# APPENDIX A
# ANALYSIS OF THE LATERAL FORCE RESISTING SYSTEM OF THE DEMONSTRATION HOUSE LOCATED IN BEAUFORT, SC (CASE STUDY I)

## LOAD CALCULATION

At the time of the demonstration house construction, the Standard Building Code (SBC) was the governing building code in Beaufort County of the State of South Carolina. Therefore, this analysis is performed according to the SBC. The 1999 SBC permits the use of ASCE 7 for calculation of design wind loads. Therefore, ASCE 7-98 is used to determine design wind loads. Seismic analysis is not included in these calculations.

The design wind loads are determined using Method 2 - Analytical Procedure (Section 6.5). The building is beyond the scope of Method 1 - Simplified Procedure (Section 6.4) because it has roof slopes greater than $10^o$. The procedure for low-rise buildings is used.

1. Basic wind speed, V, and wind directionality factor, $K_d$.
   - V = 130 mph (Figure 6.1)
   - $K_d$ = 0.85 main wind force resisting system (Table 6.6)

2. Importance factor, I.
   - I = 1.0 (Table 6-1, Category I and V>100 mph)

3. Exposure category and velocity pressure coefficient, $K_z$ or $K_h$.
   Exposure C is assumed because the terrain representative of Exposure B does not prevail in the west directions by 1,500 feet as required by section 6.5.6.1.
   - $K_z$ = 0.85 (Table 6.5, z < 15)
   - $K_h$ = 0.92 (Table 6.5, h = 22), the same $K_h$ is used with all roof pressures
   - $K_z$ = 0.87 (Table 6.5, z = 16.5 - gable end of the front wall)

4. Enclosure classification.
   The building is classified as partially enclosed because the building is located in the wind borne debris region (basic wind speed greater than 120 mph), and impact resistant glazing was not used. This classification does not affect lateral building load magnitude.

5. External pressure coefficients $GC_{pf}$, (Figure 6-4).

   The coefficients are determined for roof angle range of 30-45 degrees
   - Wall windward
     - $GC_{pf}$ = 0.56
   - Roof windward
     - $GC_{pf}$ = 0.21
   - Roof leeward
     - $GC_{pf}$ = -0.43
   - Wall leeward
     - $GC_{pf}$ = -0.37

6. Velocity pressure $q_h$.

$$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I = 0.00256\ (0.85)\ (1.0)\ (0.85)\ (130)^2\ (1.0) = 31.3\ \text{lb/ft}^2$$
$$q_h = 0.00256\ K_h\ K_{zt}\ K_d\ V^2\ I = 0.00256\ (0.92)\ (1.0)\ (0.85)\ (130)^2\ (1.0) = 33.8\ \text{lb/ft}^2$$
$$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I = 0.00256\ (0.87)\ (1.0)\ (0.85)\ (130)^2\ (1.0) = 32.0\ \text{lb/ft}^2$$

7. Design wind load p.

$p = q\ GC_p - q_i\ (GC_{pi})$

Walls

Windward

$p = (31.3)(0.56) = 17.5\ \text{lb/ft}^2$

$p = (32.0)(0.56) = 17.9\ \text{lb/ft}^2$ – gable end

Leeward

$p = (33.8)(-0.37) = -12.5\ \text{lb/ft}^2$

Roof

Windward

$p = (33.8)(0.21) = 7.1\ \text{lb/ft}^2$

Leeward

$p = (33.8)(-0.43) = -14.5\ \text{lb/ft}^2$

8. Forces

The areas are estimated using AutoCAD software package from the drawings created based on building plans provided by the wall panel supplier.

<u>NS direction</u>

Projected roof area including gable ends

$A = 1{,}289\ \text{ft}^2$

Projected roof area without gable ends

$A = 1{,}289 - (2)(141) = 1{,}007\ \text{ft}^2$

Wall area including gable ends

$A = (92)(9.1/2) + 282 = 701\ \text{ft}^2$

Roof load

$F = (1{,}007)(14.5 + 7.1) = 21{,}751$ lb

Wall load

$F = (701)(17.9 + 12.5) = 21{,}310$ lb

Total load in NS direction

**F = 21,751 + 21,310 = 43,061 lb**

<u>EW direction</u>

Projected roof area

West side: $A = 222 + 412 = 634\ \text{ft}^2$

East side: $A = 175 + 477 = 652\ \text{ft}^2$ - governs

Projected roof area of the hidden surface
A = 164 $ft^2$

Wall area
A = (54.6)(9.1/2) = 248 $ft^2$
Additional wall area from the hidden surface
A = (12)(9.1/2) = 55 $ft^2$

Roof load
F = (652 + 164)(7.1 + 14.5) = 17,626 lb
Wall Load
F = (248 + 55)(17.5 + 12.5) = 9,090 lb

Total load in EW direction
**F = 17,626 + 9,090 = 26,716 lb**

## SHEAR WALL ANALYSIS

The rigid diaphragm method with torsion was used to distribute the total lateral load between the shear walls. The use of the rigid diaphragm method was justified due to the following: (1) ceiling diaphragm and roof diaphragm were sheathed with structural panels and interconnected at the joist-to-rafter joints with screws creating a box-type structure and (2) fifty percent of the total wind load (North-South direction) was received by roof diaphragm rather than walls. The reader is referred to *Model Guidelines for Design, Fabrication, and Installation of Engineered Panelized Walls* (HUD 2002) and Appendix C of this document for a more detailed description of the rigid diaphragm method.

Figure A1 shows a schematic plan of the building including location of shear walls. All exterior walls and garage walls are shear walls. The interior partitions are not shown, and their significant lateral strength contribution is ignored in this analysis as is typical to current design practice.

The shear wall capacity was calculated using the perforated shear wall method (see *Model Guidelines for Design, Fabrication, and Installation of Engineered Panelized Walls* (HUD 2002) for method description). The allowable shear design value of 311 lb/ft was used (Structural Engineering Bulletin No. 1072, HUD, 1997). Table A1 summarizes results of the shear wall design in the North-South (NS) direction. The allowable stress design format was used.

