

Perforated Shear Wall Design

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Introduction

The perforated shear wall (Fig. 1) method is one of several options for the design of wood-frame shear walls in the *2000 International Building Code (IBC)*. Two other methods include: the segmented approach which utilizes full-height shear wall segments each with full end-restraint against overturning (Fig. 2), and the force transfer approach which utilizes strapping to transfer forces around openings. The perforated shear wall method provides one way to account for strength and stiffness of sheathed walls with openings while providing an alternative to strapping and anchors typically required by other methods.

A perforated shear wall design for a two-story building is described in this paper. The example is similar to one developed for the Commentary to the *Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (2000 NEHRP) but is modified to address wind loading using an allowable strength design approach.

A separate example is provided to demonstrate one approach for calculating deflection of a perforated shear wall.

Design Provisions

The perforated shear wall method is included in the *2000 International Building Code (IBC)*, AF&PA's *Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings, Standard Building Code (SBC, 1994 revised)*, and the *2000 NEHRP Recommended Provisions for Seismic Regulations for New Buildings (FEMA, 2000)*.

The basic method for determining design shear capacity of perforated shear walls is the same in each of these documents. However, application limits as well as requirements for shear and uplift forces between wall ends vary. For example, *2000 NEHRP* provisions contain an equation format

for determining shear and uplift forces between wall ends. This equation format provides a convenient means for calculating anchorage requirements based on shear forces resisted by perforated shear walls.

Design Equations

Basic equations for design shear capacity, and calculation of anchorage and chord forces based on story shear forces, are described below.

Design Shear Capacity

The design shear force must not exceed the design shear capacity of a perforated shear wall, $V \leq V_{wall}$. The design shear capacity, V_{wall} , of a perforated shear wall is calculated as:

$$V_{wall} = (v C_o) \sum L_i \quad [1]$$

where:

V_{wall} = design shear capacity of a perforated shear wall, lb.

v = design unit shear capacity of a segmented shear wall, plf

C_o = shear capacity adjustment factor from Table 1 which accounts for the strength reducing effect of openings on shear wall capacity

$\sum L_i$ = sum of lengths of perforated shear wall segments. A perforated shear wall segment is a section of shear wall sheathed full height and which meets minimum aspect ratio requirements in the governing building code. In most cases, this ratio is 3-1/2:1 for shear walls resisting wind and low seismic forces, ft.

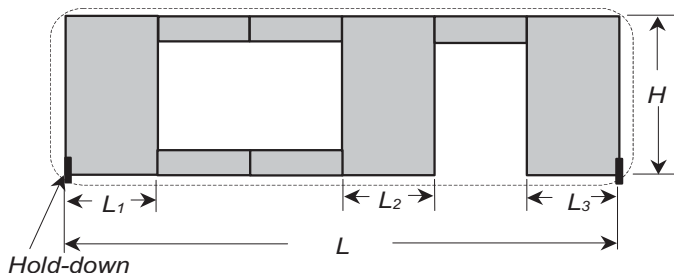


Figure 1.—Perforated shear wall.

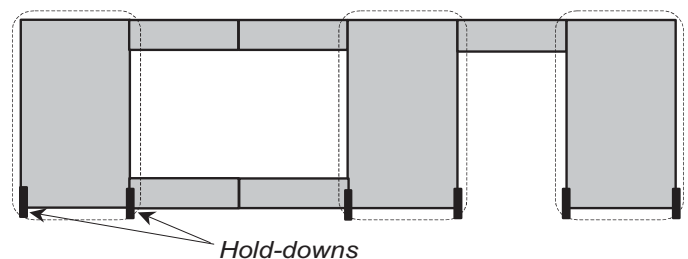


Figure 2.—Segmented shear wall.

Uplift Anchorage Force at Perforated Shear Wall Ends

At each end of a perforated shear wall, end post anchors must be designed for an uplift force, R , due to overturning:

$$R = \frac{Vh}{C_o \sum L_i} \quad [2]$$

where:

- R = uplift anchorage force at perforated shear wall ends, lb.
- V = shear force in perforated shear wall, lb.
- h = shear wall height, ft.

