

Wood Structural Panels

Combining Uplift and Shear Resistance

By Ed L. Keith, P.E., APA

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 using engineering principles in the *National Design Specification® for Wood Construction* (NDS®) and the *Panel Design Specification* (APA 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 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

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-inch 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.	
<i>Table 1</i>	

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.

- 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. Tests are ongoing to resolve sill plate attachment issues.
- For additional information on the development of the combined shear and uplift interaction equations, contact APA – The Engineered Wood Association.



Framers install wood structural panel sheathing for vertical continuity to resist both shear and uplift in this multifamily project in Atlanta

continued on next page

Example 1

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

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:

$$F_t A' = F_t A \times C_D = 1,300 \text{ lbf/ft (from APA D510, Table 4A)} \times 1.6 = 2,080 \text{ lbf/ft}$$

2. Check the stud allowable tensile capacity:

$$T_{\text{stud}} = F_t \times A \times C_D \times C_F$$

Where: F_t = (from NDS Supplement Table 4A) = 450 psi

$$A = 1.5 \text{ in.} \times 3.5 \text{ in.} = 5.25 \text{ in.}^2$$

$$C_D = (\text{load duration factor from NDS Table 2.3.2 for wind}) = 1.6$$

$$C_F = (\text{size factor from NDS Table 4A adjustment factors}) = 1.1$$

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

3. Check fastener capacity of OSB-to-stud connection:

$$Z' = Z \times C_D \times n$$

$$Z = (\text{from NDS Table 11Q}) = 73 \text{ lbf/nail}$$

$$C_D = (\text{load duration factor from NDS Table 2.3.2 for wind}) = 1.6$$

$$n = \text{number of nails per stud (Given)} = 6 \text{ nails/stud}$$

$$Z' = 73 \text{ lbf/nail} \times 1.6 \times 6 \text{ nails/stud} = 700 \text{ lbf/stud (or } 700 \times 12/16 = 525 \text{ lbf/ft)}$$

4. Check 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 in the panel may be ignored.

(To fully develop the uplift capacity of the OSB without having to consider combined shear and tension:

$$1,500 \text{ lbf/ft} \times 16/12 = 2,000 \text{ lbf/stud}$$

$$2,000 \text{ lbf/stud} / (73 \text{ lbf/nail} \times 1.6) = 17.1 \text{ nails/stud, use 17 nails/stud}$$

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.

$$\left(\frac{36''}{3''} - \frac{36''}{4''} \right) \times 2 = 6 \text{ extra nails/stud over 3 ft.}$$

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.

- b. If a plywood or OSB APA Performance-Rated Rim Board® is used to form the splice, values for $F_t A$ for "Stress Perpendicular to the Strength Axis" in the APA Panel Design Specification (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 D510 for 32 oc Span Rating (Table 5),

