

The Story Behind the 2009 IRC Wall Bracing Provisions (Part 2: New Wind Bracing Requirements)

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Introduction

A previous article (Crandell 2007) discussed the engineering analysis approach used to derive seismic wall bracing amounts in the *International Residential Code (IRC)* (ICC 2006). Also mentioned was the lack of a similarly rationalized set of wall bracing requirements to resist wind loads. This article documents the development of a rational and quantitative wall bracing analysis approach for resisting wind loads. The efforts have led to new wall bracing provisions in the 2009 IRC for wind.

Historically, residential wall bracing code provisions have relied heavily on traditional norms from past experience. Some IRC wall bracing methods (e.g., diagonal wood boards) and provisions (e.g., brace at corners and every 25 ft.) date back to the 1950s and beyond. The adequacy of these provisions have become increasingly questioned as housing styles and sizes have changed, in some cases significantly, since the 1950s (HUD 2001). In general, homes have become larger resulting in increased wind forces due to a larger profile or “sail area.” Modern homes also tend to have more open floor plans with larger interior spaces that create higher demands on designated wall bracing and less “reserve” capacity to resist wind loads. Modernization of the IRC wall bracing provisions has been deemed important by a wide variety of interested parties so that the provisions work better with modern home designs, materials, and construction techniques. Furthermore, having a rational basis for the wall bracing provisions also promotes:

1. consistency in performance requirements for various bracing methods and materials and
2. a transparent framework for making future advancements or changes.

Development of new wind bracing provisions for the 2009 IRC has occurred under the purview of an Ad Hoc Wall Bracing (AHWB) committee established by the International Code Council, Inc. (ICC). A second group established by Dr. Dan Dolan with support from the Building Seismic Safety Council (BSSC) served as a forum for consideration of various technical data and analysis decisions.

Background

Accurate analysis of lateral resistance of conventional woodframe buildings has been an elusive endeavor because of the complexity and redundancy of load paths associated with these buildings which, on the surface, appear as deceptively simple structures. The first major attempt at providing lateral design data and guidance relevant to conventional light woodframe building design is documented in a 1948 report by the National Bureau of Standards (NBS 1948). In 2000, a similar review and documentation of state-of-the-art design practices and data related to the structural performance of conventional wood frame buildings was attempted (NAHB 2000).

Over the past several years, both AHWB committees considered a significant amount of data toward the goal of providing a consistent rational basis for bracing of conventional woodframe homes against lateral wind loads. A white paper and test database documented much of the available wall bracing test data dating from the 1920s to present (Crandell 2006). The purpose of this database was to help characterize in-plane shear performance of conventional bracing methods under various standardized test methods. Most of the historic test data tended to rely on idealized wall segment boundary conditions whereby braced wall segments were tested using rigid or full restraint against overturning (e.g., ASTM E 72 or ASTM E 564). Also considered were some more recent large scale tests by APA–The Engineered Wood Association (Martin et al. 2007a), Simpson Strong Tie (2007), and National Association of Home Builders (NAHB) Research Center (2008). These tests departed from standard test methods by exploring braced wall panel boundary or restraint conditions representative of actual end-use conditions in real, conventional framed buildings or wall assemblies. Some of these newer tests were additionally summarized in an expanded test database during the course of deliberations (Martin et al. 2007b). In addition, several conventional woodframe house tests were considered in detail in the final stages of the effort (**Appendix A**).

In general, the test data demonstrated that performance of a 4-foot braced wall panel without any overturning restraint provided by surrounding building components or test rigging was reduced by 75 percent compared to the fully restrained counterpart. But, because the test assembly containing a braced wall panel included more of the surrounding structure (e.g., corners, finishes, longer walls, rim joist above, etc.), the individual braced wall panel performance tended to approach that observed under idealized test conditions of full restraint. Various whole building tests exhibited the full potential of these effects in addition to other building system contributions to lateral strength (Appendix A).

