# Evaluation of Full-Scale House Testing Under Lateral Loading

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## Introduction

Numerous studies have been conducted to measure the lateral performance of low-rise light-frame structures. However, questions remain on correlating engineering design methods to actual building performance, both in the lab and in the field. The difference between calculated and observed performance is commonly attributed to the contribution of finishes and building details that allow the building to act as a complete system.

This study uses existing whole-house structural performance data to make inferences about the range of the system effect. It analyzes published results of whole-house tests to determine the magnitude of the system effects based on comparisons of the tested ultimate shear strength of a house and the ultimate strength (capacity) predicted by engineering calculations and current building codes.

The study was conducted in two phases. First, a comprehensive literature search for whole-house testing was performed. A total of 42 studies were identified and reviewed. Appendix A provides a tabulated summary of research on full-scale buildings and other three-dimensional (3D) systems. Second, the studies with sufficient information to enable comparison of the tested performance with engineering analysis were selected for a more detailed evaluation. Table 1 provides a summary of the selected studies.

**Table 1 - List of Whole-House Structural Studies Selected for Analysis of System Effects**

| # | Title | Country | Year | # Stories | Plan Dimensions | Finishes |
|---|---|---|---|---|---|---|
| 1 | Whole Structure Testing and Analysis of a Light-Frame Wood Building (CSIRO) | Australia | 2000 | One | 30’ x 37’ | Interior gypsum |
| 2 | Shake Table Tests of a Two-Story Wood-frame House (CUREE Wood-frame Project) | USA | 2001 | Two | 20’ x 16’ | Bare Frame / Fully Finished w Stucco |
| 3 | Full-Sized House Cyclic Racking Test (BRANZ) | New Zealand | 2006 | One | 41’ x 19.7’ | Fully Finished |
| 4 | Seismic Testing of Full-Scale Two-Story Light-Frame Wood Building (NEESWood) | USA | 2006 | Two | 22’ x 58’ | Bare Frame / Fully Finished w Stucco |
| 5 | Assessment of Seismic Resistance of Conventional Wood-Frame Houses (Forintek Collaboration) | China | 2006 | Two | 20’ x 20' | Interior Gypsum |
| 6 | Full-Scale Shaking Table Tests of 3-Story Wood-Frame Construction Buildings (Japan 2x4 Home Builders Association, et. al) | Japan | 2006 | Three | 24’ x 24' | Interior and Exterior Finishes |
| 7 | Effect of Transverse Walls and Vertical Load on the Performance of Shear Walls (Forintek/Tongji) | Canada | 2006 | One | 20’ long wall | None |

## Analysis Approach

To evaluate the contribution of the various building system details to the overall strength of the structure, the ultimate tested shear strength of the house is compared to the ultimate predicted shear strength calculated using applicable engineering methods. Any additional strength observed during testing that cannot be accounted for through design is attributed to whole-house system effects. The system factor was calculated as follows:

$$SysF = \frac{\text{Measured Capacity from Test}}{\text{Predicted Capacity from Analysis}}$$

The magnitude of the system factor depends on the specific assumptions used to calculate the predicted capacity (denominator in the equation above). In this study, a range of design assumptions was used to capture a potential spectrum of system factors. The predicted capacity was estimated with and without applicable building code limitations. The design limitations relevant to this study include segment aspect ratios and combination of structural sheathing with interior gypsum finish in seismic applications. Inclusion of building code limitations in the design process typically results in an increase in the system factor because the code does not recognize the contribution of segments that are narrower than the applicable limit and/or in the case of seismic analysis the contribution of gypsum. However, those elements and materials, although not recognized by code as part of the structural load resisting system, do improve the building's performance and are a part of the overall building system. Because the majority of houses have segments that are outside of the code range and all houses have finishes, it is important to capture their contribution to the system factor. On the other hand, because there is a possibility that a structure has been optimized to a degree where the number of non-compliant segments is minimized, this study also evaluates scenarios with the contribution of all segments and gypsum wallboard finish included in the predicted capacity at their full unit shear value. The system factor calculated in this manner represents the lower bound range estimate.

To provide a better range of comparison, four different design methodologies are used to determine the predicted shear strength of the structure.

- Method 1: Perforated Shear Wall Method (Sugiyama) without building code limitations
- Method 2: Perforated Shear Wall Method (2006 International Building Code)
- Method 3: Wind Bracing Design Method used for Public Comment 2 to RB148 (ICC 2007/2008 Code Development Cycle, 2008 Final Action Agenda, International Residential Code)
- Method 4: Segmented Shear Wall Method (2006 International Building Code), with and without code limitations

Each building was analyzed using one or more applicable methods selected in each case based on the specific characteristics of the structure. For example, the perforated shear wall (PSW) method was used where hold-downs were installed only at the ends of walls or where hold-downs were not installed with overturning restraint provided by the corners (i.e., perpendicular walls). Where hold-downs were installed at each wall segment, the segmented shear wall method was used.

Methods 1 and 2 are forms of the PSW method and represent an engineered design approach. Method 1 uses the PSW method outlined by Sugiyama and Yasumura (see Appendix B), and does not place a limit on the aspect ratio of qualifying braced wall panels or on combining shear capacities of different sheathing materials (e.g., wood structural panel and gypsum wallboard). Method 2 uses the PSW method as applied in accordance with Section 2305.3.8.2 of the *2006 International Building Code* (see Appendix B). The IBC places a limit on the maximum aspect ratio for qualifying wall segments that can be included as part of the perforated shear wall. In wind design, this limit is set at 3.5:1 for walls sheathed with wood structural panels and 1.5:1 for walls sheathed with gypsum only (2:1 is permitted for blocked construction). For seismic design, the aspect ratio limit is also 3.5:1, but a further shear capacity reduction factor must be applied to wall segments with aspect ratios greater than 2:1. For the purposes of this study, Method 2 is used to analyze each house separately for wind and seismic design.

Method 3 follows the design methodology used to develop Public Comment 2 to RB148. Each qualifying wall segment is multiplied by a nominal design capacity, which is then adjusted by a partial restraint factor depending on the boundary conditions above the wall. A set of sample calculations are provided in Appendix B. As in Method 2, only the contribution of those wall segments having an aspect ratio less than a maximum allowable limit is included. For Method 3, the maximum aspect ratio ranges between 4:1 and 2:1, depending on the height of the opening next to the wall segment. Method 3 also included a system effect factor ranging between 1.2 and 1.5. However, because the purpose of this analysis is to arrive at a system effect factor, it was not included in the calculations. Method 3 was not used for shake-table studies because it is a wind design method.

In tests where hold-down anchors were installed along the length of the walls creating separate, fully-restrained segments, a segmented shear wall design method was used per the 2006 IBC (Method 4 above).

# Report 1: Whole Structure Testing and Analysis of a Light-Frame Wood Building (CSIRO)

## General Construction

A one-story, L-shaped house was tested to investigate the response of a light-framed structure under cyclic loading. The house had a footprint consisting of a 23-foot by 37-foot main portion and a 20-foot wide by 6.7-foot long extension at the northwest corner. Figure 1 shows a schematic of the building's wall layout and Table 2 provides a summary of the materials and construction methods used in the test house. In the direction of loading, the exterior walls of the building were continuously sheathed with plywood including areas above and below all openings.

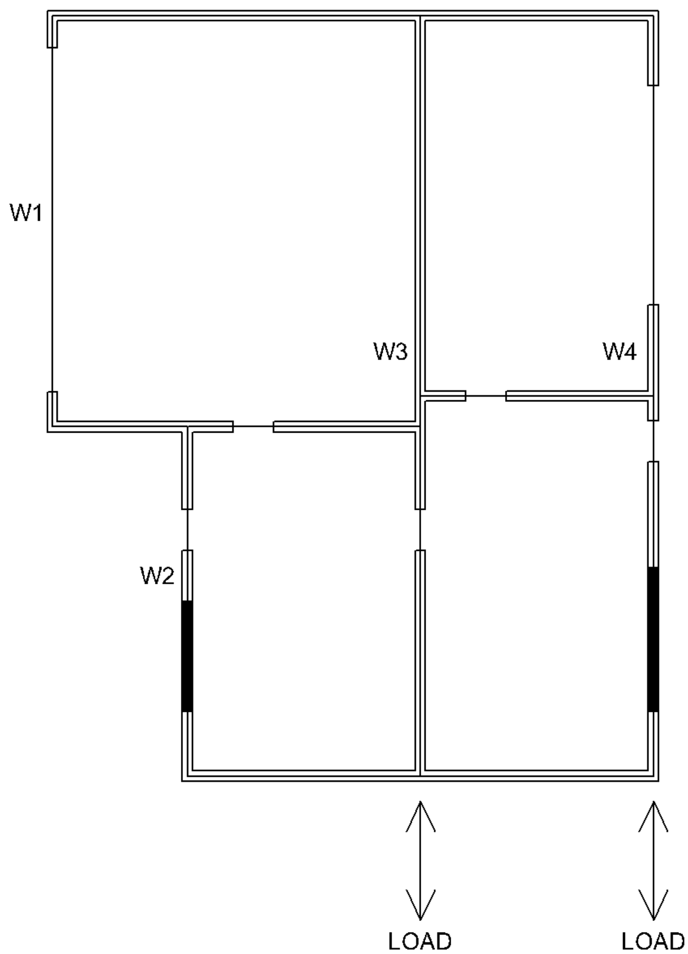


**Figure 1 - Layout of walls in CSIRO house**

**Table 2 - Building Materials and Construction Methods for CSIRO House**

| Component | Materials and Construction |
|---|---|
| Wall Framing | Radiata Pine 3.5-inch x 1.4-inch studs at 15.7 inches on center |
| Roof Framing | Pre-fabricated wood trusses at 24 inches on center |
| Exterior Wall Sheathing/Bracing | 3/8-inch plywood nailed at 6 inches around perimeter and 12 inches in the field (2-inch long x 0.113-inch dia. nails) |
| Roof Sheathing | 1/2-inch plywood nailed at 6 inches around perimeter and 12 inches in the field (2-inch long x 0.113-inch dia. nails) |
| Interior Sheathing (walls and ceiling) | 1/2-inch gypsum attached with 6g x 1.2 inch screws at 12 inches to studs with 6 inch clearance to top and bottom plates, installed horizontally, unblocked |
| Wall Anchorage/ Hold-downs | 1/2-inch anchor bolts at approximately 3.3 feet on center, no hold downs, corner framing present |
| Exterior Finishes/Fenestration | None |

## Test Methods/Protocol

Cyclic load was applied at the top of two walls (W3 and W4) using hydraulic jacks. The structure was tested to failure and the ultimate shear load was recorded using three-dimensional load cells placed underneath the bottom plates of the walls at approximately 3-foot spacing. Base shear was calculated as the sum of the forces at the base of each wall in the direction of loading.

## Analysis

Tables 3, 4, 5, and 6 provide summaries of the analysis of the predicted shear strength for the CSIRO building using the PSW methods and the RB148 method. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid roof diaphragm with all walls reaching their peak shear capacity at the same time. This is a conservative assumption as Walls W1 and W2 had not failed at the end of the test and it is unknown whether they had reached peak capacity.

2. Even though load was applied eccentrically, it was assumed that the diaphragm had no torsional response, i.e., the perpendicular walls did not contribute to the overall shear strength.

3. The measured peak total base shear was used for analysis.

4. Methods 1 and 2 (PSW) assumed full restraint at the ends of walls even though hold-down brackets were not installed. It was assumed that dead load stabilizing moment and corner framing were sufficient to resist uplift.

5. Radiata Pine lumber has a specific gravity equivalent to that of Southern Yellow Pine lumber, SG = 0.50.

6. The nominal unit shear capacity of 3/8-inch exterior plywood sheathing nailed at 6 inches around the perimeter and 12 inches in the field is 560 plf, per the *AF&PA Special Design Provisions for Wind and Seismic– 2005 Edition*.

7. There are no code recognized unit shear capacities for 1/2-inch interior gypsum sheathing attached with screws at 12 inches along the studs with 6-inch clearance from the top and bottom plates. A nominal unit shear capacity of 100 plf was chosen as a reasonable value for this sheathing configuration. (As a benchmark, 1/2-inch gypsum sheathing attached at 12 inches in the field and 8 inches around the perimeter, unblocked, has a unit shear capacity of 120 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.)

8. Per Section 2305.3.9 of the 2006 IBC, for wind design, the shear capacity of a wall segment sheathed with wood structural panels and gypsum sheathing on opposite faces is the sum of the unit shear capacities of each face. For seismic design, only the unit shear capacity of the wood structural panel is counted towards the shear capacity of the wall.

**Table 3 - Predicted Shear Strength of CSIRO House Using Method 1 (Sugiyama's PSW)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Openings (ft) | Total Length of Full Height Sheathed Segments (ft) | $C_{op}$[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| W1 | OSB & Gypsum | 20.2 | 8 | 16.4 | 3.8 | 0.08 | 660 | 1,102 |
| W2 | OSB & Gypsum | 16.4 | | 9.4 | 7.0 | 0.29 | 660 | 3,112 |
| W3 | Gypsum (both sides) | 36.4 | | 4.3 | 32.1 | 0.75 | 200 | 5,486 |
| W4 | OSB & Gypsum | 37.0 | | 21.2 | 15.7 | 0.26 | 660 | 6,352 |

[1]Adjustment factor calculated using Sugiyama equation, $C_{op} = r / (3-2r)$

| | |
|---|---|
| **Total Calculated Shear Strength** | **16,052** |
| **Total Tested Shear Strength** | **24,700** |
| **System Factor** | **1.54** |

**Table 4 - Predicted Shear Strength of CSIRO House Using Method 2 (2006 IBC PSW) w/ Wind Design Requirements**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | PSW Adjustment Factor, $C_o$[2] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| W1[3] | OSB & Gypsum | N/A | 8 | 0.0 | N/A | 660 | 0 |
| W2[4] | OSB & Gypsum | 2.6 | | 2.6 | 1.00 | 660 | 1,699 |
| W3 | Gypsum (both sides) | 36.4 | | 32.1 | 0.86 | 200 | 5,529 |
| W4 | OSB & Gypsum | 37.0 | | 15.7 | 0.55 | 660 | 5,709 |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio included

[2]Value interpolated from Table 2305.3.8.2 in 2006 IBC

[3]End segments do not meet requirements for qualifying PSW segments

[4]Only one segment meets requirements for a qualifying PSW segment. It is designed as an isolated, fully-restrained shear wall segment (conservative assumption)

| | |
|---|---|
| **Total Calculated Shear Strength** | **12,937** |
| **Total Tested Shear Strength** | **24,700** |
| **System Factor** | **1.91** |

**Table 5 - Predicted Shear Strength of CSIRO House Using Method 2 (2006 IBC PSW) w/ Seismic Design Requirements**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | Aspect Ratio Adjustment Factor[2] | PSW Adjustment Factor, $C_o$[3] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| W1[4] | OSB & Gypsum | N/A | 8 | 0.0 | N/A | N/A | 560 | 0 |
| W2[5] | OSB & Gypsum | 2.6 | | 2.6 | 0.65 | 1.0 | 560 | 937 |
| W3 | Gypsum (both sides) | 36.4 | | 32.1 | 1.0 | 0.86 | 200 | 5,529 |
| W4 | OSB & Gypsum | 37.0 | | 15.7 | 0.99 | 0.55 | 560 | 4,796 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **11,262** |
| **Total Tested Shear Strength** | **24,700** |
| **System Factor** | **2.19** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio included

[2]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 must be reduced by 2w/h

[3] Value interpolated from Table 2305.3.8.2 in 2006 IBC

[4]End segments do not meet requirements for qualifying PSW segments

[5]Only one segment meets requirements for a qualifying PSW segment, therefore it is designed as an isolated, fully-restrained shear wall segment (conservative assumption)

**Table 6 - Predicted Shear Strength of CSIRO House Using Method 3 (PC2 RB148)**

| Wall Label | Sheathing | Total Length of Qualifying PSW Segments[1] (ft) | Partial Restraint Factor (Roof only) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|
| W1 | OSB & Gypsum | 3.8[2] | 0.8 | 660 | 2,006 |
| W2 | OSB & Gypsum | 2.6 | | 660 | 1,357 |
| W3 | Gypsum (both sides) | 32.2 | | 200 | 5,144 |
| W4 | OSB & Gypsum | 15.7 | | 660 | 8,315 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **16,822** |
| **Maximum Base Shear** | **24,700** |
| **System Factor w/ W1 included** | **1.47** |
| **System Factor w/o W1** | **1.67** |

[1]Only full height sheathed segments meeting requirements of Table R602.10.5 in 2006 IRC are included

[2]Assumed that W1 wall segments were intended to meet requirement of note 2 in Table R602.10.5

## Results

Table 7 provides a summary of comparisons between the ultimate tested shear strength of the CSIRO test house and the predicted ultimate shear strength calculated by engineering analysis, including the corresponding system effect factor.

**Table 7 - Results of Strength Comparison and System Effect Factor for the CSIRO Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Peak Shear Strength of House (lb) | Whole House System Effect Factor |
|---|---|---|---|
| **PSW Method w/ Sugiyama Equation** | 16,052 | 24,700 | 1.54 |
| **PSW Method w/ 2006 IBC Wind Design Provisions** | 12,937 | | 1.91 |
| **PSW Method w/ 2006 IBC Seismic Design Provisions** | 11,262 | | 2.19 |
| **RB148 Method w/ W1 segments included** | 16,822 | | 1.47 |
| **RB148 Method w/o W1 segments** | 14,790 | | 1.67 |

The conventionally built one-story CSIRO house performed approximately 50% better than predicted by the Sugiyama perforated shear wall method that places no restrictions on wall aspect ratios or detailing. When the applicable building code restrictions were applied, as in Method 2 using wind or seismic provisions, the system effect increased further to 1.91 and 2.19, respectively. The analysis in accordance with Method 3 indicated system factors of 1.47 and 1.67, supporting the use of a 1.5 system factor for a one-story house or a top story in Public Comment 2 to RB148.

# Report 2: Shake Table Tests of a Two-Story Wood-frame House (CUREE Wood-frame Project)

## General Construction

The objective of the CUREE Wood-frame Project was to investigate the dynamic response of light-frame wood structures to uni-directional seismic loading. The research included several phases of testing of a two-story house with a 16-foot wide by 20-foot long rectangular floor plan. Construction methods ranged from fully engineered with uplift hold-downs and straps to conventional light-framed construction. Two of the tested phases are evaluated in this report: Phase 8 and Phase 10. Phase 8 was intended to represent conventional construction, whereas Phase 10 was a fully engineered system with hardware and all finishes installed including exterior stucco and fenestration. Figure 2 shows a schematic of the building's wall layout for both phases. Table 8 provides a summary of the materials and constructions methods used for Phase 8 and Phase 10. In the direction of loading, the exterior walls of the building were continuously sheathed with plywood including areas above and below all openings. Neither Phase 8 nor Phase 10 tests reached a failure of the system.