Anchorage Force for In-Plane Shear

The unit shear force, v , transmitted into the top of a perforated shear wall, out of the base of the perforated shear wall at full height sheathing, and used to size collectors (drag struts) connecting perforated shear wall segments, is calculated as:

$$v_{max} = \frac{V}{C_o \sum L_i} \quad [3]$$

where:

- v_{max} = maximum induced unit shear force, plf

Uplift Anchorage Force Between Perforated Shear Wall Ends

Perforated shear wall bottom plates at full height sheathing must be anchored for a uniformly distributed uplift force, t , equal to the unit shear force, v_{max} , calculated in Equation 3.

Tension and Compression Chords

Each end of a perforated shear wall must be designed for a tension force, T , and a compression force, C , equal to the uplift anchorage force, R , calculated in Equation 3. Each end of each perforated shear wall segment must be designed for a compression force, C .

Load Path

Requirements for shear and uplift anchorage and chord forces are based on story shear forces. Elements resisting forces contributed by multiple stories should be designed for the sum of forces contributed by each story. A continuous load path to the foundation needs to be maintained and include effects of gravity and other forces.

Other Requirements

In addition to requirements for design shear capacity, anchorage forces, and chord forces, limitations outlined in *2000 NEHRP* are also applicable:

- a. A perforated shear wall segment shall be located at each end of a perforated shear wall. Openings shall be permitted to occur beyond the ends of the perforated shear wall. However, the length of such openings shall not be included in the length of the perforated shear wall.
- b. Where out of plane offsets occur, portions of the wall on each side of the offset shall be considered as separate

Table 1.—Shear capacity adjustment factor, C_o .

Wall height (h)	Maximum opening height ^a				
	$h/3$	$h/2$	$2h/3$	$5h/6$	h
8 ft. 0 in.	2 ft. 8 in.	4 ft. 0 in.	5 ft. 4 in.	6 ft. 8 in.	8 ft. 0 in.
10 ft. 0 in.	3 ft. 4 in.	5 ft. 0 in.	6 ft. 8 in.	8 ft. 4 in.	10 ft. 0 in.
Percent full-height sheathing ^b	Shear capacity adjustment factor				
10%	1.00	0.69	0.53	0.43	0.36
20%	1.00	0.71	0.56	0.45	0.38
30%	1.00	0.74	0.59	0.49	0.42
40%	1.00	0.77	0.63	0.53	0.45
50%	1.00	0.80	0.67	0.57	0.50
60%	1.00	0.83	0.71	0.63	0.56
70%	1.00	0.87	0.77	0.69	0.63
80%	1.00	0.91	0.83	0.77	0.71
90%	1.00	0.95	0.91	0.87	0.83
100%	1.00	1.00	1.00	1.00	1.00

^a The maximum opening height is taken as the maximum opening clear height in a perforated shear wall. Where areas above and below an opening remain unsheathed, the height of the opening shall be defined as the height of the wall.

^b The percent of full height sheathing is calculated as the sum of lengths of perforated shear wall segments divided by the total length of the perforated shear wall including openings.

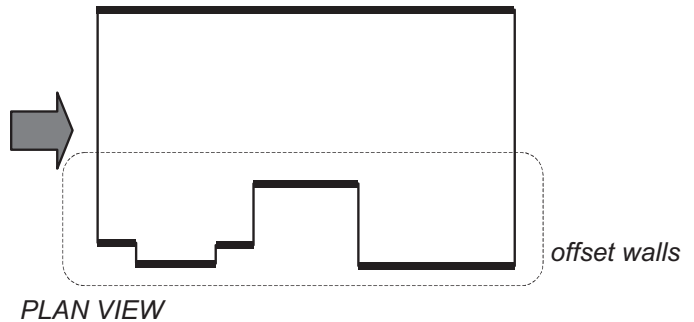


Figure 3.—In-plane versus offset walls.

perforated shear walls. Offset walls are shown as heavy lines in Figure 3.