Overview of Analysis Framework

Balancing wind load demand and wall bracing capacity served as the logical basis of the analysis framework developed by the Dolan-AHWB Committee. By far, the greatest challenge was reaching agreement on the capacity, or strength, of conventional wall bracing segments because such segments do not have explicit overturning restraint (i.e., hold-down brackets) conducive to use of accepted engineering analysis methods. Thus, expert opinions about appropriate design strength for braced wall segments varied widely. After several years of committee work and review of all of the available and relevant testing, a logical and simple framework to determine load demand and wall bracing capacity was agreed upon as:

$$\text{braced wall capacity} = (\text{fully restrained shear wall capacity}) \times (\text{net adjustment factor})$$

Shear wall capacity is based on code-recognized values or testing in the absence of relevant code recognized values. The net adjustment factor was taken as the product of a partial restraint factor and a whole-building factor, which was simplified to a value of 1.2 for all cases for reasons explained later. The actual values of the separate terms were not specifically agreed upon by either committee. As such, the net adjustment factor could be grossly characterized as a “calibration factor” to bring results in line with historic bracing requirements for 1950s or 1960s era 1,500 ft.² or less, two story or less, conventionally constructed houses. Because more than two years of effort was spent studying partial restraint effects and whole building test results, these effects are separately discussed in this paper.

Analysis Details

Shear Wall Capacities for IRC Bracing Methods

After reviewing available data (see Background), nominal unit shear values (under conditions of full-restraint against overturning) were agreed upon as documented in Table 1. The values in Table 1 apply to bracing that is fully restrained against overturning by way of connection hardware, building assembly or dead weight constraining forces, or test rigging. As nominal values, they also do not

include a safety factor which must be separately applied to determine allowable stress design (ASD) values.

With few exceptions, the values in Table 1 are derived directly from 2005 *Special Design Provisions for Wind and Seismic (SDPWS)* (AF&PA 2005). Values were adjusted to apply to spruce-pine-fir framing. Also consistent with the 2005 *SDPWS*, a factor of 2.0 was used for the purpose of converting nominal strength values to ASD values.

Values for bracing materials and methods not addressed in the 2005 *SDPWS*, such as 1 by 4 wood let-in braces, were derived from test data referenced in the Background section of this paper using a worst-case (tension) loading direction. Also, it should be noted that the nominal bracing values in Table 1 include 1/2-inch gypsum wallboard interior finish installed in accordance with Chapter 7 of the *IRC*. Thus, for walls without interior finish, the nominal values shown in Table 1 should be reduced in accordance with footnote 2 of Table 1. This reduction does not apply to Method 5 (GB) bracing. Finally, bracing requirements in the *IRC* were simplified by grouping bracing methods into two categories based on nominal shear strength: 700 plf and 400 plf as shown in Table 1.

Because of the difficulty and uncertainty in accurately associating individual wall panel test data with actual in-plane (racking) deflections (drift) in actual buildings and due to a primary concern with achieving improvements in the safety of wall bracing requirements in the *IRC*, the AHWB committees did not institute deflection limits in its determination of the nominal bracing values in Table 1.

Partial Restraint Effects on Shear Wall Capacities

In actual conventional woodframe construction, the degree of overturning restraint provided to braced wall segments varies. Therefore, it was necessary to consider how various conventional bracing methods perform with a degree of restraint (partial restraint) representative of end-use conditions.

The topic of partial restraint and its effect on wall bracing performance relative to fully restrained nominal shear values was the focus of much inquiry. The inquiry included attempts to develop and implement a partially restrained test method. This effort, however, ultimately failed to give a conclusive and objective “one-size-fits-all” answer to partial restraint effects. Effects of partial restraint are complex, variable, and difficult to separate from other whole building effects which can be fully realized only when testing whole buildings. Conditions that were found to have a significant effect on the degree of partial restraint provided to a conventional wall bracing element included:

- Length of wall extending beyond either end of the bracing element
- Wall components or opening conditions adjacent to a bracing element
- Support conditions (framing assembly stiffness and dead load above the bracing element)

Table 1.—Conventional woodframe wall bracing methods and nominal unit shear strength values.^a