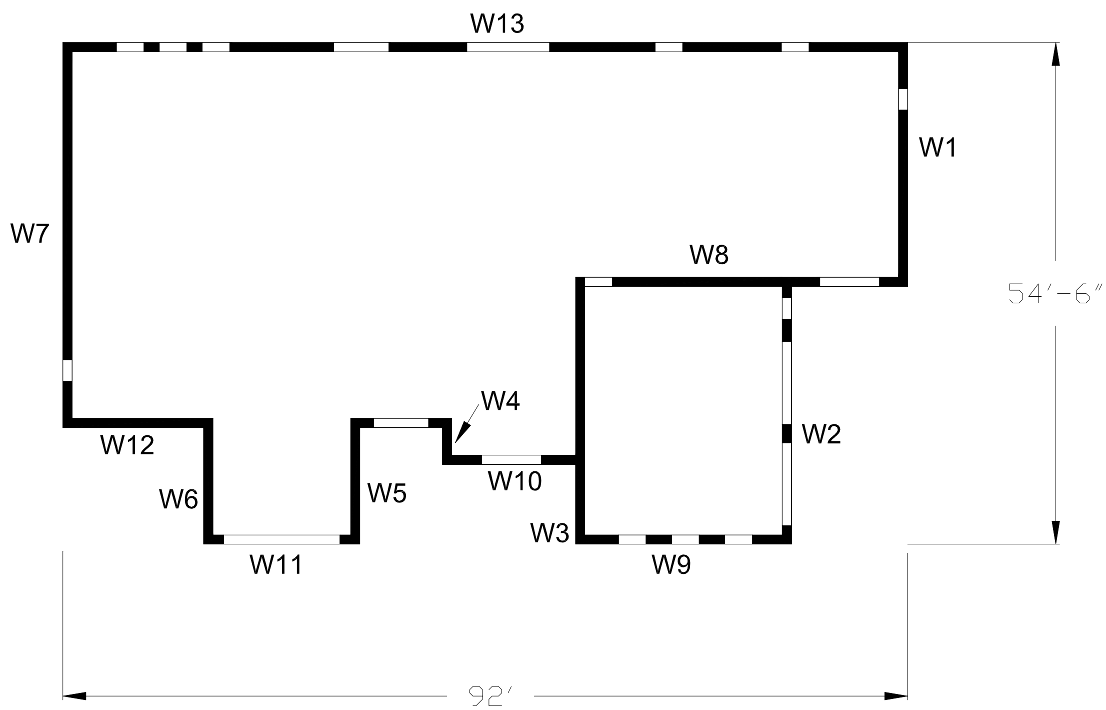



**Figure A1**
**Shear Wall Schedule**

**TABLE A1**
**NS DIRECTION (EXPOSURE C)**

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Resistance, lb | Check |
|---|---|---|---|---|---|---|
| | | -300,175 | | | | |
| W1 | 8,257 | | 2,102 | 10,359 | 6,955 | Underdesigned |
| W2 | 0 | | 0 | 0 | 0 | Underdesigned |
| W3 | 10,371 | | 878 | 11,249 | 8,736 | Underdesigned |
| W4 | 1,574 | | 29 | 1,603 | 1,326 | Underdesigned |
| W5 | 4,538 | | -119 | 4,538 | 3,822 | Underdesigned |
| W6 | 4,723 | | -492 | 4,723 | 3,978 | Underdesigned |
| W7 | 13,598 | | -2,398 | 13,598 | 11,454 | Underdesigned |
| **Total** | **43,061** | | | | | |

While the results of the analysis indicate that shear walls of the demonstration building are underdesigned by as much as 49 percent (Wall 1) this finding must be tempered by judgement and a number of qualifying assumptions. As one qualifying assumption, the provisions of ASCE 7-98 specify wind exposure categories that are conservative in respect to the demonstration site conditions. If a more realistic Exposure B is assumed, then the total lateral load in NS direction is 29,756 lb (calculations are not shown) and Wall 1 is underdesigned by only 3 percent (Table A2) which is generally an acceptable margin for engineering safety checks.

**TABLE A2**
**NS DIRECTION (EXPOSURE B)**

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Resistance, lb | Check |
|---|---|---|---|---|---|---|
| | | -207,427 | | | | |
| W1 | 5,706 | | 1,452 | 7,158 | 6,955 | Underdesigned |
| W2 | 0 | | 0 | 0 | 0 | OK |
| W3 | 7,167 | | 607 | 7,773 | 8,736 | OK |
| W4 | 1,088 | | 20 | 1,108 | 1,326 | OK |
| W5 | 3,136 | | -82 | 3,136 | 3,822 | OK |
| W6 | 3,263 | | -340 | 3,263 | 3,978 | OK |
| W7 | 9,397 | | -1,657 | 9,397 | 11,454 | OK |
| **Total** | **29,756** | | | | | |

In addition, the method for calculation of the design wind loads is based on gable roof configurations that provide conservative estimates for more aerodynamic hip roofs. Furthermore, including the effect of the partitions, dead load, and contribution of stucco siding will significantly increase the wall resistance relative to the analysis.

Tables A3 and A4 summarize results of the shear wall analysis in the East-West (EW) direction for Exposure categories C and B, respectively. The discussion in the previous paragraph is valid.

**TABLE A3**
**EW DIRECTION (EXPOSURE C)**

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Resistance, lb | Check |
|---|---|---|---|---|---|---|
| | | -296,375 | | | | |
| W8 | 2,955 | | 91 | 3,046 | 2,806 | Underdesigned |
| W9 | 3,320 | | 594 | 3,914 | 3,153 | Underdesigned |
| W10 | 1,660 | | 220 | 1,880 | 1,576 | Underdesigned |
| W11 | 0 | | 0 | 0 | 0 | Underdesigned |
| W12 | 5,026 | | 560 | 5,587 | 4,774 | Underdesigned |
| W13 | 13,755 | | -1,466 | 13,755 | 13,063 | Underdesigned |
| **Total** | **26,716** | | | | | |

**TABLE A4**
**EW DIRECTION (EXPOSURE B)**

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Resistance, lb | Check |
|---|---|---|---|---|---|---|
| | | -204,698 | | | | |
| W8 | 2,041 | | 63 | 2,104 | 2,806 | OK |
| W9 | 2,293 | | 410 | 2,704 | 3,153 | OK |
| W10 | 1,146 | | 152 | 1,298 | 1,576 | OK |
| W11 | 0 | | 0 | 0 | 0 | |
| W12 | 3,472 | | 387 | 3,859 | 4,774 | OK |
| W13 | 9,500 | | -1,012 | 9,500 | 13,063 | OK |
| **Total** | **18,452** | | | | | |

# APPENDIX B
# ANALYSIS OF THE LATERAL FORCE RESISTING SYSTEM OF THE DEMONSTRATION HOUSE IN RENTON, WA (CASE STUDY II)

This appendix incorporates engineering calculations and technical substantiation prepared by the NAHB Research Center for submittal to the building code authority as a part of the complete building permit application package assembled by the builder.

## INTRODUCTION

This proposal is a continuation of an ongoing project that was started as a task under the Partnership for Advanced Technology in Housing (PATH) program sponsored by the U.S. Department of Housing and Urban Development (HUD). The objective of this task is to bring innovative design and construction methods into the panelized wall industry to produce structural building systems that more efficiently meet performance requirements of existing building codes.

The following two sections of the report present the construction details and engineering calculations for the proposed design alternatives, respectively. The analysis is based on house Plan 2575 provided by Quadrant Homes, Bellevue, WA.

## CONSTRUCTION DETAILS

This section presents shear wall schedule and anchorage requirements. In particular, the truss plate holddowns are described in detail.

Table B1 summarizes the shear wall construction and design characteristics as prescribed by the UBC-97 (Table 23-II-I-1).

**TABLE B1**
**SHEAR WALL CONFIGURATIONS**

| Shear wall designation | Nailing schedule on panel edges | Design shear value[1], lb/ft | Construction details |
|---|---|---|---|
| P1-6 | 6 inches on center | 230 | • Nails: 8d common (0.131 inch in diameter and 2.5 inch long) or equivalent pneumatic<br>• Nailing schedule in the panel field: 12 inches on center<br>• Sheathing 7/16-inch-thick OSB on one side<br>• SPF studs spaced 16 inches on center<br>• Top and bottom plates: Hem-Fir lumber |
| P1-4 | 4 inches on center | 352 | |

[1]Design shear values include applicable adjustments in footnotes of UBC-97 Table 23-II-I-1.

Figures B1 and B2 display the shear wall schedule for the first and second story of the building, respectively. The STHD-type holddowns, which are typically used with this home plan, are replaced with HTT-type holddowns (Simpson Strong-Tie Co. on-line catalog, 2001). The HTT-type holddowns can be installed in the factory. The HTT holddowns are connected to the anchor bolts after the panel installation using threaded rods and coupling nuts. Accurate positioning of the anchor bolts can be achieved by using a template with predrilled holes that will locate the bolt at a required distance from the foundation wall corner. The HTT-type holddowns will also eliminate interference with sheathing installation (i.e., edge nailing) and exterior finish installation. Note that the STHD holddowns may continue to be used in lieu of the recommended HTT holddowns.