- c. Collectors for shear transfer shall be provided through the full length of the perforated shear wall.
- d. A perforated shear wall shall have uniform top of wall and bottom of wall elevations. Perforated shear walls not having uniform elevations shall be designed by other methods. One example of a wall with non-uniform top and bottom elevations is the stepped wall as shown in Figure 4.
- e. Perforated shear wall height, h , shall not exceed 20 ft.
- f. The allowable tabulated capacity set forth in the *2000 IBC* for wood structural panel shear walls should not ex-

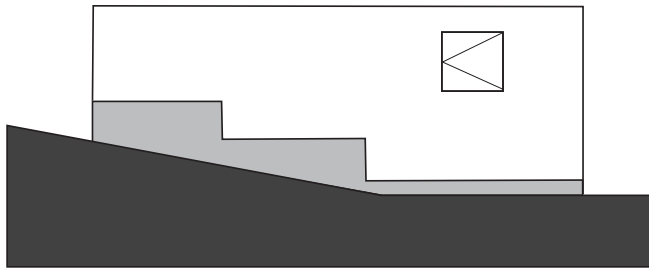


Figure 4.—Wall with non-uniform top and bottom of wall elevation (stepped wall).

ceed 490 plf. For wind design, the allowable capacity can be taken as 1.4 times 490 plf or 686 plf.

Discussion — Maximum Unit Shear

Shear is not distributed uniformly to perforated shear wall segments within the wall. Segments with greater end restraint develop greater shear than segments with less end restraint. Tests of shear wall segments with varying levels of end restraint (Chun and Karacabeyli 2000 and Salenikovich 2000) verify the influence of end restraint on shear capacity. Lower bound strengths are attributed to wall segments without end restraint and upper bound strengths are attributed to wall segments with full end-restraint.

The unit shear force determined by Equation 3 represents maximum unit shear force, v_{max} , developed in any perforated shear wall segment. The value of v_{max} is associated with the shear force developed by any perforated shear wall segments having full end-restraint (such as those at the ends of the perforated shear wall). Use of v_{max} to size shear and uplift anchorage between wall ends and collectors between perforated shear wall segments is required in lieu of a more complicated analysis of the actual distribution of shear within a perforated shear wall. This conservatism en-

sures that the capacity of each perforated shear wall segment can be developed without being limited by collector element capacity or bottom plate attachment for shear and uplift. One apparent result of the conservatism is that actual anchorage requirements for shear are in excess of the design shear force, V , resisted by the perforated shear wall.

Average unit shear in perforated shear wall segments equal to $V/\sum L_i$ is not used to size shear and uplift anchorage or collector elements because it underestimates forces that may develop in a particular perforated shear wall segment.

Discussion — Uplift Between Wall Ends

For perforated shear wall segments between wall ends, a uniform uplift anchorage force is specified for attachment of bottom plates to elements below. Designing distributed anchorage for v_{max} provides resistance to overturning of perforated shear wall segments between wall ends. Alternatively, concentrated anchorage for uplift at ends of perforated shear wall segments, to provide equivalent moment resistance, satisfies the intended purpose: to keep the segment from overturning by holding the bottom plate to elements below (Fig. 5).

In tests of long shear walls with openings (Dolan and Johnson 1996), anchor bolts resisting overturning were located within 12 in. from ends of perforated shear wall segments. It is also acceptable to restrain studs by strapping since bottom plates in turn are held down.

The conservatism of the uplift anchorage requirement between wall ends is seen when evaluating longer perforated shear wall segments. In such cases, the concentrated force to restrain the bottom plate (based on moment resistance equivalent to that provided by specified uniformly distributed uplift anchorage force) at ends of the perforated shear wall segment exceeds the force required to develop the maximum induced unit shear force, v_{max} . Recognizing