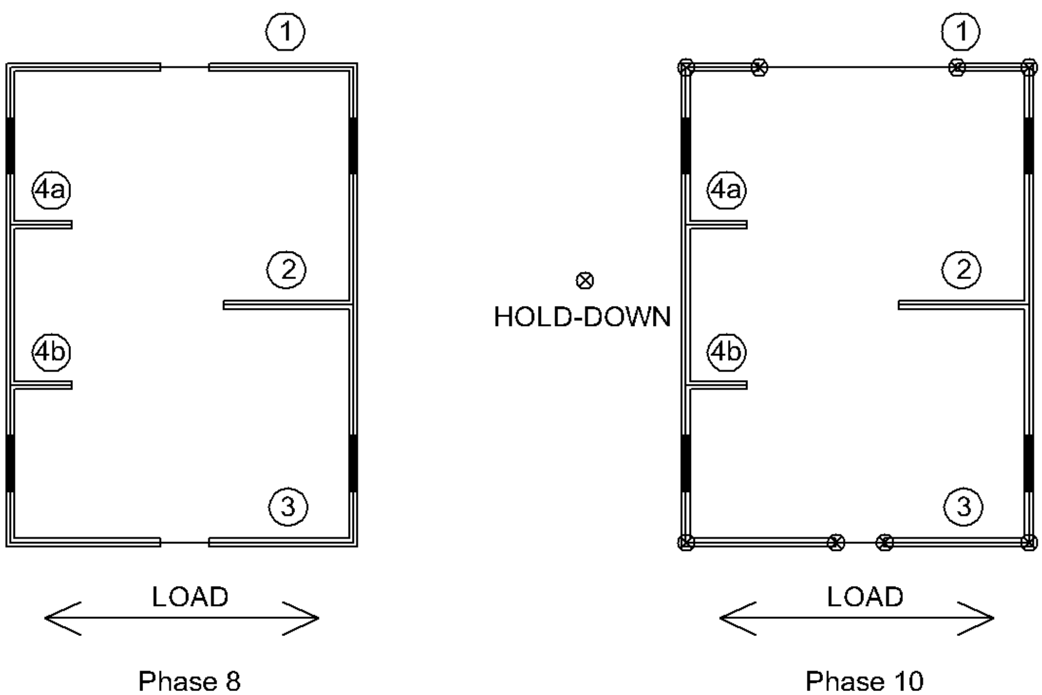


**Figure 2 - Layout of walls in CUREE project house**

**Table 8 - Building Materials and Construction Methods -- CUREE Project (Phases 8 and 10)**

| Component | Materials and Construction |
|---|---|
| Wall Framing | Douglas Fir 2x4 nominal studs at 16 inches on center |
| Floor Framing | 2x10 nominal joists at 16 inches on center |
| Roof Framing | Pre-fabricated wood trusses at 24 inches on center |
| Exterior Wall Sheathing/Bracing | Phase 8: 7/16-inch plywood nailed at 6 inches around perimeter and 12 inches in the field (8d box nails, 2.5-inches long x 0.113-inch diameter)<br>Phase 10: Wall 1 – 7/16-inch plywood nailed at 3 inches around perimeter and 12 inches in the field (8d box nails, 2.5-inches long x 0.113-inch diameter)<br>Wall 3 – 7/16-inch plywood nailed at 6 inches around perimeter and 12 inches in the field (8d box nails, 2.5-inches long x 0.113-inch diameter) |
| Floor Sheathing | 3/4-inch tongue and groove plywood nailed at 6 inches around the perimeter and 10 inches in the field (10d box nails, 3-inches long x 0.128-inch diameter), glued |
| Roof Sheathing | 1/2-inch plywood nailed at 6 inches around perimeter and 12 inches in the field (8d box gun nails, 2.5-inches long x 0.113-inch diameter), no blocking |
| Interior Sheathing | Phase 8: None<br>Phase 10: 1/2-inch gypsum attached at 16 inches on center (1.25 inch long screws), installed horizontally, blocked, taped and mudded |
| Wall Anchorage/Hold-downs | Phase 8: 1/2-inch anchor bolts 6 feet on center; no hold-downs<br>Phase 10: HTT22 hold-downs at the end of each shear wall segment, two 1/2-inch anchor bolts spaced evenly between hold-downs along each shear wall segment |
| Exterior Finishes/Fenestration | Phase 8: None<br>Phase 10: 7/8-inch stucco over 17 gauge wire lath; aluminum framed windows and pedestrian door |

## Test Methods/Protocol

Load was applied using a uni-directional shake table. For Phase 8, the seismic motion input was the Canoga Park ground motion record from the 1994 Northridge earthquake with an amplitude scaling factor of 1.2, resulting in a maximum peak ground acceleration of 0.50g. For Phase 10, the seismic motion input was the Rinaldi ground motion record from the 1994 Northridge earthquake with an amplitude scaling factor of 1.0, resulting in a maximum peak ground acceleration of 0.89g. Base shear was determined through calculations using accelerometer readings and the mass of the building. Testing during Phase 8 resulted in an approximate roof displacement of 2.5 inches, which did not result in specimen failure. Testing during Phase 10 resulted in an approximate roof displacement of 1.0 inch, which again did not result in failure of the specimen.

## Analysis of Phase 8

Tables 9 and 10 provide summaries of the analysis of the predicted shear strength for the Phase 8 building using the PSW methods. All qualifying shear walls had aspect ratios of less than 2:1. Therefore the predicted strength of the house using Methods 1 and 2 is the same for both wind and seismic design. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid roof diaphragm with all walls reaching their peak shear capacity at the same time.
2. Methods 1 and 2 (PSW) assumed full restraint at the ends of walls even though hold-down brackets were not installed. It was assumed that dead load stabilizing moment and corner framing were sufficient to resist uplift.
3. Doug Fir-Larch lumber with a SG = 0.50 was used in construction.
4. The nominal unit shear capacity of 7/16-inch exterior plywood sheathing nailed at 6 inches around the perimeter and 12 inches in the field with 8d galvanized box nails is 670 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*. Although the CUREE test report does not specify that the nails were galvanized, this report conservatively uses the value for galvanized box nails to avoid introducing an inadvertent increase in the system factor. Moreover, 8d gun nails are typically only 2-3/8 inches long, not 2.5 inches long as required for 8d box nails with a shank diameter of 0.113 inches.

**Table 9 - Predicted shear strength of CUREE house (Phase 8) using Method 1 (Sugiyama's PSW)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Openings (ft) | Total Length of Full Height Sheathed Segments (ft) | $C_{op}$ [1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| W1 | OSB | 16.0 | 8 | 3.0 | 13.0 | 0.63 | 670 | 6,797 |
| W2 | none | 5.33 | | 0 | 0 | N/A | N/A | N/A |
| W3 | OSB | 16.0 | | 3.0 | 13.0 | 0.63 | 670 | 6,797 |
| W4a & 4b | none | 4.67 | | 0 | 0 | N/A | N/A | N/A |
| | | | | | **Total Calculated Shear Strength** | | | **13,600** |
| | | | | | **Maximum Base Shear** | | | **22,700** |
| | | | | | **System Factor** | | | **1.67** |

[1]Adjustment factor calculated using Sugiyama equation, $C_{op} = r / (3-2r)$

**Table 10 - Predicted Shear Strength of CUREE house (Phase 8) using Method 2 (2006 IBC PSW)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | PSW Adjustment Factor, $C_o$[2] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| W1 | OSB | 16.0 | 8 | 13.0 | 0.78 | 670 | 6,794 |
| W2 | none | 5.3 | | 0 | N/A | N/A | N/A |
| W3 | OSB | 16.0 | | 13.0 | 0.78 | 670 | 6,794 |
| W4a & 4b | none | 4.7 | | 0 | N/A | N/A | N/A |
| | | | | **Total Calculated Shear Strength** | | | **13,590** |
| | | | | **Maximum Base Shear** | | | **22,700** |
| | | | | **System Factor** | | | **1.67** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio included

[2]Value interpolated from Table 2305.3.8.2 in 2006 IBC

## Analysis of Phase 10

Tables 11, 12, and 13 summarize the results of the analysis for the Phase 10 construction using a segmented shear wall methodology (Method 4) with and without building code limitations. The segmented method was used because Phase 10 had hold-down anchors at each end of each individual OSB sheathed shear wall panel. The first floor shear walls were analyzed relative to the measured peak total base shear. The following design assumptions were used in the analysis of the Phase 10 building:

1. The house was analyzed assuming a rigid roof diaphragm with all walls reaching their peak shear capacity at the same time.
2. The diaphragm had no torsional response, i.e., the perpendicular walls did not contribute to the overall strength.
3. All walls, including the interior walls, were designed as individual shear wall segments that were fully restrained from uplift by hold-down anchors.
4. The nominal unit shear capacity of 7/16-inch exterior plywood sheathing is 670 plf where nailed at 6 inches around the perimeter and 12 inches in the field, and is 1260 plf where nailed at 3 inches around the perimeter and 12 inches in the field. Both of these values are per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
5. There are no code recognized unit shear capacities for 1/2-inch interior gypsum sheathing attached at 16 inches in the field with screws. A nominal unit shear capacity of 100 plf was chosen as a reasonable value for this sheathing configuration. (In comparison, 1/2-inch gypsum sheathing attached with screws at 12 inches in the field and 8 inches around the perimeter, blocked, has a unit shear capacity of 140 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.)
6. Per Section 2305.3.9 of the 2006 IBC, for wind design, the shear capacity of a wall segment sheathed with wood structural panels and gypsum sheathing on opposite faces is the sum of the unit shear capacities of each face. For seismic design, only the unit shear capacity of the wood structural panel is counted towards the shear capacity of the wall.

**Table 11 - Predicted Shear Strength of CUREE house (Phase 10) using Method 4 (segmented method) w/o aspect ratio limitations**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| W1 | OSB & Gypsum | 16.0 | 8 | 3 | 3 | 1,360 | 8,160 |
| W2 | Gypsum (both sides) | 5.3 | | 5.33 | N/A | 200 | 1,066 |
| W3 | OSB & Gypsum | 16.0 | | 6.5 | 6.5 | 770 | 10,010 |
| W4a & 4b[1] | Gypsum (both sides) | 4.7 | | 2.33 | 2.33 | 200 | 940 |
| | | | | | Total Calculated Shear Strength | | **20,176** |
| | | | | | Total Tested Shear Strength | | **34,700** |
| | | | | | System Factor | | **1.72** |

**Table 12 - Predicted Shear Strength of CUREE house (Phase 10) using Method 4 (segmented method) w/o seismic design requirements**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| W1 | OSB & Gypsum | 16.0 | 8 | 3 | 3 | 1,360 | 8,160 |
| W2 | Gypsum (both sides) | 5.3 | | 5.33 | N/A | 200 | 1,066 |
| W3 | OSB & Gypsum | 16.0 | | 6.5 | 6.5 | 770 | 10,010 |
| W4a & 4b | Gypsum (both sides) | 4.7 | | 2.33 | 2.33 | N/A[1] | N/A |
| | | | | | Total Calculated Shear Strength | | **19,263** |
| | | | | | Total Tested Shear Strength | | **34,700** |
| | | | | | System Factor | | **1.80** |

[1]Does not meet aspect ratio limit of < 2:1 for gypsum sheathing per Table R2305.3.4 of 2006 IBC

**Table 13 - Predicted Shear Strength of CUREE house (Phase 10) using Method 4 (segmented method) w/ seismic design requirements**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Aspect Ratio Adjustment Factor[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| W1 | OSB & Gypsum | 16.0 | 8 | 3 | 3 | 0.75 | 1,260 | 5,670 |
| W2 | Gypsum (both sides) | 5.3 | | 5.33 | N/A | 1.0 | 200 | 1,066 |
| W3 | OSB & Gypsum | 16.0 | | 6.5 | 6.5 | 1.0 | 670 | 8,710 |
| W4a & 4b | Gypsum (both sides) | 4.7 | | 2.33 | 2.33 | N/A | N/A | N/A[2] |

| | |
|---|---|
| **Total Calculated Shear Strength** | **15,446** |
| **Total Tested Shear Strength** | **34,700** |
| **System Factor** | **2.25** |

[1]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 is reduced by 2w/h

[2]Does not meet aspect ratio limit of < 2:1 for gypsum sheathing per Table R2305.3.4 of 2006 IBC

## Results

Tables 14 and 15 provide a summary of comparisons between the predicted ultimate shear strength and the ultimate shear strength of the CUREE test house from Phases 8 and 10, respectively, as well as the corresponding system effect factor.

**Table 14 - Results of Strength Comparison and System Effect Factor for Phase 8 of CUREE Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Shear Strength of House (lb) | Whole House System Effect Factor |
|---|---|---|---|
| PSW Method w/ Sugiyama Equation | 13,600 | 22,700[1] | 1.67 |
| PSW Method w/ 2006 IBC Wind and Seismic Design Provisions | 13,590 | | 1.67 |

[1]Test specimen was not taken to failure

**Table 15 - Results of Strength Comparison and Approximate System Effect Factor for Phase 10 of CUREE Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Shear Strength of House (lb) | Whole House System Effect Factor |
|---|---|---|---|
| Segmented Method w/o Seismic Limitations | 20,176 | 34,700[1] | 1.72 |
| Segmented Method w/o 2006 IBC Seismic Design Provisions | 19,263 | | 1.80 |
| Segmented Method w/ 2006 IBC Seismic Design Provisions | 15,446 | | 2.25 |

[1]Test specimen was not taken to failure

A conventional CUREE house tested in Phase 8 without finishes resulted in a system factor of 1.67 for the PSW method of analysis. This estimate of the system factor is conservative because the house did not fail and presumably had additional reserve capacity. Also, uplift hardware required by the PSW method at the ends of every shear wall was not installed in Phase 8.

The Phase 10 house, which was engineered and finished including exterior stucco, windows and doors, resulted in a system factor ranging between 1.7 and 2.0. Again, the house had not yet reached failure at the end of the test and had only exhibited minor cracking of the exterior stucco and interior gypsum sheathing at the corners of some openings, suggesting a significant reserve capacity.

# Report 3: Full-Sized House Cyclic Racking Test (BRANZ)

## General Construction

An existing, one-story house was tested to measure the performance of a residential structure in as-built conditions. The structure was a 42-foot by 20-foot manufactured Fletcher Homes house with construction typical of homes available in New Zealand circa 1990. Figure 3 shows a schematic of the building's wall layout and Table 16 provides a summary of the materials and construction methods used in the test house. It should be noted that no exterior sheathing was present. The exterior building envelope was provided with fiber-cement weatherboard cladding.

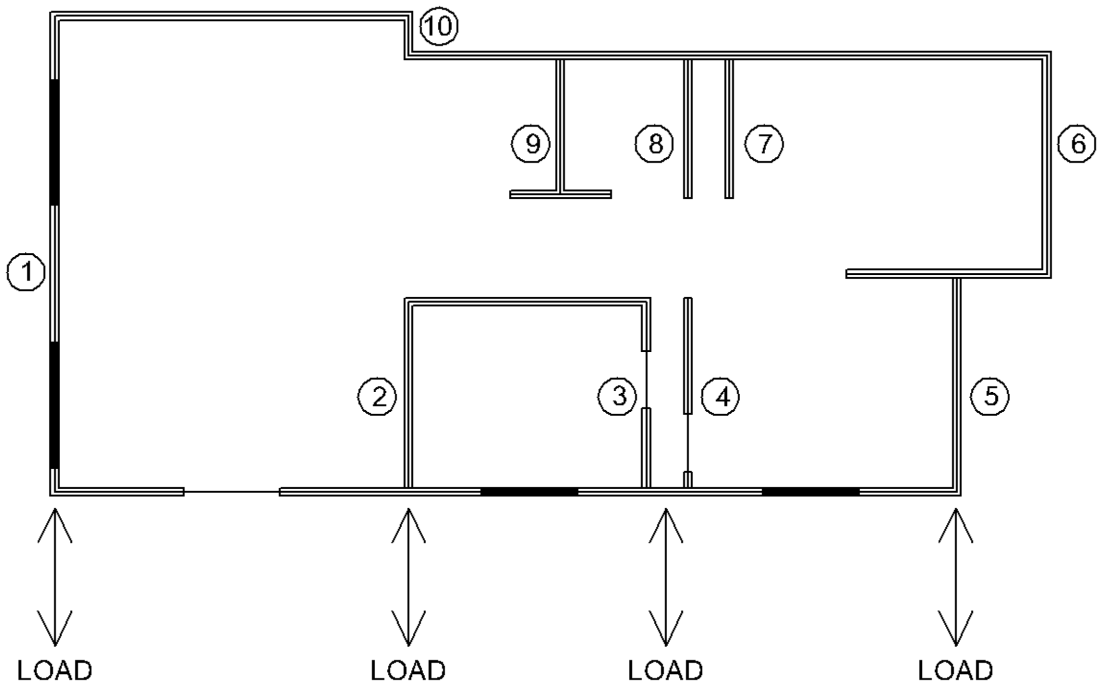


**Figure 3 - Layout of walls for BRANZ house**

**Table 16 - Building Materials and Construction Methods of BRANZ House**

| Component | Materials and Construction |
|---|---|
| Wall Framing | Radiata Pine 3.5-inch x 1.75-inch studs at 24 inches on center. |
| Roof Framing | Pre-fabricated wood trusses at 3.3 feet on center and 2.75-inch by 1.25-inch cross batons at 16 inches on center |
| Exterior Wall Sheathing/Bracing | none |
| Roof Sheathing | Corrugated metal decking |
| Interior Sheathing (walls) | 3/8-inch bracing grade fiber-reinforced gypsum installed vertically attached at 6 inches around the perimeter only (1.2-inch long x 0.098-inch dia. nails)(Attachment 1);<br>or<br>3/8-inch gypsum installed vertically with varying attachment patterns:<br>- Attachment 2: 12 inches around the perimeter and 24 inches in the field (1.2-inch long x 0.098-inch dia. nails)<br>- Attachment 3: 6 inches around the perimeter only (1.25-inch long screws)<br>- Attachment 4: 12 inches around the perimeter only (1.25-inch long screws) |
| Interior Sheathing (ceiling) | 3/8-inch bracing grade gypsum attached at 8-inches on center to cross batons (1.25-inch long screws) |
| Wall Anchorage/ Hold-downs | One 3.5-inch long x 0.129-inch dia. nail at approximately 24 inches on center, corner framing present |
| Exterior Finishes/Fenestration | 12-inch x 1/4-inch thick fiber-cement planks nailed to studs at 24 inches on center with 3.5-inch long x 0.156-inch dia. nails<br><br>residential windows and doors |

## Test Methods/Protocol

Load was applied to the top of walls 1 through 5 cyclically using hydraulic jacks. Load was distributed along the top of each wall using a timber distribution beam. The structure was tested to failure and the ultimate shear load was measured using load cells placed between the hydraulic jacks and the loading beams. Base shear was calculated as the sum of the forces applied by each jack.