Bracing method	Description and installation requirements	Nominal unit shear strength ^b		Value Used for 2009 IRC
		16 in. o.c.	24 in. o.c.	
1 (LIB)	1 by 4 let-in brace, No. 1 spruce, two 8d common (2.5 by 0.131 in.) or three 8d box (2.5 by 0.113 in.) nails per stud and plate bearing, 45° brace angle relative to plates, studs at 16 in. o.c. max (or approved equivalent metal brace). ^c	420 plf	NP	400 plf
2 (DWB)	1 by 6 or 1 by 8 diagonal wood boards at 45° angle relative to plates with two 8d common (2.5 by 0.131 in.) nails at each stud and three 8d common (2.5 by 0.131 in.) nails at each plate.	975 plf	923 plf	700 plf
3 (WSP)	Minimum 3/8 in. thick wood structural panel with 6d common (2 by 0.113 in.) nails at 6 in. o.c. (panel edges) and 12 in. o.c. (intermediate supports) or 16 ga. by 1.75 in. staples at 3 in. o.c. (panel edges) and 6 in. o.c. (intermediate supports).	715 plf	665 plf	700 plf
4 (SFB)	1/2 in. or 25/32 in. thick structural fiberboard sheathing with nails or staples in accordance with IRC Table R602.3(1) at 3 in. o.c. (panel edges) and 6 in. o.c. (intermediate supports), studs at 16 in. o.c. max.	795 plf	NP	700 plf
5 (GB) both sides	1/2 in. or 5/8 in. gypsum wall board or sheathing panels on both sides of wall with both sides attached in accordance with minimum 5d cooler nails (0.086 by 1.625 in.) or screws at 7 in. o.c. on edges of panel, including top and bottom plates.	400 plf	300 plf	400 plf
	Same as above except edge fasteners at 4 in. o.c. and blocking is required at all horizontal joints not on framing members.	600 plf	500 plf	600 plf ^c
5 (GB) one side	1/2 in. or 5/8 in. gypsum wall board or sheathing panels on one side of wall attached in accordance with minimum 5d cooler nails (0.086 by 1.625 in.) or screws at 7 in. o.c. on edges of panel.	200 plf	150 plf	200 plf
	Same as above except edge fasteners at 4 in. o.c. and blocking is required at all horizontal joints not on framing members.	300 plf	250 plf	300 plf ^c
6 (PBS)	3/8 in. or 1/2 in. thick particleboard sheathing with 8d common (2.5 by 0.131 in.) nails at 3 in. o.c. (panel edges) and 6 in. o.c. (intermediate supports).	745 plf	NP	700 plf
7 (PCP)	Minimum 3/4 in. thick Portland Cement stucco applied to wire mesh or expanded metal lath with fasteners in accordance with IRC R703.6 spaced at 6 in. o.c. on all framing members.	740 plf	NP	700 plf
8 (HSP)	Minimum 7/16 in. thick hardboard panel siding (vertical installation only) fastened with minimum 0.092 by 2 in. nails at 4 in. o.c. (panel edges) and 8 in. o.c. (intermediate supports).	700 plf	NP	700 plf

^a Nominal unit shear values shall be divided by a factor of 2 to determine an ASD value. For strength design, nominal unit shear values shall be multiplied by a resistance factor of 0.8.

^b Nominal unit shear values for all bracing methods, except Method 5 (GB) as described, include minimum 1/2 in. gypsum wall board interior finish on inside face installed in accordance with Table R702.3.5 of the IRC. A nominal unit shear strength of 200 plf is assigned to the interior finish based in part on interior finish performance in whole building tests. To determine nominal unit shear strength without interior finish, subtract 200 plf from 16 in. o.c. values and 150 plf from 24 in. o.c. values.

^c Let-in brace value is for 45° brace; however, the length of brace determined as measured by horizontal distance along the wall for a 45° brace may be substituted by an equal length of wall braced with up to a 60° let-in brace.

^d Values for GB with 4 in. o.c. fastener spacing were implemented as a 0.7 adjustment factor applied to GB bracing amounts for 7 in. o.c. fastener spacing in footnote to Table R602.10.1(1) in the IRC 2009.

- Strength of bracing method relative to strength of conventional framing and connections providing restraint to a given brace panel at its boundaries.
- Contribution of non-structural components and non-compliant bracing elements in a whole house test.

While a range of values for partial restraint factors were considered by the Dolan-AHWB Committee, specific values could not be agreed upon. The values in **Table 2**, however, are reasonably consistent with average values discussed by the Dolan-AHWB Committee. In the end, distinguishing partial restraint effects from whole building effects was not

realized by consensus. Consequently, the ICC-AHWB Committee opted to use a single net adjustment factor of 1.2 as shown in **Table 2**.