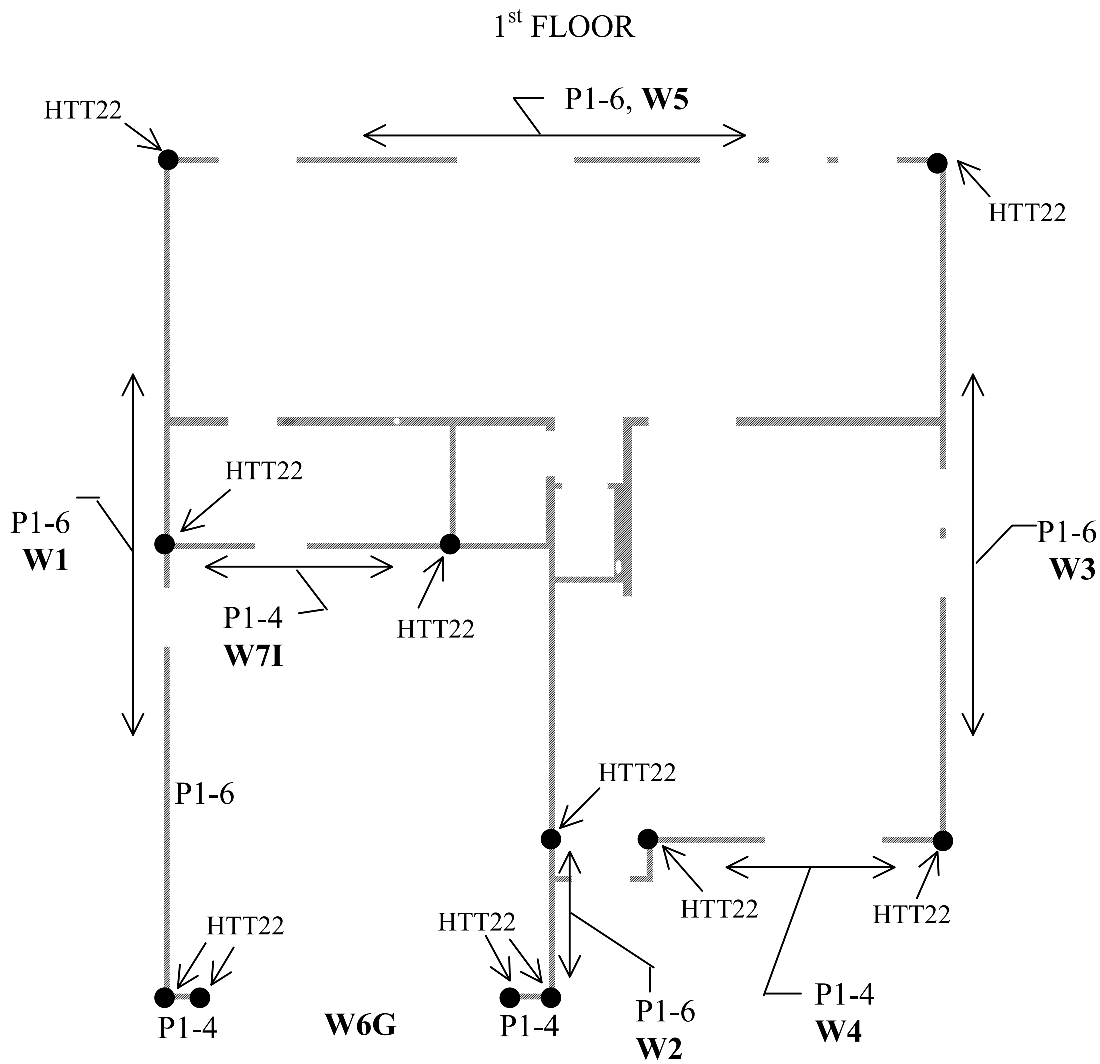



**Figure B1**
**First Floor Shear Wall Schedule**
**(Perforated Shear Wall Method, Except Segment Method used at Garage Opening)**

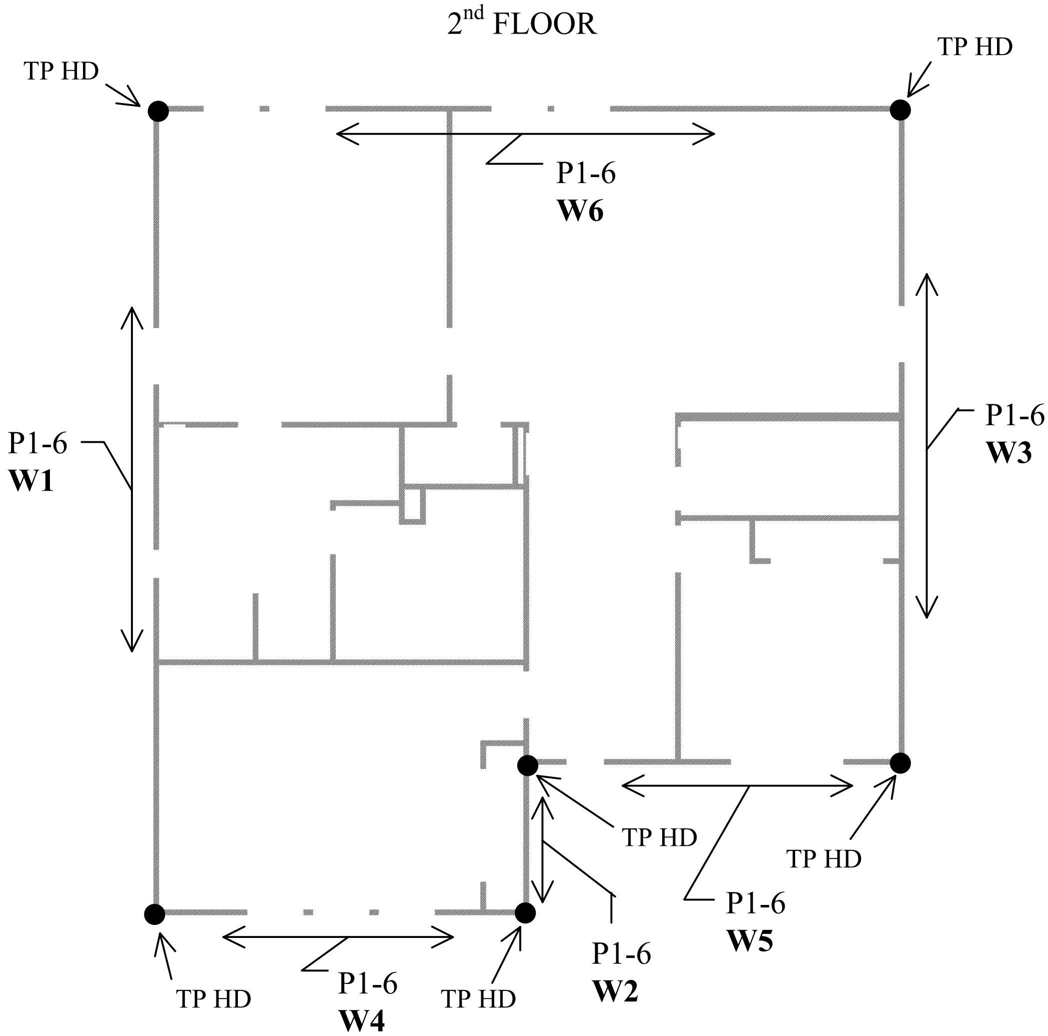



**Figure B2**
**Second Floor Shear Wall Schedule with Truss Plate**
**Reinforced Holddowns at Corners (TP HD)**
**(Perforated Shear Wall Method)**

Figure B3 shows the anchorage schedule for the front wall according to the perforated shear wall method. In addition, metal truss plates are specified around openings and at the corners.