## Analysis

Tables 17, 18, and 19 provide summaries of the analysis of the predicted shear strength of the structure using the PC2/RB148 method (Method 3) and the segmented design method (Method 4). Only gypsum sheathing is used as bracing in this building. The maximum limit on the aspect ratio for gypsum is 2:1 per Table R2305.3.4 of the 2006 IBC for both wind and seismic design. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid diaphragm, with all walls reaching their peak shear capacity at the same time.
2. The measured peak total base shear was used for analysis.
3. Where walls were designed as segmented shear walls fully restrained at the ends, it was assumed that dead load stabilizing moment and corner framing were sufficient to resist uplift even though hold-down brackets were not installed.
4. Wall double top plates acted as shear collectors.
5. Radiata Pine lumber has a specific gravity equivalent to that of Southern Yellow Pine lumber, SG = 0.50.
6. The nominal unit shear capacities used are per the manufacturers specifications noted in *Full-Sized House Cyclic Racking Test – BRANZ Report No. 119* and are as follows:
    a. 3/8-inch bracing grade fiber-reinforced gypsum sheathing nailed at 6 inches around the perimeter is 395 plf.
    b. 3/8-inch gypsum sheathing nailed at 12 inches around the perimeter and 24 inches in the field is 95 plf.
    c. 3/8-inch gypsum sheathing attached with screws at 6 inches around the perimeter only is 220 plf.
    d. 3/8-inch gypsum sheathing attached with screws at 12 inches around the perimeter only is 110 plf.

**Table 17 - Predicted Shear Strength of BRANZ Test House Using Method 4 (segmented method) w/o Aspect Ratio Limitations**

| Wall Label | Sheathing (gypsum w/ varying attachment schedules)[1] | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 1a | Attach 4 – 1 side only | 1.0 | 8 | 1.0 | N/A | 110 | 110 |
| 1b | Attach 1 – 1 side only | 5.4 | | 5.4 | N/A | 395 | 2,133 |
| 1c | Attach 4 – 1 side only | 2.3 | | 2.3 | N/A | 110 | 253 |
| 2 | Attach 4 – side 1<br>Attach 2 – side 2 | 7.9 | | 7.9 | N/A | 490 | 3,871 |
| 3 | Attach 2 – both sides | 7.9 | | 2.75 | 2.75 | 190 | 1,045 |
| 4 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.2 | | 5.2 | N/A | 315 | 1,638 |
| 5 | Attach 4 – 1 side only | 7.9 | | 7.9 | N/A | 110 | 869 |
| 6 | Attach 3 – 1 side only | 9.1 | | 9.1 | N/A | 220 | 2,002 |
| 7 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 5.8 | N/A | 205 | 1,189 |
| 8 | Attach 4 – both sides | 5.8 | | 5.8 | N/A | 190 | 1,102 |
| 9 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 5.8 | N/A | 205 | 1,189 |
| 10 | Attach 4 – 1 side only | 1.7 | | 1.7 | N/A | 110 | 187 |
| | | | | | **Total Calculated Shear Strength** | | **15,588** |
| | | | | | **Maximum Base Shear** | | **27,450** |
| | | | | | **System Factor** | | **1.76** |

[1]See Table 15 for explanation of sheathing attachment schedules

**Table 18 - Predicted Shear Strength of BRANZ Test House Using Method 4 (segmented method) w/ Aspect Ratio Limitations**

| Wall Label | Sheathing (gypsum w/ varying attachment schedules)[1] | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 1a | Attach 4 – 1 side only | 1.0 | 8 | 1.0 | N/A | 110 | N/A[2] |
| 1b | Attach 1 – 1 side only | 5.4 | | 5.4 | N/A | 395 | N/A[2] |
| 1c | Attach 4 – 1 side only | 2.3 | | 2.3 | N/A | 110 | N/A[2] |
| 2 | Attach 4 – side 1<br>Attach 2 – side 2 | 7.9 | | 7.9 | N/A | 490 | 3,871 |
| 3 | Attach 2 – both sides | 7.9 | | 2.75 | 2.75 | 190 | N/A[2] |
| 4 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.2 | | 5.2 | N/A | 315 | 1,638 |
| 5 | Attach 4 – 1 side only | 7.9 | | 7.9 | N/A | 110 | 869 |
| 6 | Attach 3 – 1 side only | 9.1 | | 9.1 | N/A | 220 | 2,002 |
| 7 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 5.8 | N/A | 205 | 1,189 |
| 8 | Attach 4 – both sides | 5.8 | | 5.8 | N/A | 190 | 1,102 |
| 9 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 5.8 | N/A | 205 | 1,189 |
| 10 | Attach 4 – 1 side only | 1.7 | | 1.7 | N/A | 110 | N/A[2] |

| | |
|---|---|
| **Total Calculated Shear Strength** | **11,860** |
| **Maximum Base Shear** | **27,450** |
| **System Factor** | **2.31** |

[1]See Table 15 for explanation of sheathing attachment schedules

[2]Does not meet aspect ratio limit of < 2:1 for gypsum sheathing per Table R2305.3.4 of 2006 IBC

**Table 19 - Predicted Shear Strength of BRANZ Test House Using Method 3 (PC2 RB148)**

| Wall Label | Sheathing (gypsum w/ varying attachment schedules)[1] | Total Length of Qualifying PSW Segments[2] (ft) | Partial Restraint Factor (Roof only) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|
| 1a | Attach 4 – 1 side only | 0 | 0.8 | 110 | 0 |
| 1b | Attach 1 – 1 side only | 0 | | 395 | 0 |
| 1c | Attach 4 – 1 side only | 0 | | 110 | 0 |
| 2 | Attach 4 – side 1<br>Attach 2 – side 2 | 7.9 | | 490 | 3,097 |
| 3 | Attach 2 – both sides | 0 | | 190 | 0 |
| 4 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.2 | | 315 | 1,310 |
| 5 | Attach 4 – 1 side only | 7.9 | | 110 | 695 |
| 6 | Attach 3 – 1 side only | 9.1 | | 220 | 1,602 |
| 7 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 205 | 951 |
| 8 | Attach 4 – both sides | 5.8 | | 190 | 882 |
| 9 | Attach 2 – side 1<br>Attach 3 – side 2 | 5.8 | | 205 | 951 |
| 10 | Attach 4 – 1 side only | 0 | | 110 | 0 |
| | | | | **Total Calculated Shear Strength** | **9,488** |
| | | | | **Maximum Base Shear** | **27,450** |
| | | | | **System Factor** | **2.89** |

[1]See Table 15 for explanation of sheathing attachment schedules

[2]Only full height sheathed segments meeting < 2:1 aspect ratio included

## Results

Table 20 provides a summary of comparisons between the ultimate shear strength of the BRANZ Project test house during testing and the predicted shear strength calculated using three different design methods, as well as the corresponding system effect factor.

**Table 20 - Results of Strength Comparison and System Effect Factor for BRANZ Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Peak Shear Strength of House (lb) | Whole House System Effect Factor |
|---|---|---|---|
| **Segmented Method w/o limiting design provisions** | 16,044 | 27,450 | 1.71 |
| **Segmented Method w/ 2006 IBC Wind and Seismic Design Provisions** | 11,860 | | 2.31 |
| **RB148 Method** | 9,488 | | 2.89 |

Analysis of the testing done by the BRANZ Institute on an existing, conventionally framed house indicates a system factor ranging between 1.7 and 2.9 based on the selected design method. Note that both design methods are predicated on the assumption of hold-downs at the ends of each segment, yet uplift hardware was not installed in this house.

# Report 4: Seismic Testing of a Full-Scale Two-Story Light-Frame Wood Building (NEESWood)

## General Construction

The NEESWood Benchmark Test project investigated the dynamic response of a two-story, 56-foot by 23-foot wood-frame building with varying levels of finishes to several levels of seismic ground motions. The degree of completion of the test house ranged from a wood structural shell with only the stud framing sheathed with OSB on the exterior to a fully-finished house with windows and doors installed. For the purpose of this study, only the results from the bare frame structure with only exterior OSB sheathing (Phase 1) and the fully finished house test (Phase 5) are analyzed for system effects. Figure 4 shows a schematic of the building's wall layout and Table 21 provides a summary of the materials and constructions methods used in the test house. The house was designed to be representative of a production house built in the 1980's and 1990's. The structural design of the house was in accordance with the 1988 UBC. The walls were continuously sheathed with OSB, including areas above and below all openings.

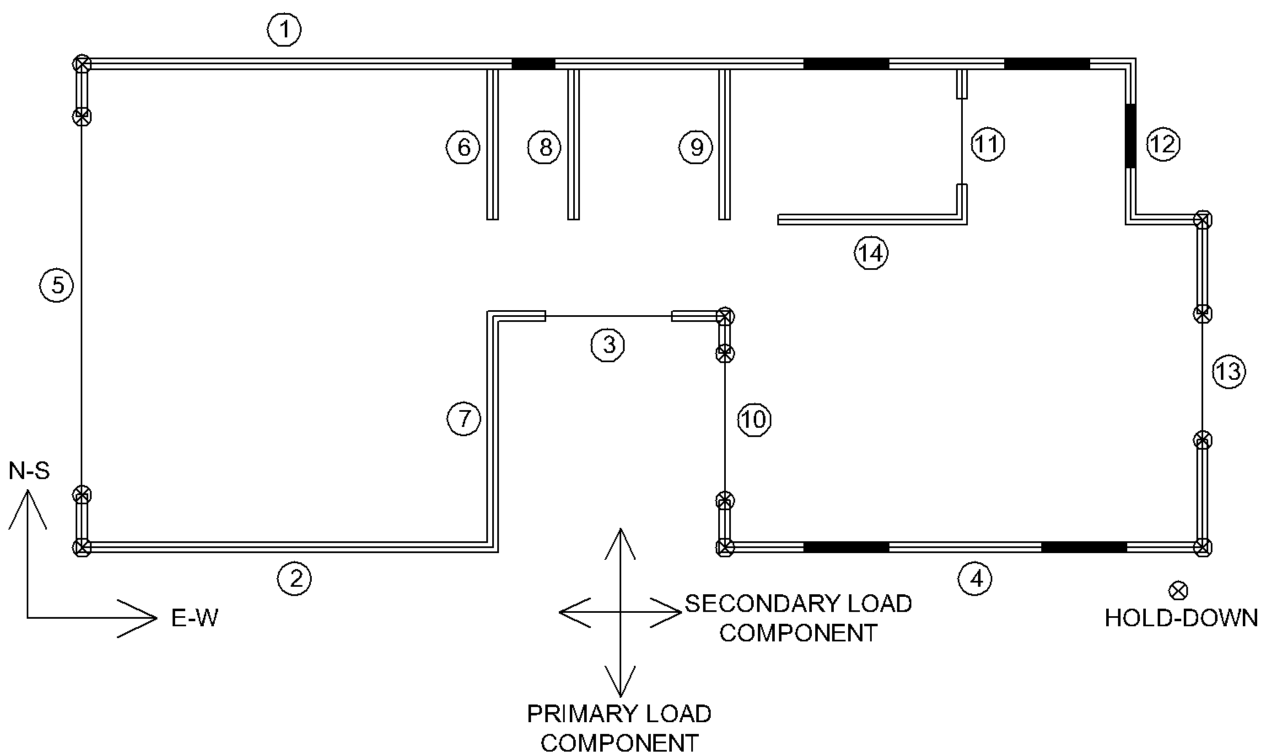


**Figure 4 - Shear wall layout of NEESWood Project test house**

**Table 21 - Building Materials and Construction Methods of NEESWood Project House**

| Component | Materials and Construction |
|---|---|
| Wall Framing | Hem Fir 2x4 nominal studs at 16 inches on center (except walls of garage which were 2x6 nominal studs) |
| Floor Framing | 2x12 nominal joists at 16 inches on center |
| Roof Framing | Pre-fabricated wood trusses at 24 inches on center |
| Exterior Wall Sheathing/Bracing | 7/16-inch OSB nailed at either 3, 4 or 6 inches around the perimeter and 12 inches in the field (8d common nails, 2.5-inches long x 0.131-inch diameter) |
| Floor Sheathing | 3/4-inch tongue and groove OSB nailed at 6 inches around the perimeter and 10 inches in the field (10d common nails, 3-inches long x 0.148-inch diameter), glued |
| Roof Sheathing | 1/2-inch plywood nailed at 6 inches around the perimeter and 12 inches in the field (8d common nails, 2.5-inches long x 0.131-inch diameter), no blocking |
| Interior Sheathing (walls) | Phase 1 – none<br>Phase 5 – 1/2-inch gypsum attached at 16 inches on center (1.25 inch long screws), installed horizontally, no blocking, taped and mudded |
| Interior Sheathing (ceilings) | Phase 1 – none<br>Phase 5 – 1/2-inch gypsum attached at 12 inches on center (1.25 inch long screws), installed horizontally, no blocking, taped and mudded |
| Wall Anchorage/ Hold-downs | PHD2 Simpson Strong-tie hold-down anchors at the end of each shear wall segment (only in N-S direction) and 1/2-inch anchor bolts at 6 feet on center (everywhere), corner framing present |
| Exterior Finishes/Fenestration | 7/8-inch stucco over 16 gauge wire lath; Tile roofing, windows, pedestrian and garage doors also installed |

## Test Methods/Protocol

Five different levels of seismic loading were applied using two linked tri-axial shake tables. Two different ground motion records were used for the seismic motion input, both recorded during the 1994 Northridge earthquake. The first ground motion was recorded at Canoga Park and was used for the first four levels of loading with amplitude scaling factors from 0.12 to 1.2 and peak ground accelerations from 0.05g to 0.50g in the North-South direction. The second ground motion record was recorded at Rinaldi and was used for the Phase 5 loading level with ground acceleration of 0.84g in the North-South direction. A perpendicular ground motion component was applied in the East-West direction. A corresponding East-West component ranged between 0.04g and 0.43g peak ground acceleration for the four levels of the Canoga Park record, and was 0.45g for the Phase 5 Rinaldi record. A corresponding vertical component ranged from 0.06g to 0.59g peak ground acceleration for the Canoga Park record, and was 0.85g for the Rinaldi record.

Base shear was determined through calculations using acceleration readings from sensors placed at each major wall line and the corresponding tributary mass of the building. Phase 1 testing was only taken to the second level of ground motion (i.e., amplitude scaling factor of 0.53), which resulted in an approximate roof displacement of 2.5 inches and only very minor splitting of the sill plates. Phase 5 testing resulted in an approximate roof displacement of 4.0 inches, but did not result in a catastrophic specimen failure. The observed damage consisted of diagonal cracking of the gypsum sheathing and exterior stucco at the corners of openings, as well as some splitting of the bottom plates and vertical hold-down studs.

## Analysis

Tables 22 through 27 provide summaries of the analysis of the predicted shear strength for the tested building using a combination of the PSW method and segmented shear wall method based on the detailing provided for individual walls. The measured peak base shear was compared to the performance of the first floor shear walls. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid diaphragm, with all walls reaching their peak shear capacity at the same time.
2. The North-South direction of loading was analyzed – the direction of the primary component of the ground motion record.
3. Three out of the five shear wall lines in the North-South direction were designed as fully-restrained, segmented shear walls, because hold-down anchors were installed at the ends of each fully sheathed segment (see Figure 3). The fourth (walls 6 & 7) and fifth (wall 12) wall lines that did not have hold-downs anchors installed were analyzed as perforated shear walls. It was assumed that the dead load stabilizing moment and corner framing were sufficient to resist uplift. It should be noted that the corner framing was also resisting direct shear from ground motion component in the East-West direction.
4. Hem Fir lumber with a SG = 0.43 was used in construction. All nominal unit shear capacities were adjusted by 0.93 in accordance with note 3 in table 4.3A of *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
5. The nominal unit shear capacity of 7/16-inch OSB sheathing nailed at 6 inches around the perimeter and 12 inches in the field and adjusted for Hem-Fir framing is 623 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
6. The nominal unit shear capacity of 7/16-inch OSB sheathing nailed at 4 inches around the perimeter and 12 inches in the field and adjusted for Hem-Fir framing is 911 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
7. The nominal unit shear capacity of 7/16-inch OSB sheathing nailed at 3 inches around the perimeter and 12 inches in the field and adjusted for Hem-Fir framing is 1,172 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
8. There are no code recognized unit shear capacities for 1/2-inch interior gypsum sheathing attached at 16 inches in the field with screws. A nominal unit shear capacity of 100 plf was chosen as a reasonable value for this sheathing configuration. (In comparison, 1/2-inch gypsum sheathing attached with screws at 12 inches in the field

and 8 inches around the perimeter, blocked, has a unit shear capacity of 140 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.)

9. Per Section 2305.3.9 of the 2006 IBC, for wind design, the shear capacity of a wall segment sheathed with wood structural panels and gypsum sheathing on opposite faces is the sum of the unit shear capacities of each face. For seismic design, only the unit shear capacity of the wood structural panel is counted towards the shear capacity of the wall.