Whole Building Effects

As a final step in developing the design approach, it was necessary to evaluate its ability to predict the performance of actual whole buildings constructed using conventional bracing methods. The intent of this step was to calibrate the design approach to actual whole building performance and, thus, better reconcile the design approach with past construction experience which necessarily is based on whole

Table 2.—Nominal shear strength adjustment factors for conventional wall bracing.^a

Walls supporting	Partial-restraint factor	Whole building factor	Net adjustment factor
Roof only	0.8	1.5	1.2
Roof + one story	0.9	1.33	1.2
Roof + two stories	1.0	1.2	1.2

^a These factors are limited to residential construction in accordance with the *2009 IRC* and bracing methods that have a nominal shear strength “capped” at about 700 plf in accordance with Table 1.

buildings. The answer was first explored by way of expert opinion based on the knowledge and experience of those familiar with lateral performance of light-frame buildings and wall assemblies. Finally, the expert opinion and design approach were compared directly to five lateral load tests of whole buildings constructed using conventional bracing methods and framing practices consistent with the *IRC* and U.S. home construction. A summary of whole house testing data considered by the Dolan-AHWB Committee can be seen in **Appendix A**.

As a result of reviewing these tests and considering the prior expert opinion, a judgment was made by the ICC-AHWB committee to apply a net adjustment factor of 1.2 which incorporated the combined effect of partial restraint and whole building system contributions to overall wall bracing performance. For the sake of completeness, “representative” factors are shown in **Table 2** to correspond with this decision; but, the individual factors were not specifically agreed upon by either AHWB committee.

Concern over the effects of a combined wind uplift and lateral load path posed one of the greatest difficulties in reaching agreement on the 1.2 net adjustment factor. While the ICC-AHWB committee’s decision was influenced by several tests of whole buildings under combined wind lateral load and uplift (**Appendix A**), additional research is needed in this area. Conventional wood framing relies on a combined load path where framing connections and components that resist overturning forces in wall bracing also resist roof uplift forces caused by wind. The problem is complicated by the spatial variation of this interaction of load paths in a whole building depending on wind direction and location of bracing that is resisting lateral forces for a given wind direction. In general, as wind uplift forces increase in higher wind zones or for particular building and roof configurations, conventional wall bracing performance will become increasingly degraded due to wind uplift effects which erode the degree of partial restraint otherwise provided to wall bracing segments. Thus, provisions were added to the *2009 IRC* wall bracing requirements to address the wind uplift load path (enhance conventional wall framing connections). These provisions apply when the design wind uplift forces at the top of a braced wall line exceed 100 plf.

Design Approach

A simple and effective design approach for determining lateral resistance of conventional wood frame homes is summarized as follows:

1. Determine braced wall line locations on the building plan for both plan directions.
2. Determine wind loads acting on each braced wall line using *ASCE 7 Minimum Design Loads for Buildings and Other Structures* provisions (ASCE 2005).¹
3. Select the nominal unit shear strength (**Table 1**) of the bracing method for each braced wall line.
4. Determine the ASD unit shear strength by dividing by a factor of 2.0 and multiplying by the net adjustment factor (**Table 2**).
5. Multiply the allowable unit shear strength by the length of bracing on each braced wall line and verify that the strength provided meets or exceeds the demand.

It should be noted that for continuous structural sheathing, the *2009 IRC* will permit an additional 15 percent strength increase due to the effect of sheathing also placed above and below wall openings as well as on wall segments that are shorter in length than required for intermittent braced wall segments. This single 15 percent increase is a simplification from three previous factors that were dependent upon maximum opening heights in a given wall.

Even with a reasonable consideration of partial restraint effects and whole building effects, simplifying assumptions in the use of the design approach to derive prescriptive *IRC* wind bracing provisions resulted in some inefficiencies. For example, lateral wind loads were determined for both plan directions of a gable roof structure, but because the *IRC* bracing tables were simplified to be independent of wind load orientation the maximum loading direction was used to determine bracing amounts. Also, bracing resistance values were generally rounded down to a single “group” value as shown in **Table 1**. In addition, the net adjustment factor applied to bracing strength was chosen to be generally conservative (**Appendix A**). But, for cases where there is little contribution from or absence of non-structural components or non-compliant bracing elements, the factor may be non-conservative. These types of judgments were necessary to minimize the complexity of prescriptive bracing provisions. The ICC-AHWB Committee was keenly aware of trading precision in the analysis for simplicity in the provisions.