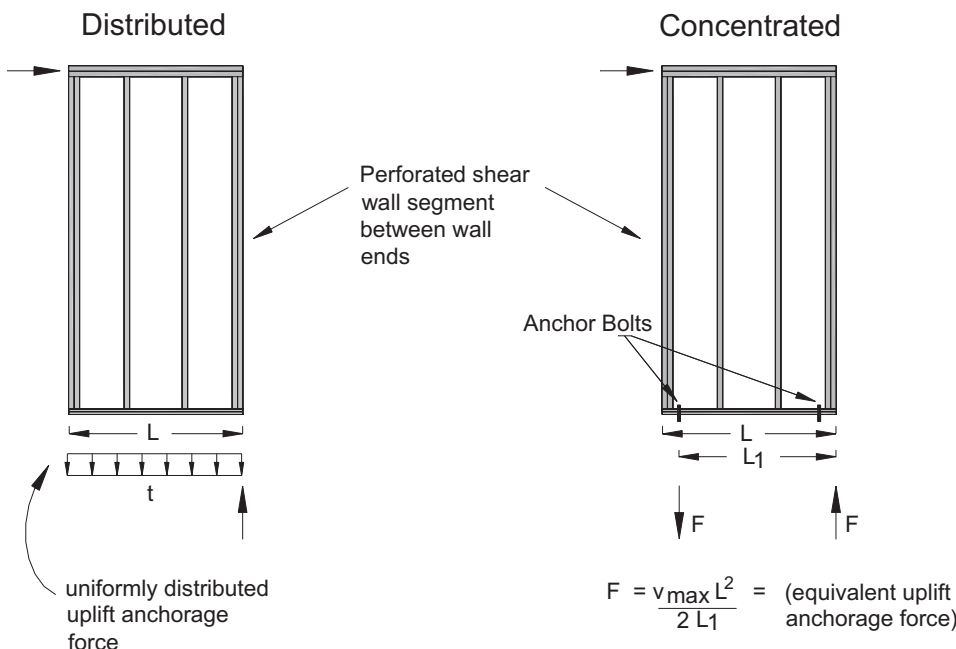
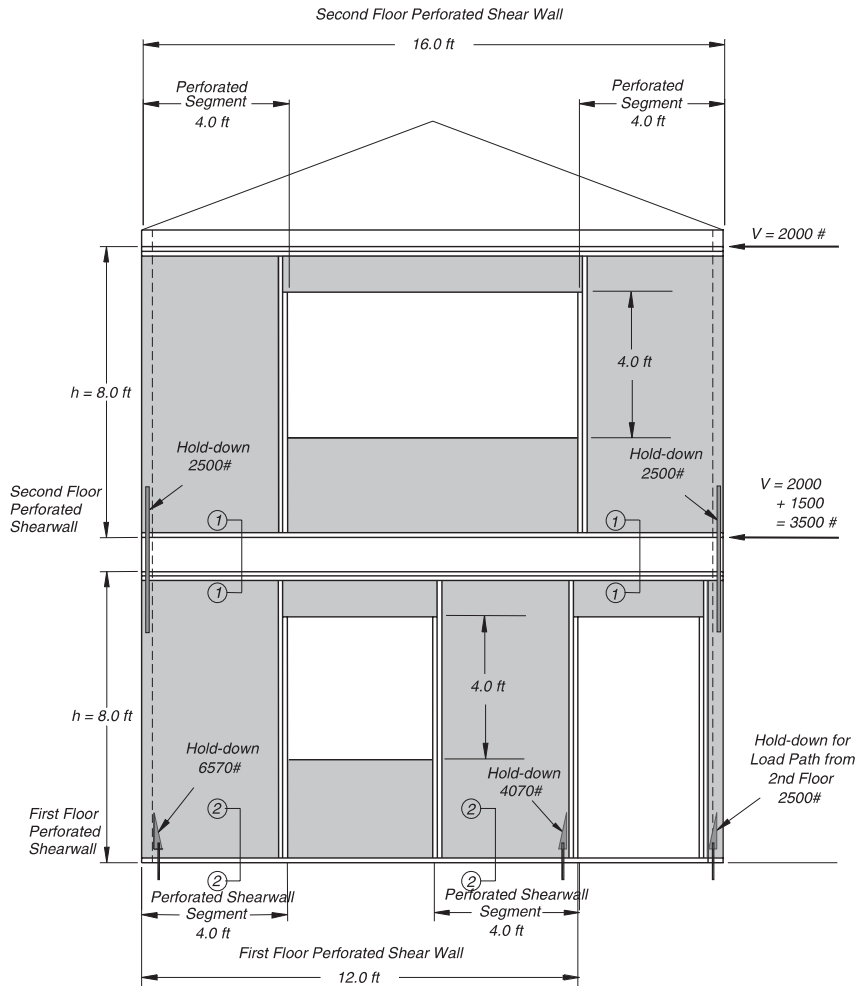


Figure 5.—Uplift anchorage force between wall ends.