**Table 22 - Predicted shear strength of NEESWood test house (Phase 1) using Method 4 (segmented method w/o aspect ratio limitations) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) | 21.83 | 8 | 2.33 | 2.33 | 911 | 4,245 |
| 6 & 7[1] | OSB (6/12 nailing) | 23 | | 7.67 | 10.75 | 623 | 8,835 |
| 8 | None | 7.5 | | 7.25 | | N/A | 0 |
| 9 | OSB (4/12 nailing) | 7.5 | | 7.67 | | 911 | 6,987 |
| 10 | OSB (4/12 nailing) | 11 | | 2 | 2 | 911 | 3,644 |
| 11 | None | 7.25 | | 1.67 | 1.67 | N/A | 0 |
| 12[2] | OSB (6/12 nailing) | 7 | | 2 | 2 | 623 | 2,972 |
| 13 | OSB (3/12 nailing) | 15.5 | | 4.75 | 4.75 | 1,172 | 11,134 |
| | | | | | **Total Calculated Shear Strength** | | **37,817** |
| | | | | | **Total Tested Shear Strength** | | **41,000** |
| | | | | | **System Factor** | | **1.08** |

[1]Walls 6 & 7 did not have hold-downs. Therefore, walls were designed as perforated shear walls

[2]Wall 12 did not have hold-downs. Therefore, the wall was designed as a perforated shear wall

**Table 23 - Predicted shear strength of NEESWood test house (Phase 1) using Method 4 (segmented method w/o seismic design requirements) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) | 21.83 | 8 | 2.33 | 2.33 | 911 | 4,245 |
| 6 & 7[2] | OSB (6/12 nailing) | 23 | | 7.67 | 10.75 | 623 | 8,835 |
| 8 | None | 7.5 | | 7.25 | | N/A | 0 |
| 9 | OSB (4/12 nailing) | 7.5 | | 7.67 | | 911 | 6,987 |
| 10 | OSB (4/12 nailing) | 11 | | 2 | 2 | 911 | N/A[1] |
| 11 | None | 7.25 | | 1.67 | 1.67 | N/A | 0 |
| 12[3] | OSB (6/12 nailing) | 7 | | 2 | 2 | 623 | N/A[1] |
| 13 | OSB (3/12 nailing) | 15.5 | | 4.75 | 4.75 | 1,172 | 11,134 |
| | | | | | | **Total Calculated Shear Strength** | **31,201** |
| | | | | | | **Total Tested Shear Strength** | **41,000** |
| | | | | | | **System Factor** | **1.31** |

[1]Does not meet aspect ratio limit of < 3.5:1 for wood structural panels per Table R2305.3.4 of 2006 IBC

[2]Walls 6 & 7 did not have hold-downs. Therefore, the walls were designed as perforated shear walls

[3]Wall 12 did not have hold-downs. Therefore, the wall was designed as a single perforated shear wall

**Table 24 - Predicted shear strength of NEESWood test house (Phase 1) using Method 4 (segmented method w/ seismic design requirements) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Aspect Ratio Adjustment Factor[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) | 21.83 | 8 | 2.33 | 2.33 | 0.63 | 911 | 2,675 |
| 6 & 7[3] | OSB (6/12 nailing) | 23 | | 7.67 | 10.75 | 1.0 | 623 | 8,835 |
| 8 | None | 7.5 | | 7.25 | | N/A | N/A | 0 |
| 9 | OSB (4/12 nailing) | 7.5 | | 7.67 | | 1.0 | 911 | 6,987 |
| 10 | OSB (4/12 nailing) | 11 | | 2 | 2 | N/A[2] | 911 | N/A |
| 11 | None | 7.25 | | 1.67 | 1.67 | N/A | N/A | 0 |
| 12[4] | OSB (6/12 nailing) | 7 | | 2 | 2 | N/A[2] | 623 | N/A |
| 13 | OSB (3/12 nailing) | 15.5 | | 4.75 | 4.75 | 1.0 | 1,172 | 11,134 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **29,631** |
| **Total Tested Shear Strength** | **41,000** |
| **System Factor** | **1.38** |

[1]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 must be reduced by 2w/h

[2]Does not meet aspect ratio limit of < 3.5:1 for wood structural panels per Table R2305.3.4 of 2006 IBC

[3]Walls 6 & 7 did not have hold-downs. Therefore, the walls were designed as perforated shear walls

[4]Wall 12 did not have hold-downs. Therefore, the wall was designed as a single perforated shear wall

**Table 25 - Predicted Shear Strength of NEESWood Test House (Phase 5) Using Method 4 (segmented method w/o aspect ratio limitations) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) & Gypsum | 21.83 | 8 | 2.33 | 2.33 | 1011 | 4,711 |
| 6 & 7[3] | OSB (6/12 nailing) & Gypsum | 23 | | 7.67 | 10.75 | 723 | 10,253 |
| 8 | Gypsum (both sides) | 7.5 | | 7.25 | | 200 | 1,450 |
| 9 | OSB (4/12 nailing) & Gypsum | 7.5 | | 7.67 | | 1,011 | 7,754 |
| 10 | OSB (4/12 nailing) & Gypsum | 11 | | 2 | 2 | 1,011 | 4,044 |
| 11 | Gypsum (both sides) | 7.25 | | 1.67 | 1.67 | 200 | 667 |
| 12[4] | OSB (6/12 nailing) & Gypsum | 7 | | 2 | 2 | 723 | 2,972 |
| 13 | OSB (3/12 nailing) & Gypsum | 15.5 | | 4.75 | 4.75 | 1,272 | 12,084 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **43,935** |
| **Total Tested Shear Strength** | **79,000** |
| **System Factor** | **1.80** |

[3]Walls 6 & 7 did not have hold-downs. Therefore, the walls were designed as perforated shear walls

[4]Wall 12 did not have hold-downs. Therefore, the wall was designed as a single perforated shear wall

**Table 26 - Predicted Shear Strength of NEESWood Test House (Phase 5) Using Method 4 (segmented method w/o seismic limitations) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) & Gypsum | 21.83 | 8 | 2.33 | 2.33 | 1011 | 4,711 |
| 6 & 7[3] | OSB (6/12 nailing) & Gypsum | 23 | | 7.67 | 10.75 | 723 | 10,253 |
| 8 | Gypsum (both sides) | 7.5 | | 7.25 | | 200 | 1,450 |
| 9 | OSB (4/12 nailing) & Gypsum | 7.5 | | 7.67 | | 1,011 | 7,754 |
| 10 | OSB (4/12 nailing) & Gypsum | 11 | | 2 | 2 | 1,011 | N/A[1] |
| 11 | Gypsum (both sides) | 7.25 | | 1.67 | 1.67 | 200 | N/A[2] |
| 12[4] | OSB (6/12 nailing) & Gypsum | 7 | | 2 | 2 | 723 | N/A[1] |
| 13 | OSB (3/12 nailing) & Gypsum | 15.5 | | 4.75 | 4.75 | 1,272 | 12,084 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **36,252** |
| **Total Tested Shear Strength** | **79,000** |
| **System Factor** | **2.18** |

[1]Does not meet aspect ratio limit of < 3.5:1 for wood structural panels per Table R2305.3.4 of 2006 IBC
[2]Does not meet aspect ratio limit of < 2:1 for gypsum sheathing per Table R2305.3.4 of 2006 IBC
[3]Walls 6 & 7 did not have hold-downs. Therefore, the walls were designed as perforated shear walls
[4]Wall 12 did not have hold-downs. Therefore, the wall was designed as a single perforated shear wall

**Table 27 - Predicted Shear Strength of NEESWood Test House (Phase 5) Using Method 4 (segmented method w/ seismic design requirements) (N-S Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Length of Segment 1 (ft) | Length of Segment 2 (ft) | Aspect Ratio Adjustment Factor[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 5 | OSB (4/12 nailing) & Gypsum | 21.83 | 8 | 2.33 | 2.33 | 0.63 | 911 | 2,675 |
| 6 & 7[4] | OSB (6/12 nailing) & Gypsum | | | 7.67 | 10.75 | 1.0 | 623 | 8,835 |
| 8 | Gypsum (both sides) | 7.5 | | 7.25 | | 1.0 | 200 | 1,450 |
| 9 | OSB (4/12 nailing) & Gypsum | 7.5 | | 7.67 | | 1.0 | 911 | 6,987 |
| 10 | OSB (4/12 nailing) & Gypsum | 11 | | 2 | 2 | N/A[2] | 911 | N/A |
| 11 | Gypsum (both sides) | 7.25 | | 1.67 | 1.67 | N/A[3] | 100 | N/A |
| 12[5] | OSB (6/12 nailing) & Gypsum | 7 | | 2 | 2 | N/A[2] | 623 | N/A |
| 13 | OSB (3/12 nailing) & Gypsum | 15.5 | | 4.75 | 4.75 | 1.0 | 1,172 | 11,134 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **31,081** |
| **Total Tested Shear Strength** | **79,000** |
| **System Factor** | **2.55** |

[1]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 must be reduced by 2w/h

[2]Does not meet aspect ratio limit of < 3.5:1 for wood structural panels per Table R2305.3.4 of 2006 IBC

[3]Does not meet aspect ratio limit of < 2:1 for gypsum sheathing per Table R2305.3.4 of 2006 IBC

[4]Walls 6 & 7 did not have hold-downs. Therefore, the walls were designed as perforated shear walls

[5]Wall 12 did not have hold-downs. Therefore, the wall was designed as a single perforated shear wall

## Results

Table 28 provides a summary of comparisons between the predicted ultimate shear strength of the NEESWood Project test house and the ultimate tested shear strength of the house in the North-South direction, as well as the corresponding system effect factor.

**Table 28 - Comparison of predicted to tested strengths and system effect factor of NEESWood Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Shear Strength of House[1] (lb) | Whole House System Effect Factor |
|---|---|---|---|
| **Phase 1 – Frame w/ OSB** | | | |
| **Combined Segmented & PSW Method** | 37,817 | 41,000 | 1.08 |
| **Combined Segmented & PSW Method w/o 2006 IBC Seismic Design Provisions** | 31,201 | | 1.31 |
| **Combined Segmented & PSW Method w/ 2006 IBC Seismic Design Provisions** | 29,631 | | 1.38 |
| **Phase 5 – Finished House** | | | |
| **Combined Segmented & PSW Method w/o Aspect Ratio Limitations Provisions** | 43,935 | 79,000 | 1.80 |
| **Combined Segmented & PSW Method w/o 2006 IBC Seismic Design Provisions** | 36,252 | | 2.18 |
| **Combined Segmented & PSW Method w/ 2006 IBC Seismic Design Provisions** | 31,081 | | 2.55 |

[1]Test specimen was not taken to failure

A comparison of results from Phases 1 (no finishes) and 5 (fully finished) indicate that finishes contribute significantly to system effects. The predicted capacity of Phase 1 without finishes and with all segments included regardless of the aspect ratio resulted in the lowest system effect of 1.08. However, it should be noted that during Phase 1 the structure's roof deflection was limited to 2.5 inches indicating a remaining reserve capacity. As a point of comparison, Phase 5 resulted in a roof deflection of nearly 4 inches. Also note that light-frame wood shear walls reach peak strength at story drifts above 2.0 inches. When all applicable seismic aspect ratio limitations were included, the system factor increased to 1.39.

When interior sheathing and finishes were added, the corresponding system factor increased significantly. In part, it is attributed to a higher loading, i.e., the system was taken to its capacity. The Phase 5 house experienced nearly 4 inches of roof deflection and sustained damage, but did not reach failure and potentially had some additional reserve capacity. The observed damage consisted of diagonal cracking of the gypsum sheathing and exterior stucco at the corners of openings, as well as splitting of the bottom plates and vertical hold-down studs.

# Report 5: Assessment of Seismic Resistance of Conventional Wood-Frame Houses (Forintek Collaboration)

## General Construction

The Forintek Canada Corp., in collaboration with other industry researchers conducted a series of tests at Tongji University in China to investigate the seismic performance of a conventionally framed house. A two-story, wood-frame building with a square footprint of 20 feet by 20 feet was tested. Figure 5 shows a schematic of the building's wall layout. Table 29 provides a summary of the materials and construction methods used in the test house. The exterior walls of the building were continuously sheathed with OSB, including areas above and below all openings.

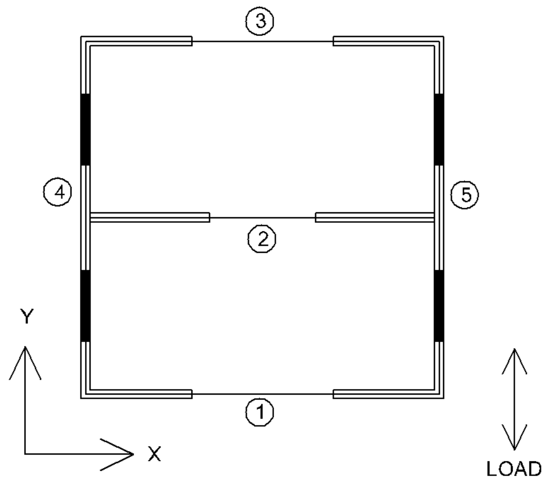


**Figure 5 - Wall layout of Forintek Collaboration project test house**

**Table 29 - Building Materials and Construction Methods of the Forintek Collaboration House**

| Component | Materials and Construction |
|---|---|
| Wall Framing | SPF No. 2 2x4 nominal studs at 16 inches on center |
| Floor Framing | JSI-20 wood I-joists at 16 inches on center |
| Roof Framing | Pre-fabricated wood trusses at 24 inches on center |
| Exterior Wall Sheathing/Bracing | 3/8-inch OSB nailed at 6 inches around the perimeter and 12 inches in the field (Twisted nails, 2.5-inches long x 0.126-inch diameter) |
| Floor Sheathing | 3/4-inch tongue and groove OSB |
| Roof Sheathing | 7/16-inch plywood |
| Interior Sheathing (walls) | 1/2-inch gypsum attached at 8 inches on center (1.1 inch long x 0.126-inch diameter screws), taped and mudded |
| Wall Anchorage/ Hold-downs | 1/2-inch anchor bolts at 4 feet on center, corner framing present |
| Exterior Finishes/Fenestration | None |

## Test Methods/Protocol

Four different levels of seismic loading were applied in each direction using a uni-axial shake table. The building was tested in each direction separately, using the same seismic loading. Only the final level of seismic loading was considered for this analysis and consisted of an artificially generated Shanghai earthquake record with a maximum peak ground acceleration of 0.55g. Additional weight was added to simulate 50% design roof and live load. Base shear was calculated from accelerometer readings at the ends of each wall and the seismic mass of the building. Testing resulted in approximate roof displacement of 2.95 inches in the direction parallel with the interior partition and 1.45 inches in the direction perpendicular to the interior partition wall. The specimen did not fail in either direction and only a few instances of cracking of the gypsum sheathing at the corners of wall openings and nails withdrawing slightly from the framing were observed.

## Analysis

Only the results from testing in the X-direction (see Figure 5) are analyzed in this report as it produced the greatest deformation and was closest to achieving ultimate strength of the structure. Tables 30 through 32 provide summaries of the analysis of the predicted shear strength for the building in the X-direction, using the perforated shear wall methods. The first floor shear walls were analyzed for the measured peak total base shear. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid diaphragm, with all walls in the direction of loading reaching their peak shear capacity at the same time.
2. All walls were designed as perforated shear walls fully restrained at the ends even though hold-down brackets were not installed. It was assumed that dead load and corner framing were sufficient to resist uplift.
3. Spruce-Pine-Fir No. 2 lumber with a SG = 0.42 was used in construction. All nominal unit shear capacities were adjusted by 0.92 in accordance with note 3 in table 4.3A of *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
4. Per Table 4.3A in the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*, the nominal unit shear capacity for 3/8-inch OSB sheathing attached to SPF lumber with 8d galvanized box nails at 6 inches on center on the panel perimeter is 671 plf. Although the report does not specify whether the nails were galvanized, this value is used to avoid overestimating the system factor. In addition, although the nails were not galvanized, they had a deformed shank.
5. The nominal unit shear capacity of 1/2-inch interior gypsum sheathing attached at 4 inches on center with drywall screws and assumed to be installed vertically to qualify as blocked is 320 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
6. Per Section 2305.3.9 of the 2006 IBC, for wind design, the shear capacity of a wall segment sheathed with wood structural panels and gypsum sheathing on opposite faces is the sum of the unit shear capacities of each face. For seismic design, only the unit shear capacity of the wood structural panel is counted towards the shear capacity of the wall.

**Table 30 - Predicted Shear Strength of Forintek Test House Using Method 1 (Sugiyama's PSW) (X-Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Openings (ft) | Total Length of Full Height Sheathed Segments (ft) | $C_{op}$[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1 | OSB (6/12 nailing) & Gypsum | 20 | 8 | 12 | 8 | 0.20 | 991 | 4,018 |
| 2 | Gypsum (both sides) | 20 | | 6 | 14 | 0.48 | 640 | 6,100 |
| 3 | OSB (6/12 nailing) & Gypsum | 20 | | 12 | 8 | 0.20 | 991 | 4,018 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **14,136** |
| **Maximum Base Shear** | **24,100** |
| **System Factor** | **1.70** |

[1]Adjustment factor calculated using Sugiyama equation, $C_{op} = r / (3-2r)$

**Table 31 - Predicted Shear Strength of Forintek Test House Using Method 2 (2006 IBC PSW) w/o seismic limitations (X-Direction)**

| Wall Label | Sheathing | Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | PSW Adjustment Factor, $C_o$[2] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 1 | OSB (6/12 nailing) & Gypsum | 20 | 8 | 8 | 0.41 | 991 | 3,254 |
| 2 | Gypsum (both sides) | 20 | | 14 | 0.67 | 640 | 6,046 |
| 3 | OSB (6/12 nailing) & Gypsum | 20 | | 8 | 0.41 | 991 | 3,254 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **12,554** |
| **Maximum Base Shear** | **24,100** |
| **System Factor** | **1.92** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio included
[2]Value interpolated from Table 2305.3.8.2 in 2006 IBC

**Table 32 - Predicted Shear Strength of Forintek Test House Using Method 2 (2006 IBC PSW) w/ seismic design requirements (X-Direction)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | Aspect Ratio Adjustment Factor[2] | PSW Adjustment Factor, $C_o$[3] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1 | OSB (6/12 nailing) & Gypsum | 20 | 8 | 8 | 1.0 | 0.41 | 671 | 2,204 |
| 2 | Gypsum (both sides) | 20 | | 14 | 1.0 | 0.67 | 640 | 6,046 |
| 3 | OSB (6/12 nailing) & Gypsum | 20 | | 8 | 1.0 | 0.41 | 671 | 2,204 |
| | | | | | **Total Calculated Shear Strength** | | | **10,454** |
| | | | | | **Total Tested Shear Strength** | | | **24,100** |
| | | | | | **System Factor** | | | **2.31** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio included

[2]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 must be reduced by 2w/h

[3]Value interpolated from Table 2305.3.8.2 in 2006 IBC

## Results

Table 33 provides a summary of comparisons between the predicted ultimate shear strength of the Forintek project test house and the ultimate tested shear strength of the house, as well as the corresponding system effect factor.

**Table 33 - Results of Strength Comparison and Approximate System Effect Factor for Forintek Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Shear Strength of House[1] (lb) | Whole House System Effect Factor |
|---|---|---|---|
| **Loading in X-Direction** | | | |
| **PSW Method w/ Sugiyama Equation** | 14,136 | 24,100 | 1.70 |
| **PSW Method w/ 2006 IBC w/o seismic limitations** | 12,554 | | 1.92 |
| **PSW Method w/ 2006 IBC Seismic Design Provisions** | 10,454 | | 2.31 |

[1]Test specimen was not taken to failure

The two-story Forintek project exhibited significantly stronger shear strength in the X-direction than predicted by the various PSW methods, with the system factor ranging between 1.7 and 2.3. It should be noted that the structure did not have hold-down brackets as required for perforated shear walls. Overturning restraint was provided by the corners and dead load. Furthermore, because the structure did not reach failure, and structural and cosmetic damage were both limited in extent, it likely had additional reserve strength above the measured capacity.

## Report 6: Full-Scale Shaking Table Tests of 3-Story Wood-Frame Construction Building (Japan 2x4 Home Builders Association, et. al.)

### General Construction

This testing sponsored by the Japan 2x4 Home Builders Association, along with several other industry and academic representatives, investigated the response of a conventionally-framed three-story home to seismic ground motion. The structure was tested with and without finish materials. The building had a 24-foot by 24-foot square footprint. Figure 6 shows a schematic of the building's wall layout and Table 34 provides a summary of the materials and construction methods used in the test house. The walls of the building were continuously sheathed with plywood, including areas above and below all openings.