¹ Wind loads were determined using Figure 6-10 of *ASCE 7-05* and were distributed to various braced wall lines on the basis of relative stiffness. Because all of the braced wall lines were assumed to be equally spaced (based on the maximum spacing between the braced wall line under consideration and adjacent braced wall lines) and also assumed to have the minimum required bracing amount, this approach resulted in an equal distribution of the total story shear force to each braced wall line in a given plan direction at a given story level. For design of individual homes, other methods such as tributary area force distribution or rigid diaphragm analysis (including torsional force distribution) may provide more efficient (and accurate) solutions. Also, *ASCE 7* Figure 6-6 may result in a simplified application of lateral wind loads.

Because of these analysis precision trade-offs, an analysis of a specific residential plan using the design procedure outlined herein may result in a more efficient (precise) solution. Alternative means and methods of design are permitted by *IRC* Section R104.11.

Example Calculation

The following example calculation demonstrates how the new *IRC 2009* wall bracing amounts for wind (Table R602.10.1(1)) were determined using the design approach described in this paper:

Site and building conditions:

Wind speed:	90 mph
Wind exposure:	B
Number of stories:	2
Wall height:	10 ft. (baseline for <i>IRC</i> table)
Roof eave-to-ridge height:	10 ft. (baseline for <i>IRC</i> table)
Braced wall line (BWL) spacing:	40 ft.
Bracing method:	2, 3, 4, 6, 7, or 8 (all 700 plf nominal shear strength per Table 1)

Determine bracing for BWL supporting one story and roof as follows:

ASCE 7 wind load on braced wall line: 5,738 lb.

ASD shear strength: $700 \text{ plf} \times 0.5 \times 1.2 = 420 \text{ plf}$

Bracing length required: $5,738 \text{ lb.}/420 \text{ plf} = 14 \text{ ft}$

Therefore, *IRC* Table R602.10.1(1) requires 14 feet of bracing for the above building and site conditions. For a more modest two-story home, wall heights would generally be 8 feet for each story level and the roof eave-to-ridge heights would be more typically 6 feet. For conditions that vary from the baseline conditions used to develop Table R602.10.1(1), bracing amount adjustment factors are provided in footnotes to Table R602.10.1(1). Thus, bracing amounts for the top story of this house geometry are reduced to about 11 feet due to the lesser wind load because there is less wall and roof area subjected to lateral wind pressures. It should be noted that 11 feet of braced wall panels would require the use of three 4-foot braced wall panels on the bottom story end walls (40 feet BWL spacing) which is reasonably consistent with historic practice. Similarly, for one-story home of traditional dimensions, two braces would typically be required by Table R602.10.1(1) which also is consistent with historic practice. For larger homes with greater braced wall line spacing and greater wind speeds and/or exposure, the bracing amounts are significantly increased relative to historic practice or prior *IRC* bracing amounts which were not specifically analyzed to resist wind loads.

Summary and Conclusions

The *IRC* wall bracing provisions for wind resistance have been based on historic practice. The provisions have not changed in 50 years while house designs have changed significantly. Modern home designs are much larger, have

larger and more open rooms, generally have higher wind load demands, and less wall area available to resist the loads. In order to improve the wall bracing provisions to work better with modern construction practices and designs, it became apparent that a logical and consistent basis was needed.

After more than two years of work, agreement was reached to establish an analysis procedure to evaluate *IRC* wall bracing amounts. The analysis procedure provided a logical framework for modernizing the *IRC* wind bracing provisions. The analysis procedure is similar to typical engineering design of segmented shear walls. Considerable effort, however, was spent developing an appropriate means of accounting for partial overturning restraint and whole-building effects which govern the performance of wall bracing in conventional woodframe homes. As a result, the new *2009 IRC* wind bracing provisions align with past successful wall bracing practice while also correcting deficiencies in their use on larger modern homes. Conventional wood-frame housing and *IRC* wall bracing provisions now have a consistent and logical framework to ensure wall bracing capacity meets wind load demand.

