

# The Story Behind *IRC* Wall Bracing Provisions

Jay H. Crandell, P.E.

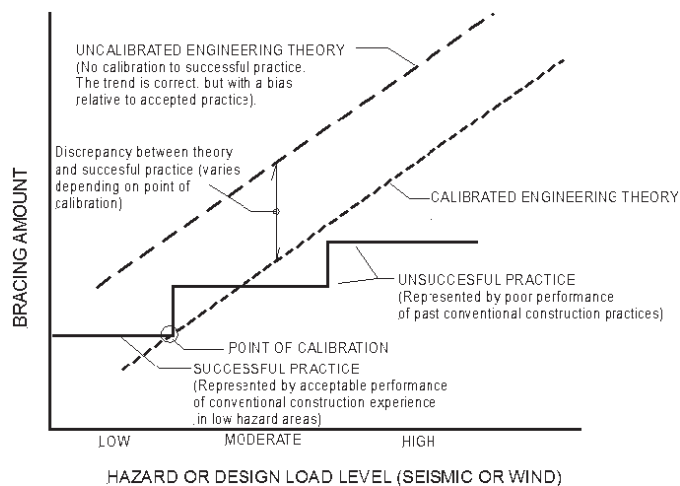
## Introduction

The conventional woodframe construction provisions of the 2006 *International Residential Code (IRC)* (ICC 2006a) may be viewed as a “melting pot” of engineering science and building art. A previous article addressed many challenges in reconciling conventional construction practices with modern methods of engineering practice (Crandell and Kochkin 2003).

In this article, the derivation of the *IRC* wall bracing provisions is documented and discussed in view of the technical research, engineering analyses, and judgments they embody. The art and science behind accurately understanding conventional wall bracing is still considered to be in its infancy and subject to disparate interpretations, even though it has been studied at various times since the early 1900s and especially in recent years (HUD 2000, HUD 2001a, Ni and Karacabeyli 2002, Paevere 2002, Crandell and Kochkin 2003, Cobeen et al. 2004, Kasal et al. 2004, Crandell 2006, APA 2007, Simpson 2007).

## Background

Any complex set of interrelated engineering design rules (including material strength properties, safety factors, resistance factors, load factors, mapped hazards or design loads, empirical equations, and equations based on principles of mechanics, etc.) is “tuned” to produce a design solution that is considered to represent or effectively calibrate to successful design or construction practice. Engineering is fundamentally an empirical science and, as such, the result of applying any engineering theory must produce answers that agree with experience. The process of calibrating or “tuning” design theory to agree with successful experience, hypothetically applied to conventional wall bracing, is illustrated in **Figure 1**. This tuning process may account for flaws or biases in codified theory for determining loads and/or resistance for a specific application, such as light-frame house construction. Because the ultimate goal is to achieve “acceptable” performance in a manner that is risk-consistent across all hazard conditions and applications, this process inevitably involves judgment, science, and politics. This observation is especially true in the development of the *IRC* wall bracing provisions, particularly the derivation of wall bracing amounts in Table R602.10.1 of the *IRC*.



**Figure 1.**—Illustration of calibration of engineering theory to successful conventional construction practice.

Unfortunately, it should be recognized that current building codes and engineering design standards do not provide analytical procedures suitable to evaluation of conventional bracing methods and building systems. For example, there are no codified design values or equations to predict the structural resistance of conventional (or “partially restrained”) wall bracing systems such as wood let-in braces, wood structural panels, Portland cement stucco, and other bracing methods recognized in the *IRC*. Codified engineering analysis conventions and data that are available only apply to detailing conditions, such as the use of hold-down anchors, that are not representative of conventional construction and the complex load-paths involved (Crandell and Kochkin 2003). The degree of error or conservative bias in attempting to apply accepted engineering conventions to the analysis of typical light-frame buildings or homes can be substantial (refer to Addendum A to this article which compares three different attempts to reconcile codified engineering analyses to results from an actual whole building test).

Bracing amounts in *IRC* Table R602.10.1 for various conventional bracing methods represent one means of addressing the concerns above in a rational manner using judgment

and the best available technical information during the drafting phase of the *IRC* in the late 1990s.<sup>1</sup>

### Setting the Stage for the *IRC*

Leading up to the drafting of the *IRC* (2000 edition), various bracing provisions for conventional construction were in use (CABO 1995, ICBO 1997). With increased concern for performance of conventional wall bracing in high hazard regions of the country and in the context of modern construction trends (e.g., larger buildings, more open interior spaces, more windows, and increasing use of narrower bracing panels not complying with the traditional 4-ft. braced panel width, etc.), these provisions were coming under increasing scrutiny and pressure to conform with engineering practice. Methods for engineering design, however, did not agree with a general perception of successful use of conventional construction practices, particularly in low hazard regions. A solution was needed to reconcile these competing truisms.