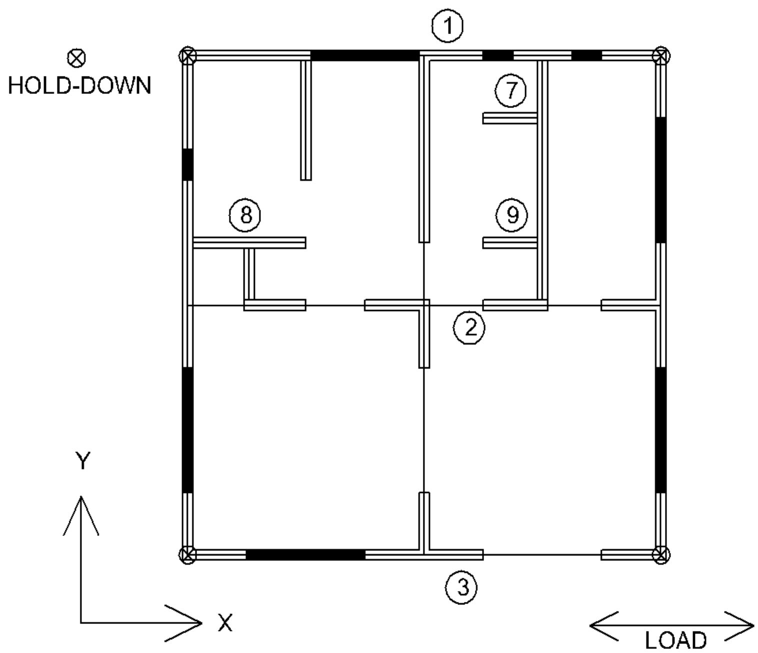


**Figure 6 - Wall layout of test house**

**Table 34 - Building Materials and Construction Methods of Japanese 2x4 Assoc. House**

| Component | Materials and Construction |
|---|---|
| Wall Framing | Doug Fir 2x4 nominal studs at 16 inches on center |
| Floor Framing | Wood I-joists at 16 inches on center |
| Roof Framing | Wood rafters and joists at 16 inches on center |
| Exterior Wall Sheathing/Bracing | 3/8-inch plywood nailed at 4 inches around the perimeter and 12 inches in the field (6d common nails, 2-inches long x 0.113-inch diameter) |
| Floor Sheathing | 3/4-inch tongue and groove OSB |
| Roof Sheathing | 7/16-inch plywood (assumed) |
| Interior Sheathing (walls) | 1/2-inch gypsum attached at 4 inches on center (1.25 inch long drywall screws), installed vertically, taped and mudded |
| Wall Anchorage/ Hold-downs | Hold-down anchors at each end of every shear wall (Perforated Shear Walls) |
| Exterior Finishes/Fenestration | Exterior ceramic siding and cement slate roofing |

## Test Methods/Protocol

Seismic loading was applied using a tri-axial shake table. Only the results of the third seismic level of loading were presented in the test report and thus considered for this analysis. It consisted of the ground motion of the Kawaguchi aftershock measured during the 2004 Mid Niigata Prefecture Earthquake. Maximum base shears are reported. Testing during the third stage was taken to an approximate roof displacement of 4.2 inches in the X-direction.

## Analysis

Tables 35 through 37 provide summaries of the analysis of the predicted shear strength for the building in the X-direction. Only the loads at the first floor shear walls were analyzed. The following design assumptions were used in the analysis:

1. The house was analyzed assuming a rigid diaphragm, with all walls reaching their peak shear capacity at the same time.
2. The measured peak total base shear was used for analysis.
3. All walls, including interior partitions, were designed as Perforated Shear Walls fully restrained at the ends. Hold-down anchors were installed at each end of every shear wall. It was assumed that for interior partition walls where no hold-downs were present, the dead load stabilizing moment and corner framing were sufficient to resist uplift.
4. Douglas Fir-Larch lumber with a SG = 0.50 was used in construction.
5. The nominal unit shear capacity of 3/8-inch exterior plywood sheathing nailed at 4 inches around the perimeter and 12 inches in the field with 6d common nails is 840 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.

6. The nominal unit shear capacity of 3/8-inch exterior plywood sheathing nailed at 3 inches around the perimeter and 12 inches in the field with 6d common nails is 1,090 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
7. The nominal unit shear capacity of 1/2-inch interior gypsum sheathing attached at 4 inches on center with drywall screws and assumed to be installed vertically to qualify as blocked is 320 plf, per the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*.
8. Per Section 2305.3.9 of the 2006 IBC, for wind design, the shear capacity of a wall segment sheathed with wood structural panels and gypsum sheathing on opposite faces is the sum of the unit shear capacities of each face. For seismic design, only the unit shear capacity of the wood structural panel is counted towards the shear capacity of the wall.

**Table 35 - Predicted shear strength of Japanese 2x4 Assoc. test house (X-Direction) using Method 1 (Sugiyama's PSW)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Openings (ft) | Total Length of Full Height Sheathed Segments (ft) | $C_{op}$ [1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1 | OSB (4/12 nailing) & Gypsum | 24 | 8 | 9 | 15 | 0.53 | 1,160 | 14,653 |
| 2 | OSB (3/12 nailing) & Gypsum | 22 | | 9 | 13 | 0.35 | 1,410 | 11,011 |
| 3 | OSB (4/12 nailing) & Gypsum | 24 | | 12 | 12 | 0.33 | 1,160 | 9,091 |
| 7 | Gypsum (both sides) | 3 | | 0 | 3 | 1.00 | 640 | 1,920 |
| 8 | Gypsum (both sides) | 6 | | 0 | 6 | 1.00 | 640 | 3,840 |
| 9 | Gypsum (both sides) | 3 | | 0 | 3 | 1.00 | 640 | 1,920 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **42,434** |
| **Maximum Base Shear** | **72,200** |
| **System Factor** | **1.70** |

[1]Adjustment factor calculated using Sugiyama equation, $C_{op} = r / (3-2r)$

**Table 36 - Predicted Shear Strength of Japanese 2x4 Assoc. Test House (X-Direction) using Method 2 (2006 IBC PSW) w/o Seismic Design Limitations**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | PSW Adjustment Factor, $C_o$ [2] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|
| 1 | OSB (4/12 nailing) & Gypsum | 24 | 8 | 15.00 | 0.63 | 1,160 | 10,962 |
| 2 | OSB (3/12 nailing) & Gypsum | 22 | | 13.00 | 0.61 | 1,410 | 11,181 |
| 3 | OSB (4/12 nailing) & Gypsum | 24 | | 12.00 | 0.55 | 1,160 | 7,656 |
| 7 | Gypsum (both sides) | 3 | | 0 | 1.00 | 640 | 0 |
| 8 | Gypsum (both sides) | 6 | | 6.00 | 1.00 | 640 | 3,840 |
| 9 | Gypsum (both sides) | 3 | | 0 | 1.00 | 640 | 0 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **33,639** |
| **Maximum Base Shear** | **72,200** |
| **System Factor** | **2.15** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio when sheathed with OSB and < 2:1 when sheathed with gypsum are included

[2]Value interpolated from Table 2305.3.8.2 in 2006 IBC

**Table 37 - Predicted Shear Strength of Japanese 2x4 Assoc. Test House (X-Direction) using Method 2 (2006 IBC PSW) w/ Seismic Design Requirements**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Total Length of Qualifying PSW Segments[1] (ft) | Aspect Ratio Adjustment Factor[2] | PSW Adjustment Factor, $C_o$[3] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) |
|---|---|---|---|---|---|---|---|---|
| 1 | OSB (4/12 nailing) & Gypsum | 24 | 8 | 15.00 | 0.75 | 0.63 | 840 | 5,954 |
| 2 | OSB (3/12 nailing) & Gypsum | 22 | | 13.00 | 0.75 | 0.61 | 1,090 | 6,483 |
| 3 | OSB (4/12 nailing) & Gypsum | 24 | | 12.00 | 0.75 | 0.55 | 840 | 4,158 |
| 7 | Gypsum (both sides) | 3 | | 0.00 | 0.75 | 1.00 | 640 | 0 |
| 8 | Gypsum (both sides) | 6 | | 0.00 | 0.75 | 1.00 | 640 | 0 |
| 9 | Gypsum (both sides) | 3 | | 0.00 | 0.75 | 1.00 | 640 | 0 |

| | |
|---|---|
| **Total Calculated Shear Strength** | **16,594** |
| **Total Tested Shear Strength** | **72,200** |
| **System Factor** | **4.35** |

[1]Only full height sheathed segments meeting < 3.5:1 aspect ratio when sheathed with OSB and < 2:1 when sheathed with gypsum are included

[2]Per Section 2305.3.8.2.2 of 2006 IBC, capacity of segments w/ 2:1< aspect ratio < 3.5:1 must be reduced by 2w/h

[3]Value interpolated from Table 2305.3.8.2 in 2006 IBC

## Results

Table 38 provides a summary of comparisons between the predicted shear strength of the conventionally framed three-story test house and the ultimate tested shear strength in the X-direction, as well as the corresponding system effect factor.

**Table 38 - Comparison of Predicted to Tested Strengths and System Effect Factor in Japanese 2x4 Assoc. Project**

| Design Methodology | Predicted Ultimate Shear Strength of House (lb) | Measured Shear Strength of House[1] (lb) | Whole House System Effect Factor |
|---|---|---|---|
| **Loading in X-Direction** | | | |
| **PSW Method w/ Sugiyama Equation** | 42,434 | 72,200 | 1.70 |
| **PSW Method w/ 2006 IBC w/o seismic design limitations** | 33,639 | | 2.15 |
| **PSW Method w/ 2006 IBC Seismic Design Provisions** | 16,594 | | 4.35 |

[1]Test specimen was not taken to failure

The three-story Japanese 2x4 Association test house achieved greater tested strengths than the strength attributed to it through the various PSW design methods, with the system factor ranging between 1.7 and 4.3. The predicted strength reduced substantially when the more stringent requirements for seismic design were taken into account. This reduction in seismic design capacity was the result of the large number of full height sheathed wall segments with aspect ratios greater than 2:1.

# Report 7: Effect of Transverse Walls and Vertical Load on the Performance of Shear Walls (Forintek/Tongji)

## General Construction

The Forintek Canada Corp., in collaboration with researchers at Tongji University in China, investigated the effects of perpendicular walls and corner framing on the response of shear walls under cyclic loading. Single 20-foot long shear walls with 4-foot long transverse end walls and two sizes of door openings were tested. A total of five tests were performed on five different wall configurations: one without additional vertical dead load, two with an additional dead load of 240 plf along the top of the transverse end walls and two with an additional dead load of 240 plf along the top of the main shear wall. Figure 7 shows a schematic of the five different test configurations and Table 39 provides a summary of the materials and construction methods used. The walls were continuously sheathed with OSB including areas above and below all openings.

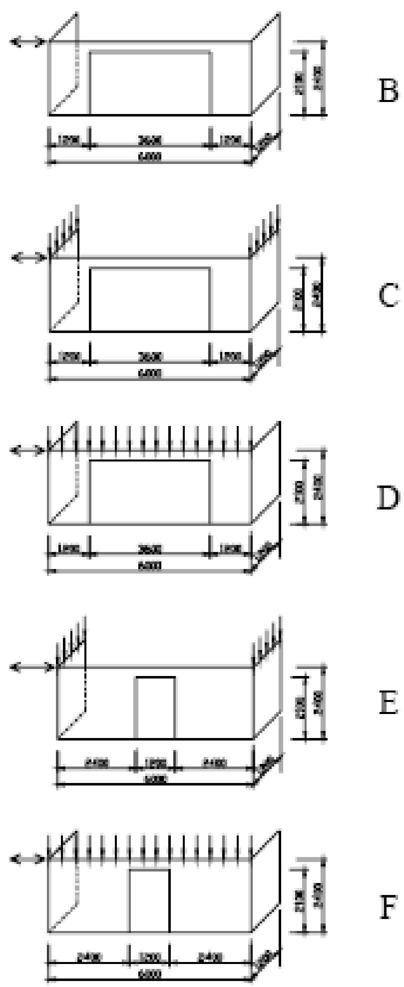


**Figure 7 - Configurations of shear walls tested during Forintek/Tongji project**

**Table 39 - Building Materials and Construction Methods for Forintek/Tongji Project**

| Component | Materials and Construction |
|---|---|
| Wall Framing | SPF No. 2 2x4 nominal studs at 16 inches on center |
| Exterior Wall Sheathing/Bracing | 3/8-inch OSB nailed at 6 inches around the perimeter and 12 inches in the field (twisted nails, 2.5-inches long x 0.129-inch diameter) |
| Interior Sheathing (walls) | None |
| Wall Anchorage/Hold-downs | 1/2-inch anchor bolts at 16 inches on center, two at the ends of each wall segment, corner framing installed, no hold-downs |
| Exterior Finishes/Fenestration | None |

## Test Methods/Protocol

Cyclic load was applied using a hydraulic jack and a spreader beam attached to the top plate of the wall at a rate of 0.8 inch/sec. The walls were tested to failure and the ultimate shear load was measured using a load cell placed between the jack and the spreader beam.

## Analysis

Because the Forintek/Tongji testing was done on isolated shear walls and corner framing, not an entire structure, it does not lend itself to an evaluation of a whole-house system effect factor. Instead, a partial restraint factor for Method 3 (PC2 to RB148) is evaluated. Because the purpose of this analysis is to arrive at a partial restraint factor, the wall strength used in Method 3 is not reduced to account for any partial restraint.

Tables 40 and 41 provide summaries of the analysis of the predicted shear strength for the walls using Method 1 (Sugiyama's PSW) and Method 3 (PC2 to RB148), as well as the corresponding partial restraint factors where appropriate. None of the more stringent seismic design requirements on aspect ratios were applicable for these wall configurations, i.e., all qualifying shear walls had aspect ratios of less than 2:1. The following design assumptions were used in the analysis:

1. Peak shear strength of the shear walls during testing was used for comparison.
2. All walls were designed as Perforated Shear Walls fully restrained at the ends even though hold-down brackets were not installed. It was assumed that dead load and corner framing were sufficient to resist uplift.
3. Walls E and F were loaded with an additional vertical uniform dead load of 240 plf and therefore were assumed to support one floor and the roof for the purposes of design using the RB148 method. All other tested walls had either no additional vertical load applied, or had vertical load applied to the transverse end walls only, but were still assumed to support only a roof to allow for design using the RB148 method.

4. Per Table 4.3A in the *AF&PA Special Design Provisions for Wind and Seismic – 2005 Edition*, the nominal unit shear capacity for 3/8-inch OSB sheathing attached to SPF lumber with 8d galvanized box nails at 6 inches on center on the panel perimeter is 671 plf. Although the report does not specify whether the nails were galvanized, this value is used to avoid overestimating the system factor. In addition, although the nails were not galvanized, they had a deformed shank.

**Table 40 - Predicted shear strengths of Forintek shear wall tests using Method 1 (Sugiyama's PSW)**

| Wall Label | Sheathing | Total Wall Length (ft) | Wall Height (ft) | Applied Gravity Load (plf) / Location | Total Length of Full Height Sheathed Segments (ft) | $C_{op}$[1] | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) | Measured Strength (lb) | Partial Restraint Factor[2] |
|---|---|---|---|---|---|---|---|---|---|---|
| B | OSB (6/12 nailing) | 20 | 8 | 0 | 8 | 0.20 | 671 | 2,749 | 4,105 | 1.0 |
| C | OSB (6/12 nailing) | 20 | | 240 / Wing Walls | 8 | 0.20 | 671 | 2,749 | 4,480 | 1.0 |
| D | OSB (6/12 nailing) | 20 | | 240 / Main Wall | 8 | 0.20 | 671 | 2,749 | 5,560 | 1.0 |
| E | OSB (6/12 nailing) | 20 | | 240 / Wing Walls | 16 | 0.61 | 671 | 8,149 | 8,520 | 1.0 |
| F | OSB (6/12 nailing) | 20 | | 240 / Main Wall | 16 | 0.61 | 671 | 8,149 | 10,690 | 1.0 |

[1]Adjustment factor calculated using Sugiyama equation, $C_{op} = r / (3-2r)$

[2]Cannot be greater than 1.0

**Table 41 - Predicted Shear Strengths of Forintek Shear Wall Tests Using Method 3 (PC1 RB148)**

| Wall Label | Sheathing | Applied Gravity Load (plf) / Location | Total Length of Qualifying PSW Segments (ft) | Nominal Unit Shear Capacity (plf) | $F_{wall}$ (lb) | Measured Strength (lb) | Partial Restraint Factor[1] |
|---|---|---|---|---|---|---|---|
| B | OSB (6/12 nailing) | 0 | 8 | 671 | 5,368 | 4,105 | 0.76 |
| C | OSB (6/12 nailing) | 240 / Wing Walls | 8 | 671 | 5,368 | 4,480 | 0.83 |
| D | OSB (6/12 nailing) | 240 / Main Wall | 8 | 671 | 5,368 | 5,560 | 1.0 |
| E | OSB (6/12 nailing) | 240 / Wing Walls | 16 | 671 | 10,736 | 8,520 | 0.79 |
| F | OSB (6/12 nailing) | 240 / Main Wall | 16 | 671 | 10,736 | 10,690 | 1.0 |

[1]Cannot be greater than 1.0

## Results

Table 42 provides a summary comparison between the predicted strength of the tested shear wall configurations using design Methods 1 and 3 and the ultimate tested shear strength of the walls, as well as the corresponding partial restraint factor.

**Table 42 - Comparison of Predicted to Tested Strengths in Forintek/Tongji Shear Wall Project**

| Design Methodology | Predicted Ultimate Shear Strength of Shear Wall (lb) | Ultimate Measured Strength of Shear Wall (lb) | Partial Restraint Factor |
|---|---|---|---|
| **Wall B** | | | |
| **PSW Method w/ Sugiyama Equation** | 2,749 | 4,105 | 1.0 |
| **RB148 Method** | 5,368 | | 0.76 |
| **Wall C** | | | |
| **PSW Method w/ Sugiyama Equation** | 2,749 | 4,480 | 1.0 |
| **RB148 Method** | 5,368 | | 0.83 |
| **Wall D** | | | |
| **PSW Method w/ Sugiyama Equation** | 2,749 | 5,560 | 1.0 |
| **RB148 Method** | 5,368 | | 1.0 |
| **Wall E** | | | |
| **PSW Method w/ Sugiyama Equation** | 8,149 | 8,520 | 1.0 |
| **RB148 Method** | 10,736 | | 0.79 |
| **Wall F** | | | |
| **PSW Method w/ Sugiyama Equation** | 8,149 | 10,690 | 1.0 |
| **RB148 Method** | 10,736 | | 1.0 |

The analysis indicates that the perforated shear wall design method consistently under-predicts the capacity of the walls restrained against uplift with corners and gravity load. Therefore, hold-down brackets may not be required for wall configurations that include corner framing. Because uplift forces increase with the wall's capacity (i.e., increased nailing schedule), additional testing would be beneficial to establish an upper bound shear strength limit on the use of corners in lieu of hold-downs in perforated shear walls.