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Appendix A –Whole Building Tests

A total of five whole building tests of conventionally framed and braced buildings were studied to calibrate the design approach to capture whole building performance (Thurston 2003, Paevere 2002, HUD 2001, Kasal et al. 2004, Fischer et al. 2001, Dorey and Schriever 1957, Reardon and Mahendran 1988). The results for three of these tests in comparison to the simplified bracing design method

are shown in **Table A1** (Thurston 2003, Paevere 2002, HUD 2001, Kasal et al. 2004, Fischer et al. 2001). Construction details for the buildings are summarized in footnote 1 of **Table A1**. Tested buildings are further illustrated in **Figures A1 through A3**.



Figure A1.—BRANZ whole house test (Thurston 2003).

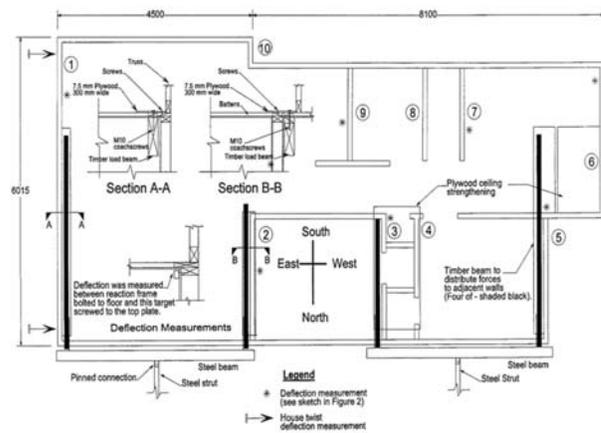
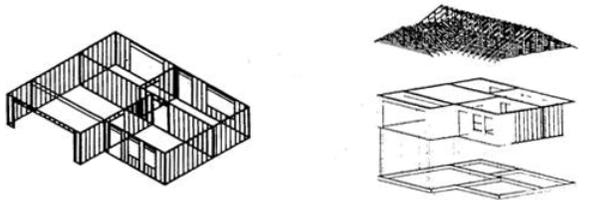


Table A1.—Conventionally framed whole building tests.

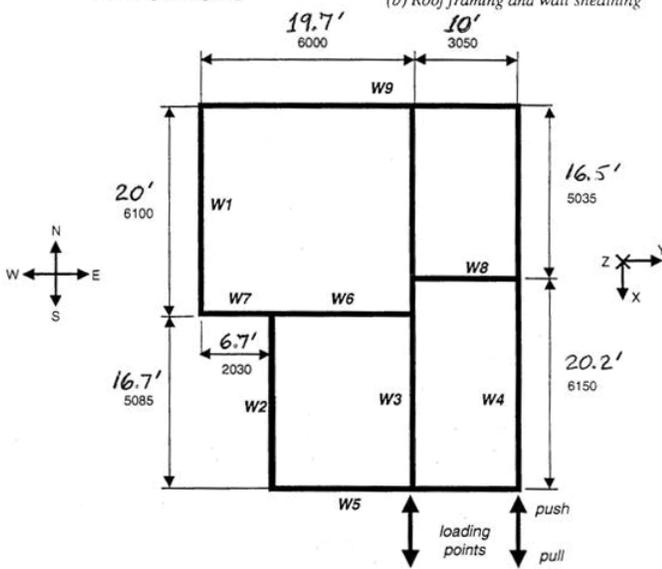
Whole house test program ¹	Tested story shear resistance		Bracing method prediction ²	Ratio of tested/prediction	
	Whole structure	Interior partitions discounted		Whole structure	Interior partitions discounted
BRANZ (Thurston 2003)	23,600 lb.	17,265 lb.	7,673 lb.	3.1	2.3
CSIRO (Paevere 2002, HUD 2001, Kasal et al. 2004)	25,000 lb.	16,526 lb.	10,971 lb.	2.3	1.5
CUREE/FEMA (Fischer et al. 2001)	N/A	> 22,700 lb.	14,118 lb.	N/A	1.6