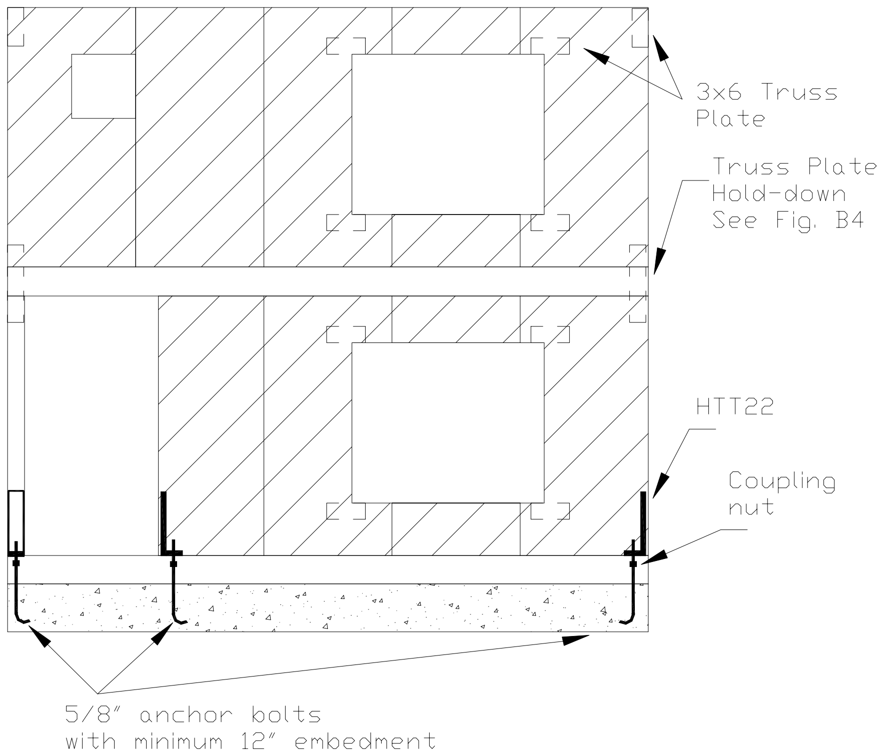
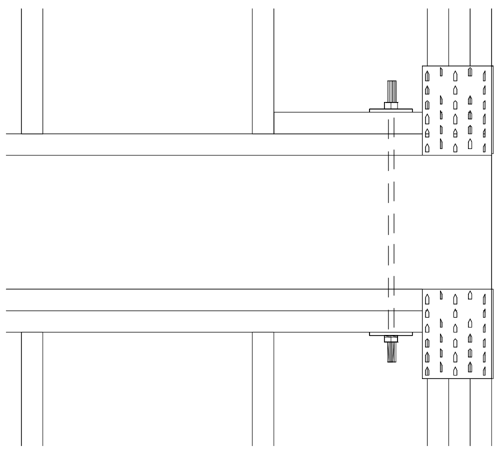



**Figure B3**
**Perforated Shear Wall (Front Wall)**

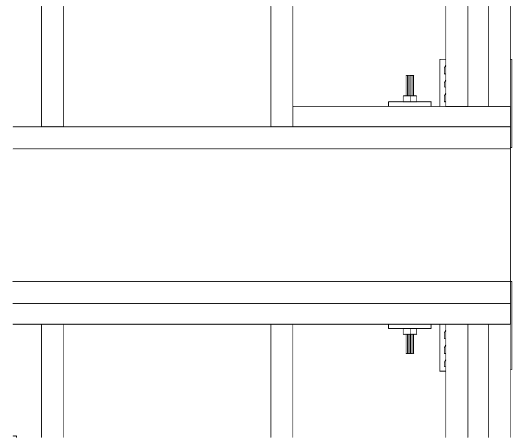
The metal straps on the second story walls are replaced with integral overturning restraints that utilize the combined resistance of metal truss plates, bolts with plate washers, and perpendicular walls. Figure B4 shows construction details of the truss plate holddowns for the second story shear walls. Specifications for the truss plate holddowns follow:

1. 1/2 bolts are located a maximum of 8 inches from the exterior face of the corner, the holes can be drilled in place;
2. 3 x 3 x 0.25-inch square washers are used with the bolts;
3. M II 20 5 x 6 inch MiTek truss plates are installed on both sides of the corner;
4. M II 20 3 x 6 inch MiTek truss plates are installed on both sides of the adjacent corner;
5. sheathing from both wall panels is attached with edge nailing to the same corner (end) stud;
6. the corner studs are pre-cut to accommodate the double bottom plate where 5x6 M II 20 plates are installed; a regular length stud is used with the adjacent wall where 3x6 M II 20 truss plates are used;
7. wall panels are nailed together at corner studs with a minimum of five framing nails (0.131 inch diameter and 3 1/4 inch long);
8. three 12-inch-long spacers are used between the studs of the main corner (corner with 5x6 inch truss plates). The spacers are nailed to each adjacent stud with a minimum of three framing nails (0.131 inch diameter and 3 1/4 inch long) (six nails total per spacer).

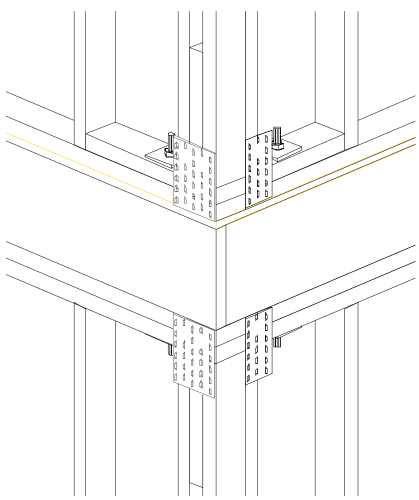
Instead, three 4.5-inch nails can be used for full penetration through both studs and the spacer. A full height stud can be used instead of the spacers.



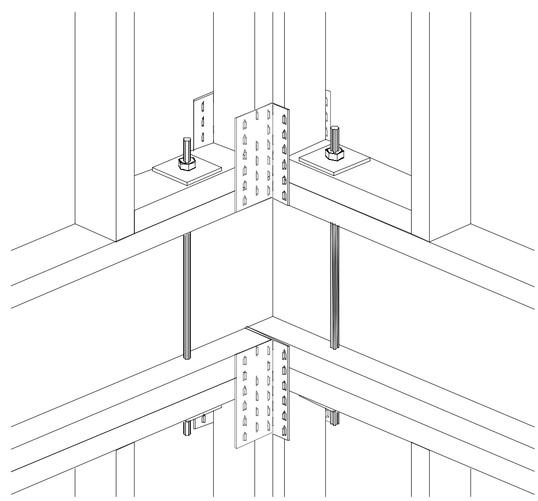
a. Outside view



b. Outside view with truss plates hidden



c. 3-D outside view



d. 3-D inside view (floor is hidden)

**Figure B4**
**Details of a Truss Plate Holddown for the Second Story Shear Walls**
**(Sheathing is Not Shown)**

This design is based on the following criteria:

1. perforated shear wall method;
2. shear wall panels (as a part of a perforated shear wall) with aspect ratio of 2.5:1 for seismic design (3.5:1 is required by the UBC-97 for the seismic zone 3, Table 23-II-G); and,
3. truss plate overturning restraints on the second story shear walls.