$$F_t A' (\text{from D510, Table 4A}) = 3,250 \text{ lbf/ft}$$

Adjusted for load duration,

$$F_t A' \times C_D = 3,250 \text{ lbf/ft} \times 1.6 = 5,200 \text{ lbf/ft}$$

$$5,200 \text{ lbf/ft} \geq 525 \text{ lbf/ft, therefore OK}$$

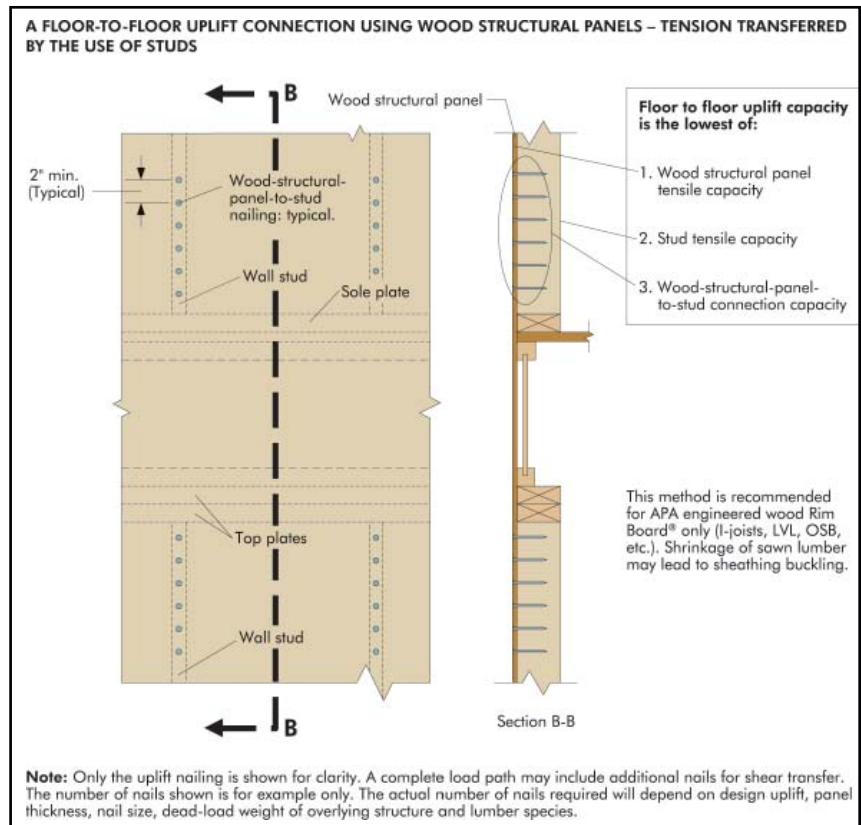


Figure 1

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 ($\frac{1}{4}$ - to $\frac{1}{2}$ -inch) 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. ■

*Edward L. Keith, P.E. is a senior engineer with APA's Technical Services Division. He has over 20 years experience with residential conforming construction and structural engineering.
Email: ed.keith@apawood.org*

APA – The Engineered Wood Association (formerly American Plywood Association) is a nonprofit trade association whose primary mission is product and application testing, research and quality auditing.

Wood structural panels can only perform as designed, provided the proper fasteners are specified and installed accordingly. Two issues to be aware of are nail nomenclature and overdriven fasteners. Surprisingly, there is no uniformity when it comes to nail nomenclature. For example, an 8d common nail does not equal an 8d gun nail. Therefore, to insure the proper fastener is provided, specify the 8d common along with the shank length and shank diameter. This holds true for all nail sizes. What if you want to use staples or another fastener type? Although APA recommends nails you can always reference NER-272 by the International Staple, Nail and Tool Association (ISANTA) for an equivalent.

(Reference: ISANTA NER-272; www.isanta.org; (708-482-8138)

Overdriven fasteners are another important concern. APA recommends:

- If < 20% fasteners are overdriven by $< \frac{1}{8}$ -inch, then they may be ignored.
- If > 20% fasteners are overdriven by $> \frac{1}{8}$ -inch, then:
 - o Prescriptive: add 1 additional fastener for every 2 overdriven
 - o Mechanics based: re-analyze capacity based on average thickness of panel measured from the bottom of the nail head. ($\frac{5}{8}$ -inch panel with fasteners overdriven by $\frac{1}{8}$ -inch = capacity of $\frac{1}{2}$ -inch panel.)

CAUTION!

If the additional nails violate the minimum spacing requirements (3-inch o.c. for 2 inch lumber for splitting), use staples and ignore the original nails.