Figure 6.—Two-story building with perforated shear walls.



that induced shear will not exceed v_{max} , it is acceptable to limit the concentrated uplift anchorage force at ends of perforated shear wall segments, due to story shear, to $v_{max}h$ where h is the wall height. This force will match the uplift anchorage force, R , from Equation 2.

Example — Two-Story Building

A design example involving a simple two-story building (Fig. 6) demonstrates application of the perforated shear wall method. Design shear capacity, shear and uplift between wall ends, and end post forces are calculated. Only lateral loads due to wind are considered. Building dead load and wind uplift forces are not included. Once forces are determined, two detailing options are considered. Configuration A uses a continuous rim joist at the second floor level sized to resist localized uplift and compression forces along the base of the wall due to story shear forces. Configuration B considers a condition where there is blocking between joists (e.g., floor framing runs perpendicular to the perforated shear wall).

Second Floor Wall

Design Shear Capacity:

Percent full-height sheathing

$$= (4 \text{ ft.} + 4 \text{ ft.})/16 \text{ ft.} \times 100 = 50\%$$

Maximum opening height ratio

$$= 4 \text{ ft.}/8 \text{ ft.} = 0.50$$

Shear capacity adjustment factor, C_o , from Table 1

$$= 0.80$$

$$V_{wall} = (v C_o) \sum L_i$$

$$= (365 \text{ plf})(0.80)(8 \text{ ft.}) = 2,336 \text{ lb.}$$

$$2,336 \text{ lb.} > 2,000 \text{ lb.} \quad \text{OK}$$

Note that $v = 365 \text{ plf}$ is for 15/32-in. rated sheathing with 8d common nails (0.131 by 2.5 in.) at 6-in. perimeter spacing resisting wind load (365 plf is obtained by multiplying the IBC Table 2306.4.1 value by the wind increase factor, or $260 \text{ plf} \times 1.4$).

Uplift Anchorage Force at Shear Wall/Ends:

$$R = \frac{Vh}{C_o \sum L_i} = \frac{(2,000 \text{ lb.})(8 \text{ ft.})}{(0.80)(8 \text{ ft.})} = 2,500 \text{ lb.}$$

Anchorage Force for In-Plane Shear:

$$v_{max} = \frac{V}{C_o \sum L_i} = \frac{(2,000 \text{ lb.})}{(0.80)(8 \text{ ft.})} = 313 \text{ plf}$$

Anchorage Force for Uplift, t , Between Wall Ends:

$$t = v_{max} = 313 \text{ plf}$$

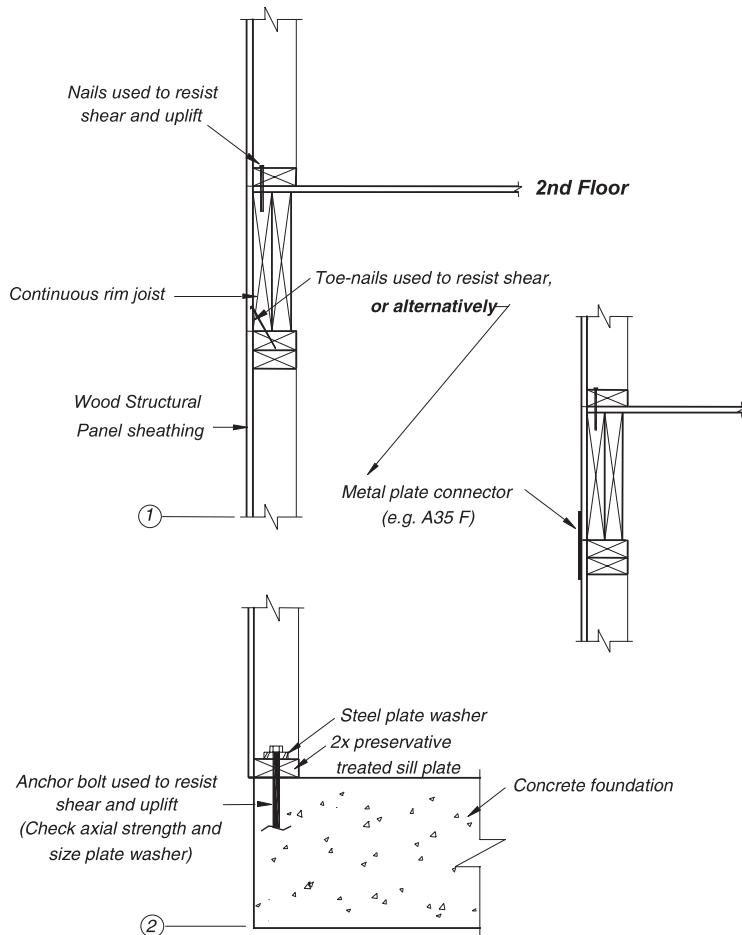


Figure 7.—Configuration A detail—continuous rim joist.

Tension Chord Force, T , and Compression Chord Force, C , at Each End of a Perforated Shear Wall Segment:

$$C = T = R = 2,500 \text{ lb.}$$

First Floor Wall

Design Shear Capacity:

Percent full-height sheathing

$$= (4 \text{ ft.} + 4 \text{ ft.}) / 12 \text{ ft.} \times 100 = 67\%$$

Maximum opening height ratio

$$= 4 \text{ ft.} / 8 \text{ ft.} = 0.50$$

Shear capacity adjustment factor, C_o , from Table 1

$$= 0.86$$

$$V_{wall} = (v C_o) \sum L_i$$

$$= (530 \text{ plf})(0.86)(8 \text{ ft.}) = 3,646 \text{ lb.}$$

$$3,646 \text{ lb.} > 3,500 \text{ lb.} \quad \text{OK}$$

Note that $v = 530$ plf is for 15/32-in. rated sheathing with 8d common nails (0.131 by 2.5 in.) at 4-in. perimeter spacing resisting wind load (530 plf is obtained by multiplying the IBC Table 2306.4.1 value by the wind increase factor, or 390 plf \times 1.4).

Uplift Anchorage Force at Shear Wall/Ends:

$$R = \frac{Vh}{C_o \sum L_i} = \frac{(3,500 \text{ lb.})(8 \text{ ft.})}{(0.86)(8 \text{ ft.})} = 4,070 \text{ lb.}$$

When Maintaining Load Path from Story Above:

$$R = R_{\text{from second floor}} + R_{\text{from first floor}} \\ = 2,500 \text{ lb.} + 4,070 \text{ lb.} = 6,570 \text{ lb.}$$

Anchorage Force Force for In-Plane Shear:

$$v_{max} = \frac{V}{C_o \sum L_i} = \frac{(3,500 \text{ lb.})}{(0.86)(8 \text{ ft.})} = 509 \text{ plf}$$

Anchorage Force for Uplift, t , Between Wall Ends:

$$t = v_{max} = 509 \text{ plf}$$

Uplift, t , can be cumulative with 313 lb. from the story above to maintain load path. Whether this occurs depends on detailing for transfer of uplift forces between wall ends.

Tension Chord Force, T , and Compression Chord Force, C , at Each End of a Perforated Shear Wall and Compression Chord Force, C , at Each End of a Perforated Shear Wall Segment:

$$C = T = R = 4,070 \text{ lb.}$$

When maintaining load path from story above,

$$C = 4,070 \text{ lb.} + 2,500 \text{ lb.} = 6,570 \text{ lb.}$$

Load Path - Configuration A Detail

Configuration A detailing (Fig. 7) uses a continuous rim joist at the second floor.

The rim joist is sized to resist forces from perforated shear wall segments between wall ends. For the second story perforated shear wall shown in Figure 8, a compression force, C , and uplift force, t , induce moment and shear in the rim joist. Continuity of load path for uplift and compression is maintained by adequate sizing of the rim joist for induced forces and adequate attachment at ends. For this example, dead load and additional compression reactions from wall studs below are ignored to simplify boundary conditions and assumed loading. Nails in the second story wall bottom plate to rim joist connection resist overturning of perforated shear wall segments between wall ends. Nails can be uniformly spaced to meet the specified uplift force (Equation 3) or alternatively, a concentrated nail schedule at ends of segments can be used to provide equivalent moment resistance to the specified uplift force. Other fasteners such as lag screws or bolts are an alternative to nails loaded in withdrawal.