The analysis of the RB148 methodology supports the selection of the partial restraint factors ranging between 0.8 for single story construction (minimum gravity load) and 1.0 for three-story construction (maximum gravity load). Although the partial restraint factor of 0.76 for Wall B was slightly below the 0.8 value used in RB148, the specimen did not have any dead load from the roof either along the wing walls or along the primary wall line as it would be present in a finished house.

## Summary and Conclusions

A total of six full-size whole house tests and one shear wall with end walls test were analyzed to compare the structure's ultimate tested shear strength with the shear strength predicted by applicable engineering design methods. The design methods were selected based on the structure's detailing and the testing protocol. One or more of following methods was used for each structure, as applicable:

| | |
|---|---|
| Method 1: | Perforated Shear Wall Method (Sugiyama) without building code limitations |
| Method 2: | Perforated Shear Wall Method (2006 International Building Code) |
| Method 3: | Wind Bracing Design Method used for Public Comment 1 to RB148 (ICC 2007/2008 Code Development Cycle, 2008 Final Action Agenda, International Residential Code) |
| Method 4: | Segmented Shear Wall Method (2006 International Building Code) with or without code limitations |

Structures were analyzed both with and without the penalty of the applicable building code limitations on the aspect ratios and combination of materials to investigate a range of scenarios for system effects. Table 43 provides a matrix of all analyses conducted and the corresponding system factors.

The following observations were made based the results of the analysis:

1. Analysis of the results for the whole-house tests supports this study's hypothesis of significant system effects in residential light-frame wood structures.
2. The magnitude of the system factors depends on the engineering method, applicable code limitations, and the presence of finish materials.
3. The current limitations on aspect ratios and combination of materials may be overly restrictive for typical house configurations.
4. For buildings that had finishes installed (at least on the interior), the system effect ranges between 1.5 and 4.3 depending on the engineering model selected. Overall, structures with finishes had a greater system effect factor than structures without finishes.
5. For buildings that had finishes installed (at least on the interior), the system factor for design methods that included all applicable building code limitations ranged between 1.8 and 4.3.
6. For two structures without any finishes (wood framing and structural sheathing only), the system factor for design methods that included all applicable building code limitations ranged between 1.39 and 1.67. Without the code limitations, the system factor for the NEESWood house had the lowest value of 1.08. However, the structure was not tested to its capacity and was likely to have a reserved strength. After finishes were installed and a higher level ground motion was applied, the system factor increased to 1.80.

7. The RB148 method, only used for evaluation of non-dynamic tests, resulted in a system factor ranging between 1.5 and 2.9, supporting the value used to develop the 2009 IRC wind wall bracing provisions.
8. The use of corners as anchorage for the perforated shear wall method in lieu of hold-down brackets did not appear to have an adverse effect on the overall ultimate shear strength of the house. This conclusion applies to all the reviewed tests including those that applied tri-axial loading.
9. There was no apparent direct correlation between the number of stories in the test specimens and the calculated system effect factor.

**Table 43 - Summary of Results: System Effect Factor and Partial Restraint Factor**

| # | Title | Test Protocol | Finishes | PSW (Sugiyama) w/o any code limitations) | PSW w/o seismic limitations | PSW w/ seismic limitations | Segmented w/o any code limitations | Segmented w/o seismic limitations | Segmented w/ seismic limitations | PC2 RB148 |
|---|---|---|---|---|---|---|---|---|---|---|
| **System Factor** | | | | | | | | | | |
| 1 | Whole Structure Testing and Analysis of a Light-Frame Wood Building (CSIRO) | Cyclic (static) | Interior gypsum | 1.54 | 1.91 | 2.19 | n/a | n/a | n/a | 1.47 - 1.67 |
| 2 | Shake Table Tests of a Two-Story Wood-frame House (CUREE Wood-frame Project) | Shake table (dynamic) | Bare Frame | 1.67 | 1.67 | 1.67 | n/a | n/a | n/a | n/a |
| | | | Fully Finished w Stucco | n/a | n/a | n/a | 1.72 | 1.80 | 2.25 | n/a |
| 3 | Full-Sized House Cyclic Racking Test (BRANZ) | Cyclic (static) | Fully Finished | n/a | n/a | n/a | 1.76 | 1.76 | 2.31 | 2.89 |
| 4 | Seismic Testing of Full-Scale Two-Story Light-Frame Wood Building (NEESWood) | Shake table (dynamic) | Bare Frame | n/a | n/a | n/a | 1.08 | 1.31 | 1.38 | n/a |
| | | | Fully Finished w Stucco | n/a | n/a | n/a | 1.80 | 2.18 | 2.55 | n/a |
| 5 | Assessment of Seismic Resistance of Conventional Wood-Frame Houses (Forintek Collaboration) | Shake table (dynamic) | Interior Gypsum | 1.70 | 1.92 | 2.31 | n/a | n/a | n/a | n/a |
| 6 | Full-Scale Shaking Table Tests of 3-Story Wood-Frame Construction Buildings (Japan 2x4 Home Builders Association, et. al) | Shake table (dynamic) | Interior and Exterior Finishes | 1.70 | 2.15 | 4.35 | n/a | n/a | n/a | n/a |
| **Partial Restraint Factor** | | | | | | | | | | |
| 7 | Effect of Transverse Walls and Vertical Load on the Performance of Shear Walls (Forintek/Tongji) | Cyclic (static) | None | 1.0 | n/a | n/a | n/a | n/a | n/a | 0.76 - 1.0 |