The first item is substantiated with extensive experimental and analytical data on the monotonic and cyclic response of the perforated shear walls. This method is adopted by the Standard Building Code (SBCCI 1999), International Building Code (IBC 2000), and NEHRP Recommended Provisions for Seismic Regulations for New Building and Other Structures (BSSC, 2001).

M II 20 truss plates (3x6) around the windows will provide a mechanism for improved force transfer between the wall segments resulting in a wall configuration that acts more as a unit. Therefore, the second assumption is justified.

The use of the truss plate overturning restraints are substantiated by engineering calculations and results of full-scale testing.

## ENGINEERING CALCULATIONS

This section presents engineering calculations that substantiate the proposed shear wall schedule. The structural analysis shows that only the EW walls of the first story are designed to resist loads approaching their allowable design resistance (up to 93 percent). The rest of the walls are considerably overdesigned and have minimum construction characteristics allowed by the 1997 UBC.

### DESIGN METHODS

### Horizontal Force Distribution

The analysis uses rigid diaphragm method to distribute lateral forces between the shear walls. This method was confirmed as the most accurate for design of light-frame buildings by recent whole-house testing programs sponsored by HUD, NAHB, and FEMA. Appendix C summarizes some of the technical information that supports the use of the rigid diaphragm method for residential light-frame wood structures.

The direct shear is distributed among the walls relative to their capacities. The torsional moment is defined as the product of the shear force acting on a given story and the eccentricity between the center of rigidity and the center of stiffness for the same story. The moment is distributed among the shear walls according to Equations (B1) and (B2).

$$V_T = \frac{M_T \; r_i \; F_i}{J} \qquad \text{(B1)}$$

$$J = \sum_{i}^{n} F_i \; r_i^2 \qquad \text{(B2)}$$

where:

$V_T$ = torsional shear load on a wall line;
$M_T$ = torsional moment – a product of total story shear load and perpendicular distance between the load vector resultant resistance vector for load direction under consideration;
$r_i$ = distance from the wall to the center of stiffness (center of resistance);
$V_i$ = design shear wall capacity;
J = torsional moment of inertia of the story.

For seismic design, 5 percent of the building dimension at the level of interest perpendicular to the direction of the force under consideration is added to the building eccentricity (Section 1630.6, 1997 UBC). The torsional shear force is additive to the direct shear. The negative torsional forces that counteract the direct shear are ignored.

### Perforated Shear Wall Method

The design shear wall capacity is calculated using perforated shear wall method. The ratio of the shear strength for a wall with openings to the shear strength of a fully sheathed wall, *F*, is determined as:

$$F = \frac{r}{3 - 2r} \qquad \text{(B3)}$$

$$r = \frac{1}{1 + \frac{A_o}{H \Sigma L_i}} \qquad \text{(B4)}$$

where:

- $r$ = sheathing area ratio;
- $A_o$ = total area of openings;
- $H$ = height of the wall; and,
- $\Sigma l_i$ = summation of length of all full height wall segments.

## LOAD CALCULATIONS

### Wind Loads

The 1997 UBC permits the use of ASCE 7, Chapter 6 for calculation of Wind Loads (UBC-97, Chapter 16, Section 1604 – Standards). Therefore, wind loads are determined according to the provisions of ASCE 7-98 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2000). The method for "buildings and other structures" is used (Section 6.5.3, ASCE 7-98).

1. Basic wind speed, V, and wind directionality factor, $K_d$.
   - V = 85 mph (Figure 6.1)
   - $K_d$ = 0.85 main wind force resisting system (Table 6.6)

2. Importance factor, I.
   - I = 1.0 (Table 6-1)

3. Exposure category and velocity pressure coefficient, $K_z$ or $K_h$.
   - Exposure B
   - The design procedure for buildings and other structures is used (Section 6.5.6.2.1).
     - $K_z$ = 0.61 (Table 6.5, Case 2, z =18'2" )
     - $K_h$ = 0.65 (Table 6.5, Case 2, h = 23'7")

4. Topographic Factor, $K_{zt}$.
Not included in the design.

5. Gust effect factor, G.
$G = 0.85$ (Section 6.5.8.1 - Rigid Structures)

6. Enclosure classification.
The building is classified as enclosed.

7. Internal pressure coefficient $GC_{pi}$.
$CP_{pi} = \pm 0.18$ (Table 6-7, enclosed structures)
In case of determining the total shear, the internal pressures cancel out.

8. External pressure coefficients $C_p$, (Figure 6-3)

*Wind in EW direction (perpendicular to ridge)*

Walls

$L/B = 40/43 = 0.93 < 1$

Windward $C_p = 0.8$
Leeward $C_p = -0.5$
Side walls $C_p = -0.7$

Roof

$h/L = 23.7 / 40 = 0.6$
$\theta = 27^o$

Windward
$C_p = -0.3$ or $C_p = 0.2$
Leeward
$C_p = -0.6$

*Wind in NS direction (parallel to ridge)*

Walls

$L/B = 43/40 = 1.1$

Windward $C_p = 0.8$
Leeward $C_p = -0.5$
Side walls $C_p = -0.7$

Roof

Cancel out for the total shear

9. Velocity pressure $q_h$.
$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I = 0.00256\ (0.61)\ (1.0)\ (0.85)\ (85)^2\ (1.0) = 9.6\ lb/ft^2$
$q_h = 0.00256\ K_h\ K_{zt}\ K_d\ V^2\ I = 0.00256\ (0.65)\ (1.0)\ (0.85)\ (85)^2\ (1.0) = 10.2\ lb/ft^2$

10. Design wind load p.

$p = q\, GC_p - q_i\, (GC_{pi})$

*Wind in EW direction (perpendicular to ridge)*

Walls

Windward

$p = (9.6)(0.85)(0.8) = 6.5 \text{ lb/ft}^2$

Leeward

$p = (10.2)(0.85)(-0.5) = -4.3 \text{ lb/ft}^2$

Side walls

$p = (10.2)(0.85)(-0.7) = -6.1 \text{ lb/ft}^2$

Roof

Windward

Negative Pressure

$p = (10.2)(0.85)(-0.3) = -2.6 \text{ lb/ft}^2$

Positive Pressure

$p = (10.2)(0.85)(0.2) = 1.7 \text{ lb/ft}^2$

Leeward

$p = (10.2)(0.85)(-0.6) = -5.2 \text{ lb/ft}^2$

*Wind in SN direction (parallel to ridge)*

Walls

Windward

$p = (10.2)(0.85)(0.8) = 6.9 \text{ lb/ft}^2$

Leeward

$p = (10.2)(0.85)(-0.5) = -4.3 \text{ lb/ft}^2$

Side walls

$p = (10.2)(0.85)(-0.7) = -6.1 \text{ lb/ft}^2$

Roof

No pressure on the main LFRS

**Forces:**

Areas are measured from the electronic drawings provided by Quadrant Homes using AutoCAD software package.