- The following summarizes the construction of each of the test buildings. None of the buildings were engineered structures (e.g., all followed conventional framing practices).
 - BRANZ test building was a complete whole building (20 by 40 ft. single story home) including all finishes and components. Its bracing included only intermittent 3/8 in.-thick gypsum wall board panels with no sheathing or bracing materials on the exterior face of the wall which included only single-nailed lap siding.
 - CSIRO test building was a partially completed whole building (30 by 37 ft. L-shaped single story home) including no exterior finishes, doors, or windows. Exterior walls in the loading direction included continuous 3/8 in. wood structural panel sheathing on the exterior and 1/2 in. gypsum wall board on the interior, both minimally fastened. There was one interior partition wall in the loaded direction. The front side of the building had only one IRC code-compliant braced wall panel (2.7 ft. wide by 8 ft. tall).
 - CUREE/FEMA test building was a partially completed whole building (16 by 20 ft. small two story home) including no exterior finishes, interior finishes, doors, or windows. Exterior walls in the loading direction were continuous 7/16 in. OSB sheathing with standard fastening practice (8d common nails at 6 in. o.c. on edges and 12 in. o.c. in field). There were no interior finishes on exterior walls or one short interior wall. The building was not tested to failure load.
- The analysis prediction was determined as follows: (actual length of wall bracing in loading direction) × (nominal unit shear value for bracing material) × (partial restraint factor, Table 2). An ASD safety factor was not applied to the nominal unit shear values for the bracing material because the prediction of ultimate capacity to maximum tested load was the goal.



(a) Wall framing only

(b) Roof framing and wall sheathing



(c) Floor plan and wall numbering

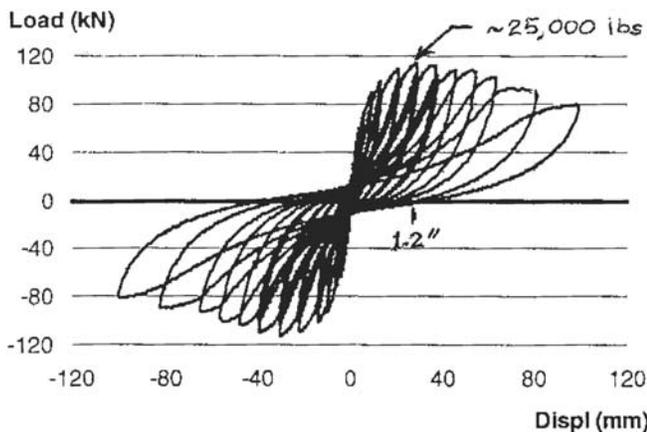
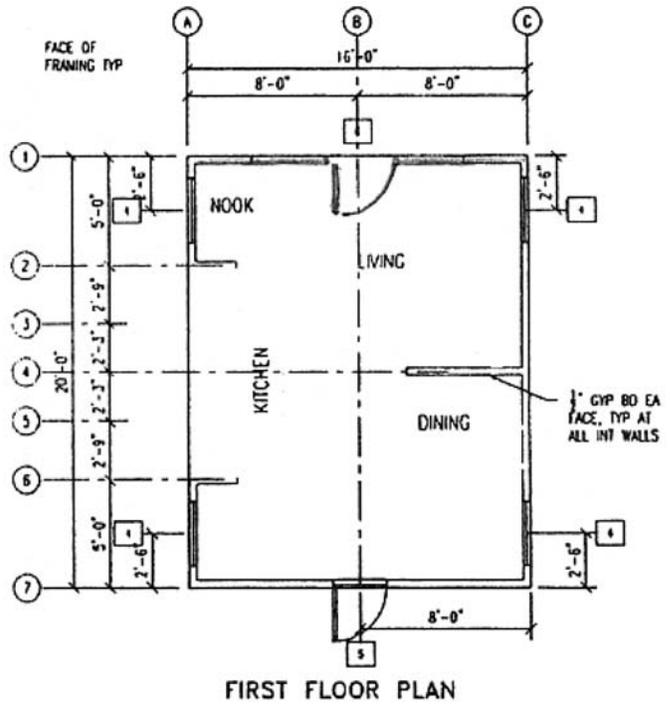
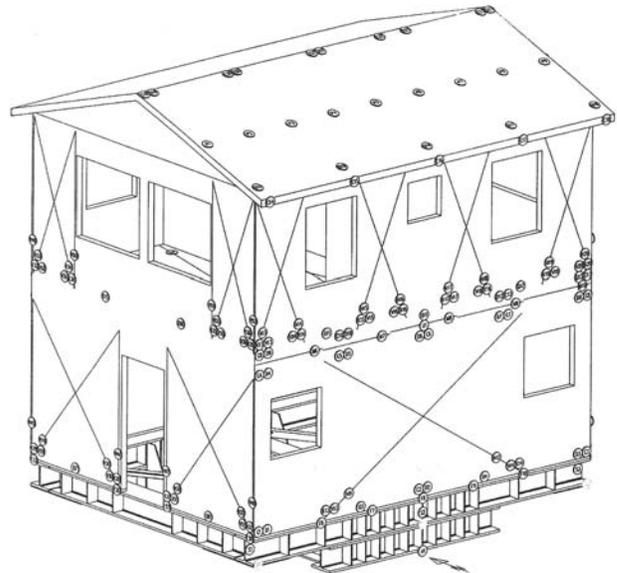