Second story hold-downs at wall ends are sized for the force specified in Equation 2.

Transfer of shear from the second story to the first story is by nails in the wall bottom plate to rim joist connection and toe nails in the rim joist to wall top plate connection. First floor anchor bolts provide for transfer of shear and uplift forces between wall ends to the foundation. Plate washers and anchor bolts are sized to provide adequate bearing area and capacity to resist uplift forces. In this case, uplift between wall ends is not additive with second story uplift between wall ends due to design of the second story rim joist. First floor hold-downs are sized to resist uplift forces and in-

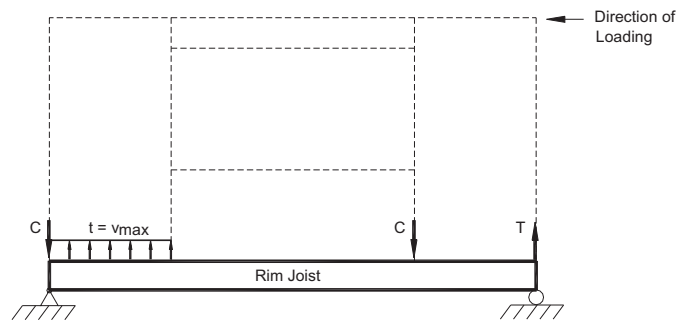


Figure 8.—Uplift and compression forces for rim joist design.

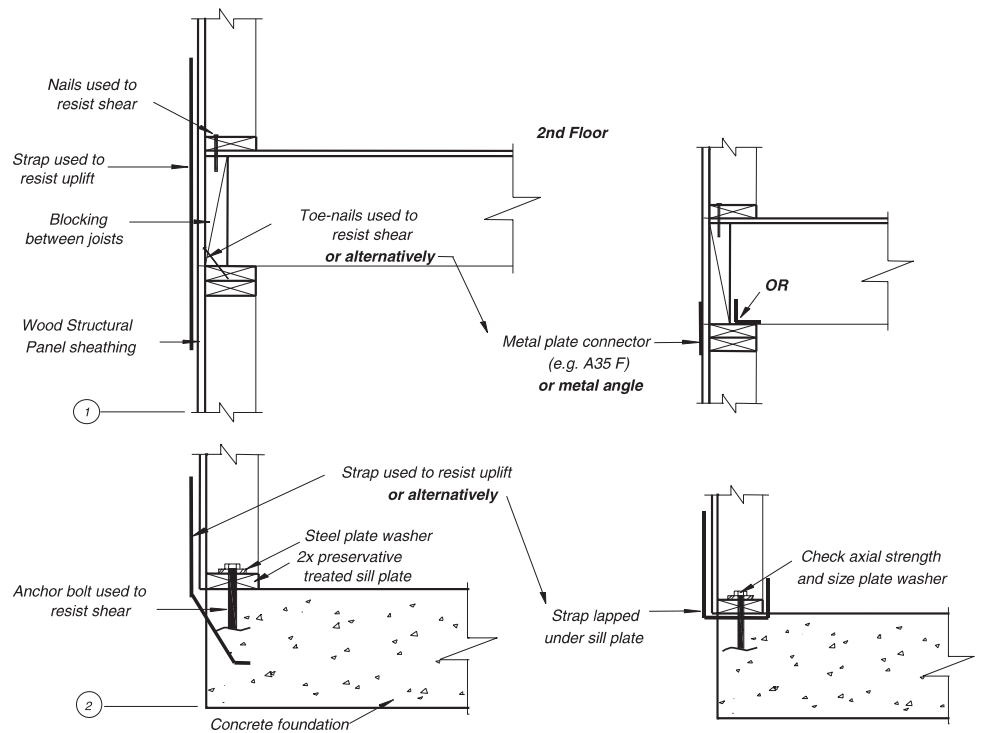
clude uplift forces from the story above, where applicable, to maintain a load path to the foundation. For simplicity, floor platform height is ignored in calculation of load path for overturning.