<u>EW direction</u>

Roof:

F = (1.7 + 5.2) (395 + 51) = 3,077 lb

Second story

F = (6.5 + 4.3) (176) + 3,077 = **4,978 lb**

First story

F = (6.5 + 4.3)(392) + 4,978 = **9,212 lb**

<u>NS direction</u>

Roof:

F = (6.9 + 4.3) (215) = 2,408 lb

Second story

F = (6.9 + 4.3) (165) + 2,408 = **4,256 lb**

First story

F = (6.9 + 4.3)(362) + 4,256 = **8,310 lb**

**Seismic Loads**

Masses of building components:

| | |
|---|---|
| Roof: | **15 psf** |
| Partitions: | **8 psf** |
| Exterior Walls: | **10 psf** |
| Diaphragm: | **10 psf** |

**Roof mass**
It is assumed that the mass per square foot of the garage part of the roof is equal to that of the main part of the building. The roof area includes the 1.5 foot overhang around the building.
$W_R$ = [(40 + 3)(35 + 3) + (20 + 3)(8.0)] (15) = **27,270 lb**

**Second story wall mass**
The mass is computed using half height of the walls and partitions of the story. It is assumed that the partitions are nailed to the ceiling as well as to the floor. The lengths of the partitions are measured from the drawings.
$W_{W2}$ = (8.1/2)[(40 + 43)(2)](10) + (8.1/2)[40 + 20 + (16.75)(2) + 12 + 5.5 + (18.2)(2) + 12.25 + 20 + 12](8) = **12,933 lb**

**Total mass acting on the second story shear walls**
$W_2$ = 27,270 + 12,933 = **40,202 lb**

**First story wall mass**
The mass is computed using single story wall height and diaphragm depth. It is assumed that the partitions are nailed to the ceiling as well as to the floor.
$W_{w1}$ = (8.1 + 1.1)[(40 + 43)(2)](10) + (8.1/2)[40 + 20 + (16.75)(2) + 12 + 5.5 + (18.2)(2) + 12.25 + 20 + 12](8) + (8.1/2)[40 + 20 + 21.75 + (6.75)(3)](8) = **24,786 lb**

**Diaphragm mass**
$W_{d1}$ = [(40)(35) + (20)(8.0)] (10) = **15,600 lb**

**Porch roof mass**
$W_{p1}$ = (9)(9)(15) = **1,215 lb**

**Total first story mass**
$W_{s1}$ = 24,786 + 15,600 + 1,215 = **41,600 lb**
**Total mass acting on the first story shear walls**
$W_1 = W_2 + W_{s1}$ = 40,200 + 41,600 = **81,800 lb**

Static design procedure (Section 1630.2.3 of UBC-97) is used to calculate base shear because it is a two-story standard occupancy structure.

**Base shear:**

$$V = \frac{2.5\, C_a\, I}{R} W$$

| | |
|---|---|
| $C_a$ | = 0.36 for Seismic Zone 3 and unknown soil type (Table 16-Q) |
| R | = 5.5 (Table 16-N) |
| I | = 1.0 (Table 16-K) |

**Vertical distribution of force (Section 1630.5, 1997 UBC)**

First story shear
$V_1$ = (2.5)(0.36)(81,800)(1.0)/5.5 = 13,385 lb

Second story shear
$V_2$ = (13,385) (40,202)(19.1)/[(40,202)(19.1) + (41,600)(8.5)] = 9,165 lb

For allowable stress design, the seismic loads are reduced by a factor of 1.4 (Section 1612.3.1, 1997 UBC).
$V_1$ = 13,385/1.4 = **9,561 lb**
$V_2$ = 9,165/1.4 = **6,546 lb**

**Shear Wall Design**

The allowable shear values are determined from Table 23-II-I-1 of the UBC-97. The values are reduced with the species adjustment factor of 0.82 to account for the specific gravity of the framing members made of Hem-Fir lumber (SG = 0.43).

<u>Assumptions:</u>

| | |
|---|---|
| Shear walls | Perforated shear wall method, holddowns only at the corners, truss plates are used around openings and at the corners for enhanced performance. The maximum allowable shear wall aspect ratio of 2.5:1 is used for segments within a perforated shear wall for both wind and seismic design. |
| Garage door | segmented shear wall with 4 inch on center nailing schedule (352 lb/ft of braced wall panels) |
| Species | Hem-Fir |
| Sheathing | 7/16 OSB |
| Nailing schedule | 6/12 inch (229 lb/ft), all EW walls of the first floor 4/12 (352 lb/ft) except the north wall (Wall 5) |
| Sheathing Nails | 8d common or equivalent pneumatic (D=0.131 inch) |
| Stud spacing | 16 inches on center |

Results of Wind Design:

### Second Floor - NS

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | 6,261 | | | | |
| W1[1] | 2,051 | | -102 | 2,051 | 8,080 | Ok |
| W2 | 466 | | 2 | 468 | 1,837 | Ok |
| W3 | 1,738 | | 101 | 1,839 | 6,847 | Ok |
| W4 | | | -29 | 29 | 1,610 | Ok |
| W5 | | | -28 | 28 | 2,241 | Ok |
| W6 | | | 56 | 56 | 4,855 | Ok |
| **Total** | **4,256** | | | | | |

[1]See Figure B2 for wall notations.

### Second Floor - EW

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | -16,903 | | | | |
| W1 | | | 277 | 277 | 8,080 | Ok |
| W2 | | | -5 | 5 | 1,837 | Ok |
| W3 | | | -272 | 272 | 6,847 | Ok |
| W4 | 921 | | 77 | 998 | 1,610 | Ok |
| W5 | 1,281 | | 75 | 1,356 | 2,241 | Ok |
| W6 | 2,776 | | -152 | 2,776 | 4,855 | Ok |
| **Total** | **4,978** | | | | | |

### First Floor - NS

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | 28,862 | | | | |
| W1[1] | 4,503 | | -504 | 4,503 | 8,314 | Ok |
| W2 | 746 | | 18 | 764 | 1,378 | Ok |
| W3 | 3,060 | | 487 | 3,547 | 5,650 | Ok |
| W4 | | | -127 | 127 | 2,367 | Ok |
| W5 | | | 231 | 231 | 3,087 | Ok |
| W6G | | | -110 | 110 | 1,322 | Ok |
| W7I | | | 6 | 6 | 4,838 | Ok |
| **Total** | **8,310** | | | | | |

[1]See Figure B1 for wall notations.

### First Floor - EW

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | -4,106 | | | | |
| W1 | | | 72 | 72 | 8,314 | Ok |
| W2 | | | -2 | 2 | 1,378 | Ok |
| W3 | | | -69 | 69 | 5,650 | Ok |
| W4 | 1,877 | | 18 | 1,895 | 2,367 | Ok |
| W5 | 2,448 | | -33 | 2,448 | 3,087 | Ok |
| W6G | 1,049 | | 16 | 1,064 | 1,322 | Ok |
| W7I | 3,838 | | -1 | 3,838 | 4,838 | Ok |
| **Total** | **9,212** | | | | | |

## Results of Seismic Design:

### Second Floor - NS

| Wall | Direct Shear Force, lb | Torsional Moment, lb-ft | Torsional Shear Force, lb | Total Shear Force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | 16,008 | | | | |
| W1[1] | 3,155 | | -262 | 3,155 | 8,080 | Ok |
| W2 | 717 | | 5 | 722 | 1,837 | Ok |
| W3 | 2,674 | | 257 | 2,931 | 6,847 | Ok |
| W4 | | | -73 | 73 | 1,610 | Ok |
| W5 | | | -71 | 71 | 2,241 | Ok |
| W6 | | | 144 | 144 | 4,855 | Ok |
| **Total** | **6,546** | | | | | |

[1]See Figure B2 for wall notations.

### Second Floor - EW

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | -32,037 | | | | |
| W1 | | | 524 | 524 | 8,080 | Ok |
| W2 | | | -9 | 9 | 1,837 | Ok |
| W3 | | | -515 | 515 | 6,847 | Ok |
| W4 | 1,211 | | 147 | 1,357 | 1,610 | Ok |
| W5 | 1,685 | | 141 | 1,826 | 2,241 | Ok |
| W6 | 3,651 | | -288 | 3,651 | 4,855 | Ok |
| **Total** | **6,546** | | | | | |

### First Floor - NS

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | 44,992 | | | | |
| W1[1] | 5,181 | | -786 | 5,181 | 8,314 | Ok |
| W2 | 859 | | 27 | 886 | 1,378 | Ok |
| W3 | 3,521 | | 759 | 4,280 | 5,650 | Ok |
| W4 | | | -198 | 198 | 2,367 | Ok |
| W5 | | | 359 | 359 | 3,087 | Ok |
| W6G | | | -171 | 171 | 1,322 | Ok |
| W7I | | | 10 | 10 | 4,838 | Ok |
| **Total** | **9,561** | | | | | |

[1]See Figure B1 for wall notations.