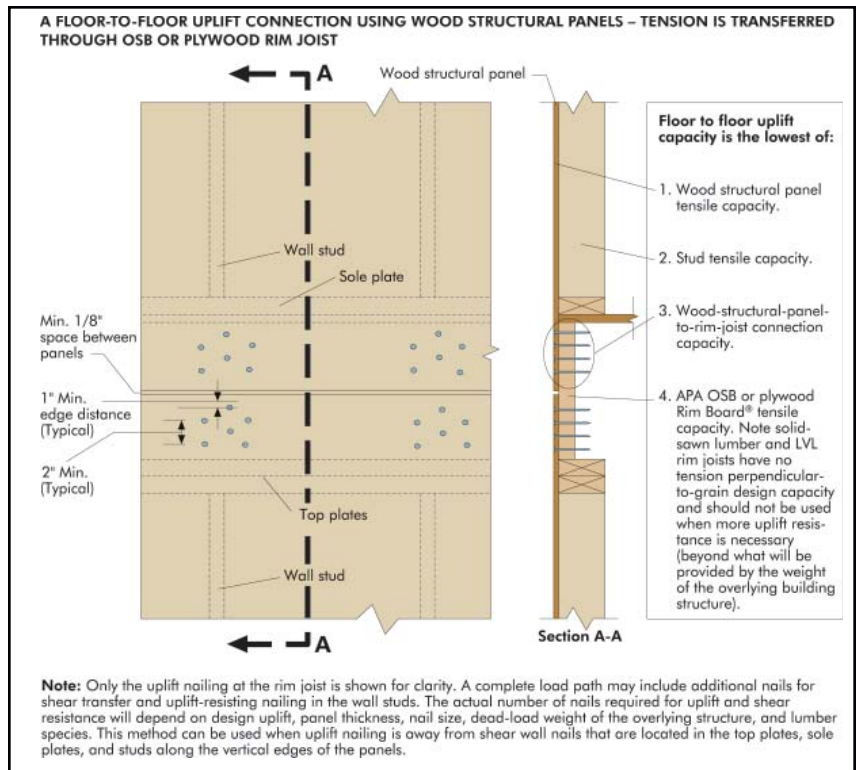


Figure 2

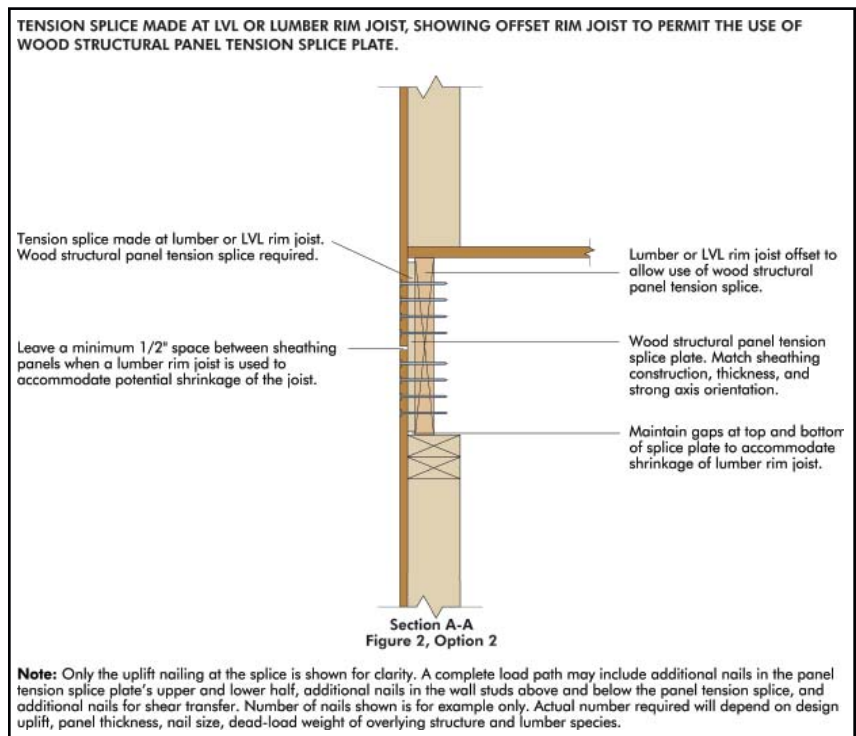


Figure 3

Visit www.structuremag.org for STRUCTURE Magazine subscription information