# APPENDIX A – Summary of Full-Scale Tests

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Full Scale Test on a Two-Story House Subjected to Lateral Load | Yokel, Somes | 1973 | USA | Monotonic w/ hydraulic jacks at (4) locations and (2) levels (levels tested separately) | 2 story house, 26 'x 47' footprint; 2x4 @ 16" oc, exterior walls and interior partitions | Single 1x4 let in brace @ 45 degrees at each corner of house; 1/2" gyp board installed per ASTM C79 | 3/8" gypsum wall board installed per ASTM C36 | Asbestos and beveled wood siding | Not to failure; drift of lower level LFT wall @ 25 psf (≈7050 lb) = 0.025"; Drift of lower level RT wall @ 25 psf (≈7050 lb) = 0.04"; Diagonal compression of brace @ 5k load = 0.002"; Total 2nd floor level displacement @ 5k load = 0.0175" |
| 2 | Testing of a Full-Scale House under Simulated Snow loads and Wind loads | Tuomi, McCutcheon | 1974 | USA | Cyclic at top plates | 1 story house, 16' x 24' footprint; 8' tall, Doug Fir 2x4 studs @ 16" oc | 3/8" plywood (vertical) nailed w/ 6d nails @ 6" and 12" oc | 1/2" gypsum wall board nailed w/ 1.25" nails @ 8" oc | 1/2" x 6" Western Red cedar siding w/ 1" lap and 7d nails | Plywood only - 1480 lbs @ 0.1"; Drywall and siding - 1770 lb @ 0.1"; Side walls added - 2220 lb @ 0.1"; Roof trusses added - 2760 lb @ 0.1" |
| 3 | Simulated Wind Tests on a House | Boughton, Reardon | 1982 | Australia | Monotonic w/ hydraulic jacks at (4) locations and (3) levels (levels tested separately); uplift loading was also conducted | Built to 1940 U.S. Air Force Specs, not typical construction; 3"x3" studs supporting roof trusses, spaced @ 10' oc. Windows between studs | Diagonal let-in bracing nailed to studs | Plywood sheathing primarily | 1.25" x 10" horizontal boards nailed directly to studs | Unknown |
| 4 | Structural Development and Evaluation of a Modular Home | Hurst | 1983 | USA | Monotonic w/ air bag and simultaneous uplift on roof (29% less than wall pressure) | 1 story house, 12' x 44' footprint; 1x4 S. Pine #2 studs @ 16 oc | 3/8" plywood, glued/stapled | 1/2" gypsum wall board glue-nailed | None | Not loaded to failure; @ 26 psf load, exterior walls saw 0.34" deflection (≈1250 lbs) and 0.63" deflection (≈1840 lbs) |
| 5 | Simulated Cyclone Wind Tests on a Timber Frame House | Boughton, Reardon | 1984 | Australia | Cyclic w/ hydraulic jacks, uplift on roof and lateral at top plate; 3 stages: 8000 cycles at 5/8 design load, 2000 cycles at 3/4 design load and 200 cycles at design load, lateral loads only applied on every 10th cycle; Total load applied to house at end was 204 kN (4.75x design load) | Built to 1983 North Queensland Home Building Code, 1 story house on columns, 7m x 13m footprint; 70mm x 45mm exterior wall framing, 70mm x 35mm interior wall framing, roof trusses w/ metal roof sheathing | Unknown | Gypsum plaster board | Fiber cement sheathing | Racking tests all done to 204 kN total load: 1) Roof sheathing removed - no failure; 2) Roof sheathing replaced, plaster cornice removed - no failure, but higher displacements; 3) 50% of external sheathing damaged by debris - failure |
| 6 | Performance of a Steel Framed Kit House Tested for 63 m/s Cyclonic Winds | Reardon, Boughton | 1985 | Australia | Cyclic w/ hydraulic jacks, uplift on roof and lateral at top plate; 3 stages: 8000 cycles at 5/8 design load, 2000 cycles at 3/4 design load and 200 cycles at design load, lateral loads only applied on every 10th cycle; Total load applied to house at end was | Built to Logan manufacturer specifications, 1 story house, 14m x 7m footprint; 70mm x 1.15 mm thick steel studs, panelized construction | Fiber cement board, glued | Fiber cement board, glued | Unknown | Ultimate racking load not determined, failed in uplift: final load of 6.92 kPa laterally w/ "very high" stiffness |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 6.92 kPa (2.2x design load) | | | | | |
| 7 | The Structural Response of a Brick Veneer House to Simulated Cyclone Winds | Reardon | 1987 | Australia | Cyclic w/ hydraulic jacks, uplift on roof and lateral at top plate; 3 stages w/ 8000 cycles at 5/8 design load, 2000 cycles at 3/4 design load and 200 cycles at design load, lateral loads only applied on every 10th cycle; Total load applied to house at end was 60 kN (3x design load) Note: roof straps broke early, so majority of testing was done w/ only monotonic racking | Built to 1984 Queensland Home Building Code, 1 story house, 7170mm x 16415mm footprint; Roof trusses attached with light gage metal straps | 900 mm of plywood bracing | Diagonal metal strap bracing; Plasterboard glued and tapped | None | Racking tests during construction: 1) Bracing and lining, no roof or ceiling linings - 2.25 kN/mm stiffness @ interior wall; 2) Roof cladding added - 2.75 kN/mm stiffness @ interior wall; 3) Ceiling lining added - 6.9 kN/mm stiffness @ interior wall; Racking tests during de-construction: 1) Complete house - 12 kN/mm stiffness; 2) X-bracing removed - 18.75 kN/mm stiffness; 3) Lining removed from interior wall line - 7.6 kN/mm stiffness; 4) Test w/ uplift forces - 6 kN/mm stiffness; 5) Lining removed from passage walls - 8.3 kN/mm stiffness; 6) Test w/ uplift forces - 5 kN/mm stiffness |
| 8 | Effects of Claddings on the Response of Houses to Wind Forces | Reardon | 1989 | Australia | Monotonic w/ horizontal load beam at top plate | Two homes tested. One (House W33) built to 1979 SSA Timber Framing Code (design wind speed of 33m/s), 1 story house, 7170mm x 16415mm footprint; conventional rafter system w/ concrete roof tiles; Other (House W24) built to 1984 Queensland Home Building Code (design wind speed of 42m/s), 1 story house, 7170mm x 16415mm footprint; Roof trusses attached with light gage metal straps | Unknown | Diagonal timber members | Unknown | Lateral displacement for 12 kN racking force (House W33): 1) Lined walls - 3.2mm; 2) Roofing added - 1.2mm; 3) Ceiling added - 0.8mm; 4) Cornice added - 0.5mm; |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 9 | Effect of Linings on the Response of Houses to Wind Forces | Reardon, Henderson | 1990 | Australia | Cyclic w/ hydraulic jacks, uplift on roof and lateral at top plate; 3 stages; 8000 cycles at 5/8 design load, 2000 cycles at 3/4 design load and 200 cycles at design load, lateral loads only applied on every 10th cycle; Total load applied to house at end was 70 kN (3x design load) | Built to Nu-Steel Manufacture's Specifications, 7.5m x 15.6m footprint; 75mm x 32mm x 1.2mm steel studs, stiffened 75mm x 79mm x 1.6mm top plate, built up roof trusses using 101mm x 47mm x 1.0mm Z sections | Diagonal metal strap bracing | Diagonal metal strap bracing; Plasterboard glued and tapped | Cement weatherboard | 1) No linings, bracing in exterior walls only - 3.5 kN/mm stiffness; 2) No linings, bracing in both interior & exterior walls - 5.1 kN/mm stiffness; 3) Linings installed, not tapped or mudded - 16.5 kN/mm stiffness; 4) Linings tapped and mudded - 25.6 kN/mm stiffness at walls, 8.3 kN/mm stiffness at window header; 5) Cornice added - 29.3 kN/mm stiffness at walls, 25.0 kN/mm stiffness at window head |
| 10 | Load Characteristics of Three-Dimensional Wood Diaphragms | Phillips | 1990 | USA | Quasi-Cyclic loading to each wall, both individually and at the same time (phase 4) | 1 story house, 32' x 16' footprint; 2x4 framing, 8' high walls, Wall 1 end wall had window, interior walls had doors, 2 internal shear walls, roof consisting of (7) - wood trusses, plywood on top, 1/2" gypsum board on ceiling | None | 1/2" gypsum wall board; Wall 2 int. wall had 1/2" plywood on both sides, other (Wall 3) had 1/2" gyp on both sides | T1-11 plywood siding, looks to be installed vertically | Wall 1 ≈ 8400 lb load; Wall 2 ≈ 3850 lb load; Wall 3 ≈ 6500 lb load; Wall 4 ≈ 7000 lb load |
| 11 | Simulated Wind Loading of a Two Storey Test House | Reardon, Henderson | 1996 | Australia | Monotonic w/ hydraulic jacks uniformly at top of wall; uplift loading was also conducted | 2 story house, 15m x 10m footprint; split level both 1 and 2 story sections, 1 story section is 9m x 8m, 2 story section is 6m x 10m; 90mm x 35mm F5 studs, 2 exterior walls, 1 "demising" wall, 1 interior partition | Plywood (1 story section) and diagonal bracing (2 story section) (exact construction not shown) | Unknown | Unknown | Deflections due to uniform lateral load of 1 kN/m: 1) 10m long 2 story wall - a) w/o roof tiles = 3.3mm, b) w/ roof tiles = 1.8mm; 2) 8m long 1 story wall - a) w/o roof tiles =1.6 mm, b) w/ roof tiles = 1.3mm; deflections due to combined loading @ 28m/s wind load (15.4 kN lat on 2 story section, 16.3 kN lat on 1 story section): 3) 10m long 2 story wall = 2.2mm; 4) 8m long 1 story wall = 1.6mm |
| 12 | Lateral Performance of Cold-Formed Steel-Framed Domestic Structures | E.F. Gad, C.F. Duffield, G.L. Hutchinson, D.S. Mansell, G. Stark | 1997 | Australia | Dynamic w/ shake table, (1) swept sine wave test with constant displacement amplitude and (1) racking testing with increasing amplitude till failure; simulated dead load of 2300kg was added using concrete slab supported on E-W walls | 1 story house, 2.3m x 2.4m x 2.4m high; 75mm x 35mm x 1mm studs @ 600mm oc, N-S walls had 900mm x 2100m openings, over designed hold-downs to prohibit uplift/bottom plate bending failure, brick veneer corners not connected together | Diagonal 25mm wide x 1mm thick metal strap bracing (E-W walls) | 10mm thick plasterboard on ceilings and walls, screwed @ 300mm and 400mm oc (ceiling) and 200mm and 600mm oc (walls); skirting boards, 55mm cornices and set corner joints were used too | Brick veneer tied back to studs at 3rd course from bottom, and every 5th after that | 1) Un-braced, un-lined frame ≈ 0.35 kN load (0.025 kN/mm stiffness); 2) Un-lined, braced walls ≈ 9 kN load; 3) Un-braced but lined walls & ceiling (gyp board only) ≈ 22 kN load; 4) Braced and lined walls & ceiling ≈ 26 kN load (failure of straps occurred similar to when no lining was in place) |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 13 | Whole Structure Testing and Analysis of a Light-framed Wood Building - Phase 3 Destructive Testing | Paevere, Foliente, Macindoe, Banks, Kasal, Collins | 2000 | Australia | Static-Cyclic loading at top plate level, (2) out of (4) shear walls loaded | 1 story house, L shaped, 9.5m x 11.1m footprint; 90mm x 35mm SYP studs @ 400mm oc, 3 out of 4 shear walls had large openings, roof trusses @ 600mm oc, no blocking, sheathed | 9.5mm plywood nailed w/ 50mm x 2.87mm nails @ 150mm and 300mm oc | 13mm gypsum wall board screwed w/ self drilling screws @ 300mm oc | None | Wall 3 - 33 kN; Wall 4 - 50 kN; Wall 1 -12 kN; Wall 2 - 30 kN |
| 14 | Shake Table Tests of a Two-Story Wood-Frame House | Fischer, Filiatrault, Folz, Uang, Seible | 2001 | USA | Dynamic tri-axial w/ shake table, ordinary ground motions of Design Basis Earthquake (DBE) (Northridge 1994) and near-field ground motions of Max Credible Earthquake (MCE) (Northridge 1994); weights added to maintain constant seismic weight during different phases of testing | Built to 1994 UBC, 2 story house 20' x 16' footprint; 2x4 framing @ 16" oc; Note - various engineered straps and hold downs present for testing of interior finishes | 3/8" OSB nailed w/ 8d box nails @ 6" and 6" oc. | 1/2" gypsum wall board, horizontal, screwed w/ #6 screws @ 16" oc (walls) and @ 12" oc (ceilings), taped and mudded | 7/8" plaster stucco | Phase 9 (interior finishes and stucco) ≈29k @ 4.25" displacement |
| 15 | Full-Scale Shaking Table Studies of Wood-Frame Residential Construction (Earthquake 99 Wood-frame House Project) | Ventura, Taylor, Prion, Kaharrazi, Pryor | 2001 | USA | Dynamic uni-axial w/ shake table (Northridge 1994) | 2 story house, 20' x 25' footprint | OSB panel wall system; Horizontal board sheathing (test 13) | Gypsum wall board | Plaster stucco | 1) Horizontal board sheathing, no hold-downs, stucco or blocking (Test 13) - 80 kN @ 125mm; 2) Hold-downs, no stucco (Test 14) - 70 kN @ 45mm; 3) Hold-downs and stucco (Test 10) - 55 kN @ 10mm |
| 16 | Seismic Evaluation of an Asymmetric Three-Story Wood-Frame Building | Mosalam, Machado, Gliniorz, Naito, Kunkel, Mahin | 2002 | USA | Dynamic tri-axial w/ shake table, ordinary ground motions of Design Basis Earthquake (DBE) (Northridge 1994) and near-field ground motions of Max Credible Earthquake (MCE) (Northridge 1994, max accel of 0.5g); weight of building = 70.93k | Built to 1964 UBC w/ 1997 UBC checks, 3 story house w/ car ports underneath, 32' x 16' footprint; 2x4 framing @ 16" oc; | 3/8" plywood nailed w/ 8d nails @ 6", 5" and 4" in various places | 5/8" gypsum wall board nailed w/ 6d cooler nails @ 6" oc | 7/8" plaster stucco over 18 gauge metal lath | Phase I (no finishes): 1) East wall: story 3 - 9.67k, story 2 - 20.31k, story 1 - 25.86k; 2) West wall: story 3 - 17.39k, story 2 - 19.06k, story 1 - 22.2k: Phase III (w/ finishes): 3) East wall: story 3 - 11.76k, story 2 - 27.25k, story 1 - 43.61k; 4) West wall: story 3 - 18.01k, story 2 - 30.67k, story 1 - 39.69k; |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 17 | Full-Sized House Cyclic Racking Test - No. 119 | Thurston | 2003 | New Zealand | Cyclic w/ hydraulic jacks at (4) locations, timber loading beams took load directly into walls via plywood above ceiling | 1 story house, 19.7' x 41' footprint, 2.4m high; 90mm x 45mm exterior studs, 65mm x 45mm interior studs, all at 600mm oc, plates nailed to floor with 1-nail @ 600mm oc | None | Wall 1 (w/ windows) - 10mm gypsum board screwed @ 300mm oc (perimeter only), section between windows - bracing grade gypsum wall board (1630mm long) nailed @ 150mm & 300mm; Wall 2 - 10mm gypsum wall board nailed @ 300mm oc (side 1) and bracing grade gypsum wall board nailed @ 150mm & 300mm (side 2); Wall 3 - 10mm gypsum wall board nailed @ 300mm oc; Wall 4 - 10mm gypsum wall board nailed @ 300mm oc (side 1) and screwed at 150mm & 300mm (side 2); Wall 5 - 10mm gypsum wall board screwed @ 300mm oc (perimeter only); 10mm gypsum wall board screwed at 150mm & 300mm oc. | Fiber cement planks 300mm x 6mm | Peak resistance of house ≈ 122 kN total |
| 18 | Lateral Load Testing and Analysis of Manufactured Homes | Koerner, Schmidt, Goodman, Richins | 2004 | USA | Monotonic lateral w/ airbag | | Gypsum interior, siding exterior | Gypsum wall board both sides | Exterior siding | Only to 30 psf --> gives strains at that loading |
| 19 | Experimental and Analytical Deformations of the Wood-Framed Building under Lateral Load | Malesza, Miedzialowski | 2004 | Poland | Monotonic w/ loading beam in tension (assumed to be attached to floor diaphragm) | 2.74m tall, Polish Fir 45mm x 135mm stud @ 600mm | 12mm thick chip board inside nailed w/ N2.6/60mm nails @ 100mm oc (perimeter only) | None | 12.5mm special gypsum outside, staples K1.6/50mm @ 80mm oc around perimeter of Fermacel board. | ≈ 12 kN @ 0.18mm |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 20 | A preliminary Report on the Effect of Various Boundary Conditions on the Performance of the 2006 IRC Method 3 Braced Wall Panel | Dolan and Pryor | 2005 | USA | Monotonic w/ steel rod in tension (attached to roof/floor diaphragm, pick point in center) | Built to IRC 2006, Method 3 Wall Bracing; 1 story box, 20' x 20' footprint; 2x4 @ 16 oc, 2x10 roof/floor joist w/ 7/16" OSB nailed w/ 8d common nails @ 6" and 12" | (1) single 4x8 OSB sheathing, assuming nailed at 6" and 12", not other exterior brace along wall | None | None | 1) Load Parallel to roof/floor joists - 4100 lb; 2) Load Perpendicular to roof/floor joists - 3175 lb |
| 21 | APA Full-Scale House Test Project - 3-D testing with 4:1 aspect ratio wall bracing | | 2006 | USA | Monotonic in tension at top plates | 1 story structure, 25' x 37.5' footprint, 8' tall; SPF 2x4 framing @ 24"oc | 7/16" OSB nailed w/ 8d cooler nails @ 6" and 12" oc | None | None | 1) (3) 48" panels, corner returns - 5000 lb; 2) (3) 48" panels, no returns - 4500 lb; 3) (6) 24" panels & openings, continuous band of sheathing above and below openings, no return - 8000 lb; 4) (6) 24" panels & openings, hold downs @ corners of house - 8700 lb |
| 22 | Assessment of Seismic Resistance of Conventional Wood-frame Houses | NI, Rainer, Chen, Tongji, Karacebeyli, Follesa | 2006 | China | Dynamic uni-axial w/ shake table (up to 0.50g); 50% of live load added @ roof and floor level | Built to 1995 National Building Code of Canada, 2 story house, 6m x 6m footprint, 8' tall; SPF 38mm x 89mm framing @ 16" oc, 1 interior partition wall | 3/8" OSB nailed w/ 3.2mm x 65mm twisted nails @ 150mm and 300mm oc | 1/2" gypsum wall board screwed w/ 3.2mm x 28mm screws @ 200mm oc | Unknown | 1) 1.2m door in direction of load - 68.76 kN; 2) 2.4m door in direction of load - 78.57 kN; 3) 3.6m door direction of load - 107.2 kN; 4) (2) 1.2m windows in direction of load - 101.3 kN |
| 23 | Dynamic Tests of Traditional Wooden House in Kyoto Using Large-scale Shaking Table | Suda, Susuki, Shimizu, Ogasawara | 2006 | Japan | Dynamic uni-axial w/ shake table (ground motion records from Hyogoken-Nanbu earthquake (1995), max accel of 400 $cm/s^2$) | Kyo-machiya type, 2 story, 6m x 11.8m footprint, 8.5m tall; 110mm x 110mm column frames | Mud walls (bamboo lath & mud); then reinforced with ladder frames and additional wall inserts where none existed | Mud walls (bamboo lath & mud) | Unknown | 1) Un-reinforced house = 1/34 rad @ 200 $cm/s^2$; 2) Reinforced house = 1/47 rad @ 200 $cm/s^2$ |
| 24 | A study of Collapsing Process of Wood Conventional Houses - Shaking Table Tests of Real-size Models | Mikio, Yasuhiro, Hiroshi, Isao | 2006 | Japan | Dynamic uni-axial and tri-axial w/ shake table (ground motion records from JR-TAKATORI wave (1995)) | Built to Japanese 1960 code, 2 story, 3.64m x 5.45m footprint; 120mm x 120mm and 105mm x 105mm column frames, no metal connectors at top and bottom of columns | Wooden strap bracing, 27mm x 105mm | Gypsum sheathing attached to columns | Unknown | House with gypsum sheathing was 155% stronger then house w/o gypsum sheathing |
| 25 | A Collapsing Response Analysis of Existing Wood House Subjected to Seismic Motion | Tatsuya, Chikahiro, et al | 2006 | Japan | Dynamic uni-axial w/ shake table (ground motion records from JR-TAKATORI wave, 1995) | Built to Japanese 1979 code, 105mm x 105mm columns, (2) N90 nails at bottom of columns, tendon joints at tops, only "L" corner section of 1st fl tested, two other walls made of plywood, lumber and steel bracing to isolate failure | Wooden strap bracing, 25mm x 105mm | Gypsum Sheathing (12.5mm) or Plywood (5.5mm)? | Mortaring w/ wood lath | Unknown |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 26 | Estimating Seismic Performance of Timber Structures with Plywood-sheathed Walls by Pseudo-dynamic Tests and Time-history Earthquake Response Analysis | Yasumura, Richard, Davenne, Uesugi | 2006 | Japan | Dynamic uni-axial w/ shake table (ground motion records from El Centro (1940) max 0.4g); loaded only at one corner, eccentric | Typical Japanese construction one room box, 3m x3m x 3m footprint; 105mm x 105mm columns @ 1000mm oc, w/ (2) hold-downs @ bottom, (1) hold-down @ top; rigid (R) diaphragm w/ blocking & perimeter nailing, flexible (S) diaphragm w/o blocking & nailing | 7.5mm plywood nailed w/ N50 nails @ 150mm oc | None | None | Displacement response of wall opposite of loading location (X3) is 2x as large for rigid diaphragm as it is for flexible diaphragm; No noticeable difference in response of wall at loading location (X1) between the diaphragms. |
| 27 | Effect of Transverse Walls and Vertical Load on the Performance of Shear Walls without Hold-down | Cheng, Ni, Lu, Karacabeyli | 2006 | Canada | Monotonic w/ flexible (pinned) spreader beam @ 7.5 mm/min; Cyclic w/ flexible (pinned) spreader beam per ISO 16670; Vertical load of 3.5 kN/m | Built to Part 9 of 1995 NBCC; 1 story w/ transverse walls, 6m x 2.4m main wall, 1.2m x 2.4m transverse walls, SPF No. 2 38mm x 89mm studs @ 400mm oc, nailed w/ 89mm x 39mm common nails, (1) baseline full wall, (1) garage door (3.6m x 2.1m), (1) regular door (1.2m x 2.1m) | 9.5mm OSB nailed w/ 65mm x 3.3mm common nails @ 150mm & 300mm oc | None | None | 1) Baseline (monotonic) - 10.7 kN/m; 2) Garage door (monotonic) - 6.9 kN/m; 3) Garage door (monotonic) (side walls loaded vert) - 8.6 kN/m; 4) Garage door (monotonic) (main wall loaded vert) - 10.1 kN/m; 5) Regular door (monotonic) (side walls loaded vert) - 7.8 kN/m; 6) Regular door (monotonic) (main wall loaded vert) - 8.6 kN/m |
| 28 | Effect of Upper Storey / Floor on the Performance of Wood Shear Walls | Liu, Ni, Rainer, Wensheng | 2006 | Canada | Monotonic w/ flexible (pinned) and rigid spreader beams @ 7.5 mm/min; Cyclic w/ flexible (pinned) and rigid spreader beams per ISO 16670; Vertical load of 3.5 kN/m on one wall, others had no vertical load other then second story dead weight | Built to Part 9 of 1995 NBCC; 1 & 2 story walls, 6m x 2.4m walls, SPF No. 2 38mm x 89mm studs @ 400mm oc, nailed w/ 89mm x 3.9mm common nails, (1) baseline 1 story w/ flex beam, (1) baseline 1 story w/ rigid beam, (1) 2 story w/ garage door (3.6m x 2.1m) (bottom) and two windows above (no vert load added), (1) 2 story w/ garage door (3.6m x 2.1m) (bottom) and two windows above (vert load added) | 9.5mm OSB nailed w/ 65mm x 3.3mm common nails @ 150mm & 300mm oc | None | None | 1) Baseline (monotonic) w/ flex beam - 6.4 kN/m of full height sheathing; 2) Baseline (monotonic) w/ rigid beam - 8.1 kN/m of full height sheathing; 3) 2 story w/o added vert load (monotonic) - 9.3 kN/m; 4) 2 story w/ added vert load (monotonic) - 12 kN/m |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 29 | Racking Tests on Rooms and Isolated walls to Investigate Uplift Restraint and Systems Effects | Thurston | 2006 | New Zealand | Cyclic, directly into ceiling diaphragm (strengthened) | 1/2 scale rooms, 3.3m x 2.1m x 1.1m high; 90mm x 45mm stud framing @ 600mm oc, single 620mm x 700mm window walls parallel to load, single 975mm x 680mm door in walls perpendicular to load | 9mm MDF board nailed w/ 30mm x 2.5mm couts @ 150mm oc (perimeter only) | 10mm gypsum plaster board screwed w/ 32mm x 6g screws @ 150mm oc (perimeter only) | None | 1) R1 (int. sheathing only, w/o tape and mud) ≈22.5 kN; 2) R2 (int. sheathing only, taped and mudded) ≈25 kN; 3) R3 (int. & ext. sheathing, BP nailed to foundation) ≈39 kN; 4) R4 (int. & ext. sheathing, BP bolted to foundation) ≈55 kN; 5) R5 (both sides used int. sheathing, corners w/ wing walls, BP nailed to foundation) ≈39 kN; 6) R6 (both sides used int. sheathing, corners w/ wing walls, 3 kN strap at door openings) ≈46 kN; 7) R7 (both sides used int. sheathing, corners w/ wing walls, BP bolted to foundation) ≈39 kN |
| 30 | Tests on Partially Anchored Wood-Framed Shear Walls | Girhammar, Kallsner | 2006 | Sweden | Monotonic w/ hydraulic jacks at top plate of wall, constant displacement rate, vertical loads applied to simulate construction above | 2400mm and 4800mm long walls, Pine C24 45x120mm studs @ 600mm oc | C40, 8mm thick hardboard nailed w/ 50mm x 2.1mm ring shank nails, pre-drilled holes, @ 100mm and 200 mm oc | None | None | 1) 3.23 kN vert load @ tension corner, 2400mm long wall - 19.4 kN; 2) 6.46 kN vert load @ tension corner, 2400mm long wall - 21.9 kN; 3) 1.29 kN total vert load (spread uniformly across top), 4800mm long wall - 55.3 kN; 4) 3.23 kN total vert load (spread uniformly across top), 4800mm long wall - 55.5 kN; 5) 6.46 kN total vert load (spread uniformly across top), 4800mm long wall - 58.4 kN; |
| 31 | Seismic Testing of a Full Scale Two Story Light-Frame Wood Building: NEESWood Benchmark Test | Christovasilis, Filiatrault, Wanitkorkul | 2006 | USA | Dynamic tri-axial w/ shake table, ordinary ground motions of Design Basis Earthquake (DBE) and near-field ground motions of Max Credible Earthquake (MCE) | Built to 1988 UBC, CUREE index building, 2 story house, 22' x 58'-4" footprint; 2x4 hem-fir framing @ 16" oc | 7/16" plywood nailed w/ 8d common nails @ 6", 4" or 3" and 12" oc. | 1/2" gypsum wall board (walls and ceilings), horizontal, screwed w/ #6 screws @ 16" oc | 7/8" plaster stucco | In long dimension direction: 1) Phase 1 (wood structural elements only) - 18.6k @ 0.32"; 2) Phase 3 (gypsum installed on structural walls) - 21.6k @ 0.30"; 3) Phase 4 (gypsum installed on all walls and ceiling) - 22.8k @ 0.27"; 4) Phase 5 (stucco applied on exterior) - 29.7k @ 0.12"; In short dimension direction: 5) Phase 1 (wood structural elements only) - 41.0k @ 2.50"; 6) Phase 3 (gypsum installed on structural walls) - 35.3k @ 1.39"; 7) Phase 4 (gypsum installed on all walls and ceiling) - 35.8k @ 1.02"; 8) Phase 5 (stucco applied on exterior) - 38.2k @ 0.65"; |
| 32 | Shear Wall Test | Noory, Smith, Asiz | 2006 | Japan | Monotonic w/ loading beam, vertical load of 11.1 kN applied uniformly (ASTM E 564) | 12' x 8' walls, Built to recommendations of Canadian Mortgage and Housing Corporation, 38mm x 89mm SPF #2 @ 610mm oc | 11.1mm OSB nailed w/ 60mm pneumatic nails @ 150mm and 300mm oc | 12.7mm gypsum wall board screwed w/ 40mm screws @ 150mm and 300mm oc | 38mm rigid insulation; 85mm x 17mm x 2.4m long wood strips spaced @ 406mm, nailed through insulation to OSB w/ 16d nails; 1x8 clapboard siding nailed w/ 63.5mm nails into wood strips | Ultimate load: 1) Framing and OSB only - 20 kN @ 26mm; 2) Insulation and wood straps added - 21 kN @ 31mm; 3) Clapboard siding added - 23 kN @ 19mm; 4) Interior gypsum added - 23.5 kN @ 13mm |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 33 | Full Scale Shaking Table Tests of 3-Story Wood-frame Construction Buildings | Okiura, Kawai, Umemori, Murakami, Isoda | 2006 | Japan | Dynamic tri-axial w/ shake table (ground motion records from Kobe earthquake wave, 1995) | Two 3 story houses, 7.28m x 7.28m footprint; 2x4 wood-frame construction, house A w/ only wood framing, house B w/ finish materials including cement slate plates on roof | 9mm plywood nailed w/ CN50 nails @ 100mm oc | 12.5mm gypsum wall board w/ GNF40 @ 100mm oc (house A and B); 9mm plywood nailed w/ CN50 nails @ 75mm oc (house B) | Ceramic siding boards (house B) | Ultimate base shear: House A - 230 kN; House B - 289 kN; Story Stiffnesses: House A Story 3 - 4.61 kN/mm (x-dir) and 5.79 kN/mm (y-dir); House A Story 2 - 4.61 kN/mm (x-dir) and 5.61 kN/mm (y-dir); House A Story 1 - 6.26 kN/mm (x-dir) and 6.69 kN/mm (y-dir); House B Story 3 - 4.61 kN/mm (x-dir) and 5.79 kN/mm (y-dir); House B Story 2 - 5.95 kN/mm (x-dir) and 5.61 kN/mm (y-dir); House B Story 1 - 7.23 kN/mm (x-dir) and 7.91 kN/mm (y-dir); |
| 34 | Shaking Table Tests and Seismic Design Method of Wooden Houses with Eccentricity | Takahiro, Mikio, Katsuhiko, Kenji, Yoshimitsu, Isao | 2006 | Japan | Dynamic uni-axial w/ shake table (ground motion records from JMA Kobe N-S wave, 1995); weights installed below roof, between 7.3 kN and 14.2 kN | Built using conventional Japanese frame construction (columns and plywood), 1 story, 4.55m x 3.64m footprint; 105mm x 105mm columns @ 910mm oc, stiff roof diaphragm using 105mm x 180mm beams at each column, 12mm thick plywood sheathing on top; soft stiffness roof diaphragm using same beams, 45mm x 60mm joists @ 455mm and 180mm wide x 13mm thick plants in dir of loading, nailed w/ (2) nails per plank; various configurations/layouts of exterior walls with openings to create eccentricity, openings either 910mm, 1820mm or 2730mm wide | 9mm plywood nailed w/ 45mm x 2.87mm nails @ 200mm oc along columns | None | None | Stiff Roof: Symmetrical layout (D8) - 14.2 kN and 13.61 kN; One less shear panel on one side (D9) - 6.14 kN and 5.14 kN; Two less shear panels on one side (D10) - 9.53 kN and 5.8 kN; Soft Roof: Symmetrical layout (D8) - 8.17 kN and 8.29 kN; One less shear panel on one side (D9) - 8.17 kN and 6.58 kN; Two less shear panels on one side (D10) - 4.36 kN; |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 35 | Maximum Wall Bracing Rating that is Compatible with NZS 3604 Construction Phase II - Testing of Room 1 | Thurston | 2007 | New Zealand | Cyclic, directly into ceiling diaphragm @ each wall; vertical floor and roof live load of 0.2 kPa added using weights | Built to NZS 3604:1999, 1 story, 6.49m x 3.29m footprint, 2.42m high; founded on piles (not continuous foundation), 90mm x 45mm stud framing @ 600mm oc, (1) - 1.95m x 0.8m door and (1) - 1.6m x 1.235m window in each wall parallel to load, strapping tied stud to bottom plate at all openings, not to foundation directly though | None | 10mm GIB Braceline® by Winstone Wall boards | None | Test 2 - max applied load = 46.9 kN (total length of bracing elements 7.48m) |
| 36 | Shaking Table Tests of Two-Story Passively-Controlled Wood Frames with Inner and Outer Walls | Matsuda, Sakata, Kasai, Ooki | 2008 | Japan | Dynamic uni-axial w/ shake table (ground acceleration records from Kobe earthquake wave, 1995, max acceleration of 0.83g); weights of 1.44 kN/$m^2$ and 1.67 kN/$m^2$ added | Built using conventional Japanese frame construction (columns and plywood), 2 story frame, 2.73m x 2.73m footprint; 3 column lines x 4 column lines, 105mm x 105mm columns @ 910mm oc, 105mm x 30mm intermediate columns, 105mm x 180mm beams, 28mm thick plywood floor and roof | 1m wide wood panel "structural element" w/ 1.96m shear force resistance, located in center column line of frame; Visco-elastic dampening k-braces | 12mm gypsum wall board screwed @ 150mm oc on columns and intermediate columns | 12mm ceramic siding (only on center bay) | Not taken to failure: 1) Wood panel structural elements w/ gypsum and ceramic siding ≈47 kN @ 35mm; 2) Visco-elastic dampeners w/ gypsum only ≈40 kN @ 20mm; 3) Visco-elestic dampeners w/ gypsum and ceramic siding ≈42 kN @ 15mm |
| 37 | Shaking Table Test and Seismic Property Evaluation of Wooden House with Various Types of Shear Walls | Umemori, Tsuzuki, Miyazawa | 2008 | Japan | Dynamic uni-axial and tri-axial w/ shake table (ground acceleration records from Kobe earthquake wave, 1995); weight was added to simulate building materials and live load | Built using conventional Japanese frame construction (columns and plywood), 2 story house, 6.2m x 4.2m footprint; 105mm x 105mm columns @ 910mm oc, 2m x 2.03m openings in centers of walls, 1m x full height openings at ends of long dimension (2 out of 3 configurations reported on here), hold down connectors at all shear wall ends | 9.5mm plywood; 11mm OSB SIPs panels | 12.5mm gypsum wall board; 9.5mm plywood | None | Story Shear Forces: 1) Configuration 1 (plywood and gypsum in short direction, plywood both sides in long direction) - 118.1 kN @ 1/26 rad (long direction) and 117.6 kN @ 1/34 rad (short direction); 2) Configuration 2 (SIPs panels, gypsum and plywood in center short direction wall) - 116.2 kN @ 1/44 rad (long direction) and 185.2 kN @ 1/37 rad (short direction); 3) Configuration 3 (SIPs panels, corners closed i.e., openings towards center) - 117.2 kN @ 1/39 rad (long direction) and 122.2 kN @ 1/41 rad (short direction) |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 38 | Seismic Performance Verification of Traditional Wooden House Based on Cyclic Loading Tests and Analytical Methods | Nakaji, Yamada, Suda, Suzuki | 2008 | Japan | Cyclic, directly to top of frame at each frame, in short direction | Built using traditional Japanese frame construction (columns and frames) circa 1898, 1 story house with raised copula like middle roof section, 18.5m x 9.55m footprint; 175mm x 175mm columns, 135mm x 330mm beams | None | None | 75mm mud-plastered hanging walls | Frame X1 (4.6m high, (3) - bays of 1.82m, 1.82m and 3.64m) - 106.6 kN @ 0.09 rad |
| 39 | Moment Resistance of Traditional Wooden Structure by Dynamic and Static Tests | Maeno, Suzuki, Saito | 2008 | Japan | Dynamic uni-axial w/ shake table (ground motion records from El Centro (1940) max accel 250 $cm/s^2$); static racking at top of frame; 110 kN weight added on top | Built using traditional Japanese frame construction (columns and frames) circa 1898, 1 story frame, 3.75m x 2.25m footprint; 308mm round columns, 114mm x 181mm beams, notched joints only, columns only set on foundation (no pins) | Frame only | None | None | Bending moment at top of columns: Dynamic loading - 0.9 kN*m @ 1/120 rad; Bending moment at bottom of columns: Dynamic loading - 1.8 kN*m @ 1/120 rad; |
| 40 | Pseudo-dynamic Lateral Load Tests on Full-Scale Two Story Wooden Structure with Moment Resisting Frames | Yasumura, Uesugi, Davenne, Suzuki | 2008 | Japan | Dynamic uni-axial w/ hydraulic jacks at each story height (ground motion records from El Centro (1940) max accel 20.5g) | Built to 2000 Japanese building codes, 2 story frame only, 6m x 3m footprint; 120mm x 300mm columns and 120mm x 390mm beams w/ moment connections in loading direction, columns fixed to base of test apparatus, 120mm x 240mm beams in transverse direction, single glue laminated sugi panel at each corner of transverse wall plane, same panels used as flooring | Moment frames using moment connections of (14) 12mm dowels and (12) 12mm bolts; transverse direction - 30mm thick x 1000mm wide glue laminated sugi panels (similar to plywood construction) screwed w/ 69.3mm x 3.85mm screws @ 150mm oc | None | None | Lateral load from actuator load cells: First floor - 90 kN @ 75mm; Second floor - 70 kN @ 60mm |
| 41 | Full-scale Shaking Table Tests of Wood Framed Houses | Ohashi, Watahiki, Machida | 2008 | Japan | Dynamic uni-axial w/ shake table (ground motion records from JMA Kobe N-S wave, 1995); weights to simulate floor live load of 0.6 $kN/m^2$ | Built to traditional Japanese construction (columns and frames), 2 story house, 8.19m x 9.1m footprint; 85mm x 300mm columns, 120mm x 330mm beams | Diagonal wood brace; Cross Panel (wood lattice type panel) | 12mm gypsum wall board | 12mm ceramic siding | Braced home - 315 kN @ 0.034 rad; Cross Panel home - 360 kN @ 0.039 rad |