### First Floor - EW

| Wall | Direct shear Force, lb | Torsional Moment, lb-ft | Torsional shear force, lb | Total shear force, lb | Wall Capacity, lb | Check |
|---|---|---|---|---|---|---|
| | | -22,807 | | | | |
| W1 | | | 398 | 398 | 8,314 | Ok |
| W2 | | | -14 | 14 | 1,378 | Ok |
| W3 | | | -385 | 385 | 5,650 | Ok |
| W4 | 1,948 | | 100 | 2,049 | 2,367 | Ok |
| W5 | 2,541 | | -182 | 2,541 | 3,087 | Ok |
| W6G | 1,088 | | 87 | 1,175 | 1,322 | Ok |
| W7I | 3,983 | | -5 | 3,983 | 4,838 | Ok |
| **Total** | **9,561** | | | | | |

## DESIGN OF OVERTURNING RESTRAINTS

### Design of Truss Plate Holddowns (Figure B4) for the Second Story Shear Walls

The design of overturning restraints is governed by the seismic analysis. The uplift forces are calculated based on the capacity of the wall that can be achieved during a seismic event rather than on the reduced forces calculated using the R-factor. This design procedure is consistent with the seismic design philosophy incorporated in the procedures for determination of the structural seismic loads. This approach results in a more accurate connection design.

According to the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA-273) (BSSC, 1997), capacity of the shear walls specified for the second floor is $v$ = 720 lb/ft

Including adjustment for specific gravity of Hem-Fir lumber (SG=0.43):

$$v = (720)(1- (0.5\text{-SG})) = (720)(1-(0.5-0.43)) = 670 \text{ lb}$$

Uplift force (based on the actual capacity of the shear wall):

$$T = v\ h = (670)(8) = 5{,}360 \text{ lb}$$

where:
- $v$ = unit shear capacity;
- h = shear wall height.

The uplift capacity of the truss plate holddowns (Figure 4):

$$U1 = (145 \text{ psi})[(15 \text{ in}^2)](0.85)(3.2) = 5{,}916 \text{ lb}$$

where:

- 145 psi = allowable lateral resistance value for M II 20 MiTech Truss Connector Plates installed in Hem-Fir lumber (ICBO Evaluation Report ER-4922, ICBO 1999) for the EE Plate Orientation;
- 15 in$^2$ = area of the double bottom plate covered by the truss plate;
- 0.85 = reduction coefficient due to truss plate installation on the narrow face of the member (Section 2.3.4, ER-4922);
- 3.2 = reduction coefficient that adjusts the average test value to the design value (Section 7.1.9, ANSI/TPI 1-1995, TPI 1995).

This approach for determining the resistance of a truss plate hold-down was validated with full scale testing as reported in *Case Study II – Light-Frame Wood Panels* of this publication. In summary, the testing of a shear wall showed that this type of a holddown with a 12.75 inch contact area and assembled with SPF lumber resisted an uplift force of 5,377 lb.

The uplift capacity of the adjacent corner:

$$U2 = (145 \text{ psi})[(4.5 \text{ in}^2)](0.85)(3.2) = 1{,}775 \text{ lb}$$

where:

$4.5\ in^2$ = area of the double bottom plate covered by the truss plate;

Capacity of the truss plate holddown including the corner resistance:

$$UT = U1 + U2 = 5{,}916 + 1{,}775 = 7{,}691 \text{ lb}$$

Holddown overstrength factor = Uplift Resistance/Uplift Force = 7,691/5,360 = 1.44 > 1.0 - OK

The holddown overstrength factor should be interpreted as a factor that if greater than unity indicates that the sheathing nails will reach their capacity before the holddown reaches its capacity. Thus, a ductile shear wall response is ensured for a holddown overstrength factor of greater than unity. It should be noted that the uplift force is not reduced by any portion of the roof dead load and the second story dead load. Therefore, the overstrength factor of 1.44 can be considered as a conservative estimate.

**Design of Overturning Holddowns for the First Story Shear Walls**

According to the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA-273) (BSSC, 1997), capacity of the shear walls specified for the first floor:

$$v = 900 \text{ lb/ft}$$

Including adjustment for specific gravity of Hem-Fir lumber (SG=0.43):

$$v = (900)(1- (0.5\text{-SG})) = (900)(1-(0.5-0.43)) = 837 \text{ lb}$$

Uplift force:

$$T = v\ h = (837)(8) = 6{,}696 \text{ lb}$$

Total uplift force including the second story:

$$T = 6{,}696 + 5{,}360 = 12{,}056 \text{ lb}$$

The uplift capacity of HTT22 is 13,150 lb (Simpson Strong-Tie Co. on-line catalog, 2001)

Holddown overstrength factor = 13,150/12,056 = 1.1 - OK

Note that the uplift force is not reduced by any portion of the dead load of the second story. Thus, the lower story uplift for anchorage design may be considered to be conservative and the actual overstrength factor is greater than 1.1.

## APPENDIX C
## RIGID DIAPRAGM METHOD

Due to a complex three-dimensional force distribution mechanism involved in the analysis of the stiffness characteristics of the diaphragm–shear wall assemblies, there is a lack of guidelines on the selection of the appropriate lateral force distribution procedures in the model building codes. As a typical practice, the current building codes provide stringent rules in favor of more conservative flexible diaphragm method. Recently, several studies have been conducted towards answering the problem of the lateral force distribution between the shear walls in light-frame buildings. A summary of these studies follows.

[1] This study investigated load sharing mechanism between shear walls in a 16 foot by 32 foot one-story light-frame house. The roof diaphragm was sheathed with plywood panels using eight-penny nails. The sheathing was not glued or blocked. Gypsum panels were used on the ceiling. Results of the testing showed that the roof diaphragm exhibited nearly rigid behavior. The load distribution between the shear walls depended on both wall stiffness and wall position within the building. The walls perpendicular to the direction of loading resisted between 8 and 25 percent of the total load due to diaphragm rotation.

[2] The researchers presented a three-dimensional finite element model of a light-frame wood house. The model was validated using results from a full-scale testing program. The model was used to evaluate the accuracy of the rigid and flexible diaphragm design methods using a 16 foot by 32 foot light-frame wood buildings with two partition walls. Results of the modeling showed that the flexible diaphragm method misrepresented the shear wall forces with an error exceeding 120 percent, whereas the rigid diaphragm method predicted the shear wall forces with a maximum error of 21 percent. Both methods provided results that overestimated and underestimated the finite element model predictions. The load sharing mechanism modeled by the rigid diaphragm method was representative of the experimental and finite element modeling results, whereas the flexible diaphragm method provided an arbitrary force distribution based on building geometry (i.e. tributary areas) rather than stiffness.

[3] NEHRP Seismic Design Provisions incorporate the state-of-the-art design methods for analysis of structures against seismic forces. The Provisions follow the methodology previously introduced in the UBC to define rigid and flexible diaphragm buildings. However, for light-frame structures, the Provisions require using the rigid diaphragm approach if the diaphragm is assembled with structural panel sheathing (Section 5.2.3.1). This indicates that the Provisions encourage using the rigid diaphragm method as opposed to the flexible diaphragm method for design of light-frame buildings.

