are located in the top plates, sole plates, and studs along the vertical edges of the panels.

b. Figure above shows panel overlap at middle of rim joist. Minimum sheathing-over-rim-board overlap is 1-1/2". Such an overlap is sufficient to support two rows of 10d (0.131" x 2-1/2") nails, each row at 3" on center, while maintaining a minimum panel edge distance of 3/8".

b. If a plywood or OSB APA Performance-Rated Rim Board® is used to form the splice, values for F_tA for "Stress Perpendicular to the Strength Axis" in the APA *Panel Design Specification,* Form D510, Table 4A may be used. For example, for APA 1-inch-thick OSB Rim Board, use the value for 32 oc Span Rated oriented strand board:

From Table 4A of Form D510 for 32 oc Span Rating (Table 5),
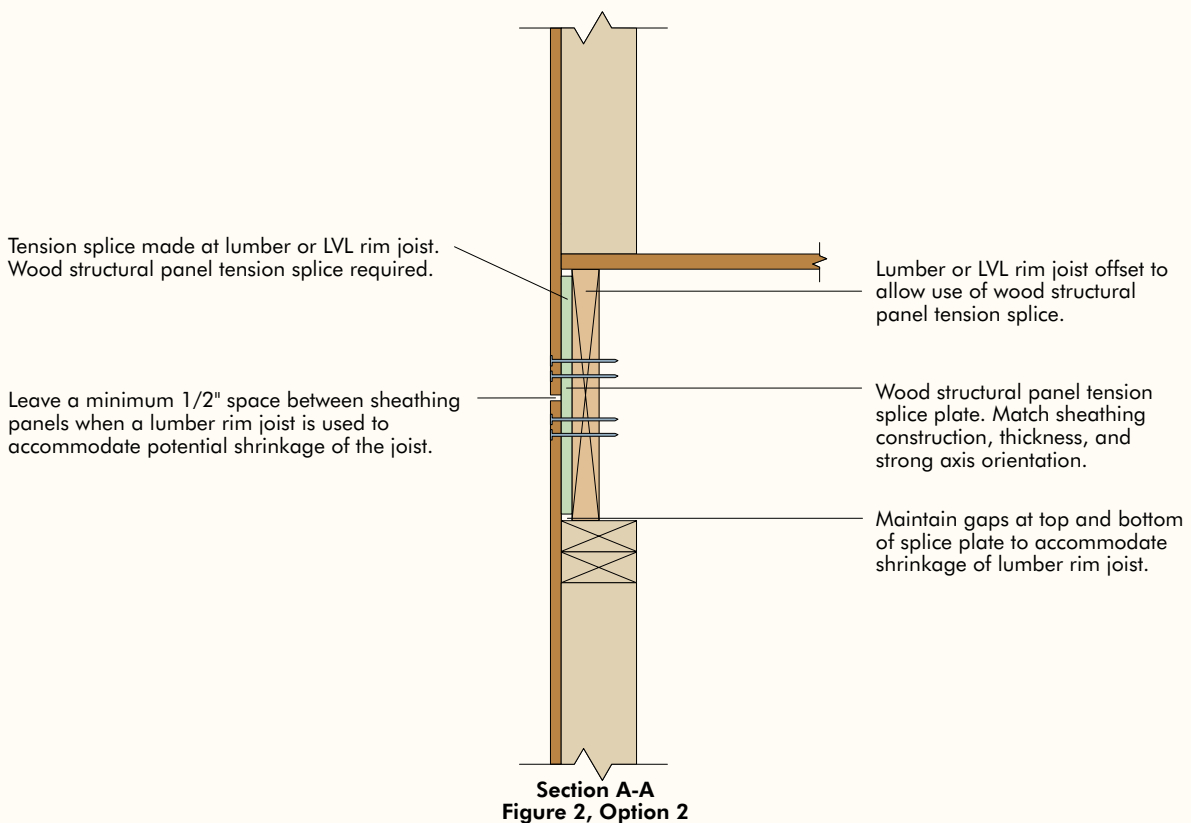F_tA' (from Form D510, Table 4A) = 3,250 lbf/ft
Adjusted for load duration,
$$F_tA' \times C_D = 3{,}250 \text{ lbf/ft} \times 1.6 = 5{,}200 \text{ lbf/ft}$$
$$5{,}200 \text{ lbf/ft} > 525 \text{ lbf/ft, therefore OK}$$

c. Use caution when a wood structural panel splice plate is used in conjunction with a lumber rim joist. As the wood structural panel is an engineered wood product, it is typically dry when received and therefore not as susceptible to shrinkage as is the typical lumber rim joist. For this reason, the wood structural panel splice plate must be cut slightly narrower than the rim joist (1/4" to 1/2") and be centered over the rim joist prior to attachment of the wall sheathing. For the same reason, a similar gap should be maintained between the edges of the outer uplift-resisting sheathing panels where they meet over the rim joist (Figure 3). These gaps may close as the lumber dries to equilibrium moisture content.

FIGURE 3

TENSION SPLICE MADE AT LVL OR LUMBER RIM JOIST, SHOWING OFFSET RIM JOIST TO PERMIT THE USE OF WOOD STRUCTURAL PANEL TENSION SPLICE PLATE

Tension splice made at lumber or LVL rim joist. Wood structural panel tension splice required.

Leave a minimum 1/2" space between sheathing panels when a lumber rim joist is used to accommodate potential shrinkage of the joist.

Lumber or LVL rim joist offset to allow use of wood structural panel tension splice.

Wood structural panel tension splice plate. Match sheathing construction, thickness, and strong axis orientation.

Maintain gaps at top and bottom of splice plate to accommodate shrinkage of lumber rim joist.

Section A-A
Figure 2, Option 2

Note: Only the uplift nailing at the splice is shown for clarity. A complete load path may include additional nails in the panel tension splice plate's upper and lower half, additional nails in the wall studs above and below the panel tension splice, and additional nails for shear transfer. Number of nails shown is for example only. Actual number required will depend on design uplift, panel thickness, nail size, dead-load weight of overlying structure and lumber species.

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