Load Path - Configuration B Detail

Configuration B detailing (Fig. 9) addresses a condition where a continuous rim joist is not provided. In Configuration B, floor framing runs perpendicular to the shear wall with blocking between floor framing members.

Nails in second story wall bottom plate-to-blocking connections and toe-nails in rim joist-to-wall top plate connections transfer in-plane shear forces. Transfer of shear to the foundation is by anchor bolts in the first story wall bottom plate-to-concrete connection. Transfer of uplift forces between wall ends, from the second story to the first story, is

Figure 9.—Configuration B detail—blocking between joists.



by metal strapping. Load path for uplift between wall ends is maintained by strapping into the foundation or lapping the strap around the first story bottom plate. When the strap is lapped around the bottom plate, the anchor bolt and plate washer must be sized to resist induced forces. First floor hold-downs are sized to resist uplift and include uplift forces from the story above, where applicable, to maintain a load path to the foundation. For simplicity, floor platform height is ignored in calculation of load path for overturning.

Discussion - Configuration A and B Detail

In Configuration A and B, fastening for shear and uplift between wall ends is provided over the length of full-height sheathed wall sections. Fastening between full-height segments will be controlled by minimum construction fastening requirements. For bottom plates on wood platforms this would typically require one 16-penny common nail at 16 in. on center. In some cases, it may be preferable for construction convenience to extend a single bottom plate fastening schedule across the entire length of the perforated shear wall rather than require multiple fastening schedules.

Load path for uplift between wall ends is simplified in Configuration A by design of the rim joist to resist induced forces. Detailing for shear and uplift between wall ends can be accomplished with standard connectors between the wall bottom plate and rim joist. In Configuration B, strapping is used to restrain bottom plate uplift between wall ends. Anchor bolts and plate washers sized for accumulated uplift forces maintain load path to the foundation for uplift forces between wall ends.

Shear Wall Deflection

One method for calculating deflection of a perforated shear wall is shown in Figure 10. The calculation procedure accounts for the presence of openings within the wall and is comparable to the process used to calculate deflection of a segmented shear wall.

Summary

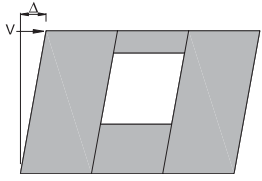
A two-story example demonstrates one application for use of perforated shear walls. Methods for determining perforated shear wall design capacity and bottom plate anchorage requirements, based on story shear forces, are shown. In the example, sufficient strength to resist applied loads is provided by the perforated shear wall without the use of typical strapping and anchors required by other methods. Instead, anchorage requirements for bottom plates in a perforated shear wall provide an option to typical strapping and anchors that may be difficult to install.

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Example—Perforated Shear Wall Deflection



The deflection of a perforated shear wall can be calculated from standard equations for deflection of wood structural panel shear walls. Using IBC Equation 23-2, the deflection of a wood structural panel shear wall can be calculated as:

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

where:

- A = Area of boundary element cross section, in.²
- b = Wall width, ft.
- d_a = Deflection due to anchorage details, in.
- E = Modulus of Elasticity of boundary element, psi
- e_n = Deformation of mechanically fastened connections, in.
- Gt = Rigidity through the thickness of wood structural panel, lb./in. of panel depth
- h = Wall height, ft.
- v = Maximum unit shear due to design loads at the top of the wall, lb./ft.

For the first floor perforated shear wall in Figure 5, standard inputs for variables of A , d_a , E , e_n , Gt , and h apply. Maximum unit shear, v , is taken as $v_{max} = V/[\sum L_i] = 509 \text{ plf}$ and wall width, b , is taken as $b = \sum L_i = 8 \text{ ft}$.

Assuming deflection due to anchorage is 0.125 in. and boundary elements (end posts) consisting of two 2x4 Douglas Fir, No. 2 grade lumber, deflection of the perforated shear wall is calculated as follows:

$$\Delta = \frac{8(509)8^3}{(1,600,000)(10.5)(8)} + \frac{(509)8}{38,000} + 0.75(8)(0.049) + 0.125$$

$$\Delta = 0.016 + 0.107 + 0.294 + 0.125 = 0.54 \text{ in.}$$

Figure 10.—Perforated shear wall deflection.

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