[4] NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings provides recommendations for seismic analysis of existing structures. In Chapter 8, which discusses wood construction, the Commentary indicates that recent studies demonstrated rigid behavior of the diaphragms in wood buildings. Therefore, the lateral loads should be distributed to the shear walls based on the relative stiffness instead of tributary areas.

[5] This project investigated the applicability of various design methods for distribution of the lateral forces between the shear walls in a light-frame building. The analytical results were

validated using experimental data from testing of a full-scale L-shaped one-story wood-frame 30 foot by 36 foot house. Results of the project indicated that the rigid diaphragm method accurately predicted force distribution between the shear walls with a maximum error of 11 percent, whereas the error of the flexible diaphragm method exceeded 37 percent. This building had a conventional roof diaphragm assembled without gluing or blocking the sheathing.

[6] One of the objectives of this project was to evaluate the relative stiffness of the diaphragm–shear wall system of a 16 foot by 20 foot house. Eight diaphragm configurations were investigated each with different combination of nailing schedule, adhesive, and blocking. Diaphragms assembled with nails and without adhesive or blocking were classified as flexible according to the UBC-97 provisions, whereas diaphragms assembled with either adhesive or blocking or both were classified as rigid. This testing program used shear walls without perforations and a building configuration without intermediate shear walls. Both of these building attributes contributed towards the selection of the flexible diaphragm approach. Therefore, results of this testing program conservatively represent the actual response of light-frame homes that typically have many openings. It should be noted that the definition of the rigid diaphragm provided in the UBC does not necessarily mean that buildings that fall beyond the scope of the rigid diaphragm definition will actually behave as flexible diaphragm buildings.

**References:**

[1] Phillips, T. L., Itani, R. Y., and McLean, D. I. 1992. Lateral Load Sharing by Diaphragms in Wood-Framed Buildings. Journal of Structural Engineering, Vol. 119(5), pp. 1556-1571.

[2] Kasal, B., and Leichti, R. J.1992. Incorporating Load Sharing in Shear Wall Design of Light-Frame Structures. Journal of Structural Engineering, Vol. 118, No. 12, pp. 3350-3361.

[3] Building Seismic Safety Council. 1997. NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA Publication 302). Washington, DC.

[4] Building Seismic Safety Council. 1997. NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA Publication 274). Washington, DC.

[5] Foliente, G., Paevere, P., Kasal, B., and Collins, M. 2000. Whole Structure Testing and Analysis of a Light-Frame Wood Building. Phase 2 – Design Procedures Against Lateral Loads. BCE DOC 00/177. CSIRO, Australia.

[6] Fisher, D., Filiatrault, A., Folz, B., Uang, C., and Seible, F. 2000. Shake Table Tests of a Two-Story Woodframe House. Report No. SSRP – 2000/15. Division of Structural Engineering, University of California, San Diego.

# APPENDIX D
# METRIC CONVERSION FACTORS

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, Metric Practice.

| To convert from | to | multiply by |
|---|---|---|
| **Length** | | |
| inch (in.) | micron (μ) | 25,400 |
| inch (in.) | centimeter | 2.54 |
| inch (in.) | meter (m) | 0.0254 |
| foot (ft) | meter (m) | 0.3048 |
| yard (yd) | meter (m) | 0.9144 |
| mile (mi) | kilometer (km) | 1.6 |
| **Area** | | |
| square foot (sq ft) | square meter (sq m ) | 0.09290304 |
| square inch (sq in) | square centimeter (sq cm) | 6.452 |
| square inch (sq in.) | square meter (sq m ) | 0.00064516 |
| square yard (sq yd) | square meter (sq m ) | 0.8391274 |
| square mile (sq mi) | square kilometer (sq km ) | 2.6 |
| **Volume** | | |
| cubic inch (cu in.) | cubic centimeter (cu cm) | 16.387064 |
| cubic inch (cu in.) | cubic meter (cu m) | 0.00001639 |
| cubic foot (cu ft) | cubic meter (cu m) | 0.02831685 |
| cubic yard (cu yd) | cubic meter (cu m) | 0.7645549 |
| gallon (gal) Can. liquid | liter | 4.546 |
| gallon (gal) Can. liquid | cubic meter (cu m) | 0.004546 |
| gallon (gal) U.S. liquid* | liter | 3.7854118 |
| gallon (gal) U.S. liquid | cubic meter (cu m) | 0.00378541 |
| fluid ounce (fl oz) | milliliters (ml) | 29.57353 |
| fluid ounce (fl oz) | cubic meter (cu m) | 0.00002957 |
| **Force** | | |
| kip (1000 lb) | kilogram (kg) | 453.6 |
| kip (1000 lb) | Newton (N) | 4,448.222 |
| pound (lb) | kilogram (kg) | 0.4535924 |
| pound (lb) | Newton (N) | 4.448222 |
| **Stress or pressure** | | |
| kip/sq inch (ksi) | megapascal (Mpa) | 6.894757 |
| kip/sq inch (ksi) | kilogram/square centimeter (kg/sq cm) | 70.31 |
| pound/sq inch (psi) | kilogram/square centimeter (kg/sq cm) | 0.07031 |
| pound/sq inch (psi) | pascal (Pa) ** | 6,894.757 |
| pound/sq inch (psi) | megapascal (Mpa) | 0.00689476 |
| pound/sq foot (psf) | kilogram/square meter (kg/sq m) | 4.8824 |
| pound/sq foot (psf) | pascal (Pa) | 47.88 |

| To convert from | to | multiply by |
|---|---|---|
| **Mass (weight)** | | |
| pound (lb) avoirdupois | kilogram (kg) | 0.4535924 |
| ton, 2000 lb | kilogram (kg) | 907.1848 |
| grain | kilogram (kg) | 0.0000648 |
| **Mass (weight) per length)** | | |
| kip per linear foot (klf) | kilogram per meter (kg/m) | 0.001488 |
| pound per linear foot (plf) | kilogram per meter (kg/m) | 1.488 |
| **Moment** | | |
| 1 foot-pound (ft-lb) | Newton-meter (N-m) | 1.356 |
| **Mass per volume (density)** | | |
| pound per cubic foot (pcf) | kilogram per cubic meter (kg/cu m) | 16.01846 |
| pound per cubic yard (lb/cu yd) | kilogram per cubic meter (kg/cu m) | 0.5933 |
| **Velocity** | | |
| mile per hour (mph) | kilometer per hour (km/hr) | 1.60934 |
| mile per hour (mph) | kilometer per second (km/sec) | 0.0268 |
| **Temperature** | | |
| degree Fahrenheit (°F) | degree Celsius (°C) | $t_C = (t_F-32)/1.8$ |
| degree Fahrenheit (°F) | degree Kelvin (°K) | $t_K = (t_F + 459.7)/1.8$ |
| degree Kelvin (°F) | degree Celsius (°C) | $t_C = (t_K - 32)/1.8$ |

*One U.S. gallon equals 0.8327 Canadian gallon
**A pascal equals 1000 Newton per square meter.

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples and submultiples of the SI units.

| Multiplication Factor | Prefix | Symbol |
|---|---|---|
| $1,000,000,000 = 10^9$ | giga | G |
| $1,000,000 = 10^6$ | mega | M |
| $1,000 = 10^3$ | kilo | k |
| $0.01 = 10^{-2}$ | centi | c |
| $0.001 = 10^{-3}$ | milli | m |
| $0.000001 = 10^{-6}$ | micro | μ |
| $0.000000001 = 10^{-9}$ | nano | n |