| # | Title | Author | Year | Country | Loading Type | Design/Code Compliance | Exterior Bracing | Interior Finish | Exterior Finish | Ultimate Tested Strength |
|---|---|---|---|---|---|---|---|---|---|---|
| 42 | Comparison of Seismic Performance of an Aged Wooden House and Newly Built One | Fukumoto, Koshihara, Tsuchimoto, Kawai, Isoda, Shimizu | 2008 | Japan | Dynamic tri-axial w/ shake table (ground motion records from JR-TAKATORI wave, 1995) | Built to traditional Japanese construction (columns and frames) circa 1974, 2 story house, 5.94mm x 5.82mm footprint; weights to simulate roof finish and 2nd floor live load added | Frame only | Inner mud wall | Mortar finish w/ metal lath on wood lath | Ultimate load in Y-direction: Old house - 100 kN; New house - 140 kN; Ultimate load in X-direction: Old house - 205 kN; New house - 200 kN |

# APPENDIX B – Example Calculations

**Example Calculation: the Sugiyama PSW (Method 1)**
**CSIRO House Test**

Nominal unit shear capacities:

**$F_{sOSB}$** = 560 *plf* [Table 4.3A of AF&PA Wind and Seismic Design Provisions – 2005 Edition: 3/8" WSP – Sheathing, 6d, 6/12 nailing, wind]
**$F_{sGyp}$** = 100 *plf* [Based on extrapolation from tabulated values in Table 2305.3.8.2 in 2006 IBC]

Capacities are additive per Section 2305.3.9 of 2006 IBC for wind design:
**$F_s$** = 560 *plf* + 100 *plf* = 660 *plf*

**$F_{wall}$** = **$F_s$ L $C_{op}$**

Where:
**$F_{wall}$** = Capacity of perforated shear wall line
**$F_s$** = Un-adjusted nominal unit shear capacity
**L** = Length of the perforated shear wall
**$C_{op}$** = Opening adjustment factor according to Sugiyama equation

$$C_{op} = \frac{r}{(3-2r)}$$

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

Where:
**$A_o$** = Total area of the openings
**H** = Height of the wall
**$\Sigma L_i$** = Summation of length of full height wall segments

**Wall W1**
*(8-foot high, 20.2-foot long wall with one 16.4-foot wide by 6.9-foot high garage door opening in center; sheathed with OSB on exterior, gypsum on interior)*

No maximum requirement on qualifying aspect ratios

$$r = \frac{1}{(1+\frac{6.9\,ft \times 16.4\,ft}{8\,ft \times 3.8\,ft})} = 0.213$$

$$C_{op} = \frac{0.213}{(3-(2\times 0.213))} = 0.083$$

Ultimate capacity of the wall:
**$F_{wall}$** = $660\,plf \times 20.2\,ft \times 0.083$ = 1,102 *lb*

**Example Calculation: the 2006 IBC PSW (Method 2) with Wind Design Requirements**
**CSIRO House Testing**

Nominal unit shear capacities:

$\mathbf{F_{sOSB}}$ = 560 *plf* [Table 4.3A of AF&PA Wind and Seismic Design Provisions – 2005 Edition: 3/8" WSP – Sheathing, 6d, 6/12 nailing, wind]
$\mathbf{F_{sGyp}}$ = 100 *plf* [Based on extrapolation from tabulated values in Table 2305.3.8.2 in 2006 IBC]

Capacities are additive per Section 2305.3.9 of 2006 IBC for wind design:
$\mathbf{F_s}$ = 560 *plf* + 100 *plf* = 660 *plf*

$\mathbf{F_{wall}}$ = $\mathbf{F_s\,L\,C_{op}}$

Where:
$\mathbf{F_{wall}}$ = Capacity of perforated shear wall line
$\mathbf{F_s}$ = Un-adjusted nominal unit shear capacity
**L** = Sum of the lengths of qualifying wall segments
$\mathbf{C_o}$ = Opening adjustment factor from Table 2305.3.8.2 of 2006 IBC

**Wall W1**
*(8-foot high, 20.2-foot long wall w/ one 16.4-foot by 6.9-foot high garage door opening in center; sheathed with OSB on exterior, Gypsum on interior)*

Wind design requirements per Section 2305.3.4 of 2006 IBC place a maximum limit of 3.5:1 on all qualifying wall segments.

Check for qualifying wall segments:

Maximum allowed aspect ratio ≤ 3.5:1

Wall segment aspect ratio = $\frac{8\,ft}{1.9\,ft}$ = 4.13 > 3.5 doesn't qualify for wind design

**No qualifying wall segments in Wall W1**

**Wall W4**
*(8-foot high, 37-foot long wall w/ one 8-foot by 4-foot high window opening, one 4-foot by 6.5-foot door opening and one 9.5-foot by 6.5-foot door opening; narrowest fully sheathed wall segment is 3.9-foot long; sheathed with OSB on exterior, gypsum on interior)*

Maximum allowed aspect ratio ≤ 3.5:1

Typical wall segment aspect ratio = $\frac{8\,ft}{3.9\,ft}$ = 2.05 ≤ 3.5 check ok for Wind Design

$\mathbf{F_s}$ = 660 *plf*

Calculate the percent of the wall line that has full height sheathing:

% of Full Height Sheathing = $\frac{3.9\,ft + 3.9\,ft + 3.9\,ft + 3.9\,ft}{37\,ft}$ = 43%

Interpolate $\mathbf{C_o}$ from Table 2305.3.8.2 for maximum opening heights between 5H/6 and H and percent of full height sheathing between 40% and 50%:

$\mathbf{C_o}$ = 0.55

Calculate ultimate capacity of the wall:

$\mathbf{F_{wall}}$ = $660\,plf \times (3.9\,ft + 3.9\,ft + 3.9\,ft + 3.9\,ft) \times 0.55$ = 5,709 *lb*

**Example Calculation: the 2006 IBC PSW (Method 2) with Seismic Design Requirements**
**CSIRO House Testing**

Nominal unit shear capacities:

**$F_{sOSB}$** = 560 *plf* [Table 4.3A of AF&PA Wind and Seismic Design Provisions – 2005 Edition: 3/8" WSP – Sheathing, 6d, 6/12 nailing, wind]

**$F_{wall} = F_s\, L\, C_{op}$**

Where:

**$F_{wall}$** = Capacity of perforated shear wall line
**$F_s$** = Un-adjusted nominal unit shear capacity
**L** = Sum of the lengths of qualifying wall segments
**$C_o$** = Opening adjustment factor from Table 2305.3.8.2 of 2006 IBC

**Wall W1**
*(8-foot high, 20.2-foot long wall w/ one 16.4-foot by 6.9-foot high garage door opening in center; sheathed with OSB on exterior, Gypsum on interior)*

Seismic design requirements per Section 2305.3.4 of 2006 IBC place a maximum limit of 2:1 on all qualifying wall segments. If aspect ratio is greater than 2:1 but less than 3.5:1, **$F_s$** must be reduced by 2w/h.

Check for qualifying wall segments:

Maximum allowed aspect ratio ≤ 2:1

Typical wall segment aspect ratio = $\frac{8\,ft}{1.9\,ft}$ = 4.13 > 3.5 doesn't qualify for seismic design

**No qualifying wall segments in Wall W1**

**Wall W4**
*(8-foot high, 37-foot long wall w/ one 8-foot by 4-foot high window opening, one 4-foot by 6.5-foot door opening and one 9.5-foot by 6.5-foot door opening; narrowest fully sheathed wall segment is 3.9-foot long; sheathed with OSB on exterior, Gypsum on interior)*

Check for qualifying wall segments:

Maximum allowed aspect ratio ≤ 3.5:1

Typical Wall Segment Aspect Ratio = $\frac{8\,ft}{3.9}$ = 2:0 ≤ 2.05 ≤ 3.5:1

Reduce the strength of the wall for seismic design:

Aspect Ratio Adjustment Factor = $\frac{2\times(3.9\,ft)}{8\,ft}$ = 0.99

Per Section 2305.3.9 of 2006 IBC, unit shear capacity of shear walls with dissimilar sheathing materials designed for anything other than wind shall be the greater of:

2x the capacity of the weaker material

or

the capacity of the stronger material

$\mathbf{F_s}$ = 560 *plf*

Calculate the percent of the wall line that has full height sheathing:

% of Full Height Sheathing = $\frac{3.9\,ft + 3.9\,ft + 3.9\,ft + 3.9\,ft}{37\,ft}$ = 43%

Interpolate $\mathbf{C_o}$ from Table 2305.3.8.2 for maximum opening heights between 5H/6 and H and percent of full height sheathing between 40% and 50%:

$\mathbf{C_o}$ = 0.55

Calculate ultimate capacity of the wall:

$\mathbf{F_{wall}}$ = $560\,plf \times 0.99 \times (3.9\,ft + 3.9\,ft + 3.9\,ft + 3.9\,ft) \times 0.55$ = 4,796 *lb*

**Example Calculation: PC1 RB148 Method (Method 3)**
**CSIRO House Testing**

Nominal unit shear capacities:

**$F_{sOSB}$** 3/8" = 560 *plf* [Table 4.3A of AF&PA Wind and Seismic Design Provisions – 2005 Edition: WSP – Sheathing, 6d, 6/12 nailing, wind]
**$F_{sGyp}$** = 100 *plf* [Based on extrapolation from tabulated values in Table 2305.3.8.2 in 2006 IBC]

Capacities are additive per Section 2305.3.9 of 2006 IBC for wind design:

**$F_s$** = 560 *plf* + 100 *plf* = 660 *plf*

**$F_{wall}$** = **$F_s$ L $C_{prf}$**

Where:

**$F_{wall}$** = Capacity of perforated shear wall line
**$F_s$** = Un-adjusted nominal unit shear capacity
**$C_{prf}$** = Partial restraint factor [from RB148]
**L** = Sum of the lengths of qualifying wall

Aspect ratio criteria for continuously sheathed structures:

Per Section R602.10.5 of 2006 IRC, buildings having continuous wood structural panel sheathing above and below all openings shall have minimum braced wall lengths in accordance with Table R602.10.5. For segments next to wall openings, aspect ratio limits range from 2:1 to 4:1 depending on the opening height. At garage openings in one-story structures, segments with 4:1 aspect ratio are permitted.

**Wall W1**

*(8-foot high, 20.2-foot long wall w/ one 16.4-foot by 6.9-foot high garage door opening in center; sheathed with OSB on exterior, gypsum on interior)*

Maximum allowed aspect ratio ≤ 4:1

Typical wall segment aspect ratio = $\frac{8\,ft}{1.9\,ft}$ = 4.13 > 4 doesn't qualify

**No qualifying wall segments in Wall W1**

However, because the aspect ratio is close to the IRC limit, analysis is performed for both scenarios, with and without W1, to investigate the impact on the system factor.

**Wall W4**

*(8-foot high, 37-foot long wall w/ one 8-foot by 4-foot high window opening, one 4-foot by 6.5-foot door opening and one 9.5-foot by 6.5-foot door opening; narrowest fully sheathed wall segment is 3.9-foot long; sheathed with OSB on exterior, gypsum on interior)*

Check adjacent opening % of full wall height:

% = $\frac{6.9\,ft}{8\,ft} \times 100$ = 87%

Per Table R602.10.5, wall segments with adjacent openings of 87% of the wall height must have maximum aspect ratio of 3:1.

Maximum allowed aspect ratio ≤ 3:1

Typical wall segment aspect ratio = $\frac{8\,ft}{3.9\,ft}$ = 2.05 ≤ 3 check ok

For shear wall carrying only roof above:

$\mathbf{C_{prf}}$ = 0.8

Calculate ultimate capacity of the wall:

$\mathbf{F_s}$ = 660 *plf*
$\mathbf{F_{wall}}$ = $660\,plf \times (3.9\,ft + 3.9\,ft + 3.9\,ft + 3.9\,ft) \times 0.8$ = 8,315 *lb*