Figure A2.—CSIRO whole house test (Paevere 2002, HUD 2001, Kasal et al. 2004).



FIRST FLOOR PLAN

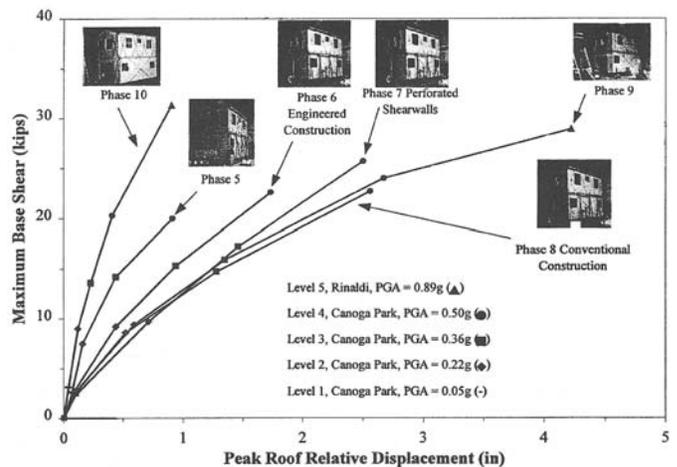


Figure A3.—FEMA/CUREE whole house test – Phase 8 (Fischer et al. 2001).

Table A1 clearly shows that whole buildings have enhanced performance relative to estimates from the simplified bracing design approach. Even when discounting the whole building test results by removing the contribution of interior partition walls (when present) and considering that two of the homes were not complete whole buildings, the minimum ratio of tested/predicted was 1.5. The “weakest” built home (BRANZ house) had the lowest predicted strength as expected, but the largest observed ratio of 3.1 or 2.3 when the contribution of interior partition walls (not counted as bracing) were discounted from the tested strength of the building. The contribution of interior partition walls were “backed-out” of the test results for the purpose of determining a whole building factor independent of the amount of interior partitions that might (or might not) be present in any given home and which are not regulated with respect to ensuring adequate wall bracing.

In addition, the AHWB committees considered several whole building tests of conventional residential construction that included simulation of combined wind uplift and lateral forces (Dorey and Schriever 1957, Reardon and Mahendran 1988). In one test conducted in Canada (Dorey and Schriever 1957), a 120 mph wind load was applied resulting in an approximate 4,400 lb. story shear force together with a 180 plf uplift force reaction at the top of the

windward wall. The only observed damage was cracking in a gypsum joint at the ceiling-wall in a corner of the building and the maximum braced wall line drift was 1/8 inch. The building was braced with 1 by 4 wood let-in braces and 3/8-inch gypsum wall board interior finish. There was no exterior sheathing and aluminum siding was used. A similar test was conducted in Australia (Reardon and Mahendran 1988) with 24 inch on center wall framing, 3/8-inch gypsum wall board interior finish, no exterior sheathing, 1 by 2 let-in braces (not traditional 1 by 4 wood let-ins), and brick veneer siding. This house was tested to a 120 mph open terrain wind load with simultaneous lateral and uplift loading. The only observed damage was a failure of the roof overhang which precipitated a 1/3-inch upward movement of one end of the garage door header. Braced wall line deflections were only 1/10 inch at maximum load. Together, these tests gave some assurance that in lower wind regions conventional construction can perform adequately even under conditions of combined wind uplift and lateral loading.

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