Fortunately, the American Forest & Paper Association/American Wood Council (AF&PA/AWC) staff had discovered a shear wall analysis method developed in Japan some 20 years prior (Line and Douglas 1996, Line 2002). The method is known as the perforated shear wall (PSW) design method. While still requiring a “non-conventional” load path (i.e., a hold-down bracket) at the end of a PSW, the method provided a means of evaluating an otherwise conventionally framed braced wall system with varying amounts and sizes of wall openings (including full-height openings representing spaces between intermittent conventional brace panels). This method is currently recognized with a number of seemingly conservative constraints (relative to tested or actual performance) in *Special Design Provisions for Wind and Seismic* (AF&PA 2005). It is featured in the design examples and whole building test comparison in Addendum A to this article.

Shortly after the PSW design method was discovered, the homebuilding industry, along with the U.S. Department of Housing and Urban Development (HUD), became interested in its application as part of a multi-year research program at the National Association of Home Builders (NAHB) Research Center, Inc. ([www.nahbrc.org](http://www.nahbrc.org)). The research initiative was aimed at developing engineering methods to more efficiently design conventional woodframe homes. One of the many objectives of this research program was to

develop a lateral design method for wood frame homes that could be used to answer a growing call to justify and improve conventional wall bracing provisions in the model building codes. This research initiative occurred at a time leading up to unification of the three major national model building code organizations as the International Code Council (ICC) and the drafting of the *IRC* (2000 edition).

Thus, work began immediately to extend the PSW design method to applications without requiring hold-down anchors at the ends of braced wall lines. The goal of this effort was to morph the design method into one which could be used to design the lateral resistance of a truly conventional woodframe building. As a result, testing of three-dimensional wall assemblies with corners restraining the ends of PSW-braced wall lines was conducted. Additionally, tests were run comparing the difference in performance of PSW-braced wall lines with:

- variations of base connections (e.g., nails vs. bolts),
- various sizes of openings and widths of bracing segments (e.g., 4:1 segments), and
- incremental enhancements to framing (e.g., truss plate or strap reinforcements at “weak links”).

In addition, a whole building test was conducted (Paevere 2002) to verify performance and investigate accurate means of distributing horizontal forces to conventional braced wall lines through a conventional (unblocked) diaphragm without a clearly defined chord member along the eaves (e.g., no fascia board or perimeter nailing of the roof diaphragm). Most of these studies were reported in an earlier article (Crandell and Kochkin 2003). Many of these studies occurred after derivation of the *IRC* bracing amounts in Table R602.10.1, but in general they have served to confirm what was done rather than identify glaring deficiencies (although this should not be taken to mean there are no deficiencies or opportunities for improvement).

### Drafting of *IRC* Wall Bracing Provisions

In the late 1990s, a drafting committee was assembled by ICC to prepare a draft of the *IRC*. This committee included members of the building community, code official community, and general interests. Early in this process, NAHB sponsored development of a revised set of bracing provisions based on some of the earliest results of the research mentioned previously.

At one point, a set of provisions had been developed and were tentatively approved by the committee. These provisions included seismic bracing amounts (similar to current Table R602.10.1 in the *IRC*) plus a separate table addressing wind bracing.

In addition, these provisions introduced the concept of continuous structural sheathing (current section R602.10.5 of the *IRC*) and a number of other coordinated features to enable efficient application of the provisions to conditions that were considered problematic in modern conventional construction. Furthermore, this early draft increased brac-

<sup>1</sup> In recent years, several advancements have continued to occur in understanding the performance of conventionally braced wood-frame buildings. While these are not addressed in this article, it is worth mentioning a generalized mechanics-based model under development by AF&PA's American Wood Council (AWC) staff that provides a means to design wall bracing using detailing and load paths inherent to conventional construction as well as engineered construction. Ultimately, such a tool may allow conventional construction to be considered as an engineered system on par with perforated shear walls and segmented shear walls, each with their respective trade-offs in performance vs. detailing and construction efficiency.

ing amounts in high seismic areas relative to past practice (also allowing only fully sheathed or continuous-sheathed wall systems in high seismic regions) while maintaining previous bracing methods and amounts used for many years in lower seismic hazard regions.

The wind bracing requirements introduced significant improvements, addressing perhaps the major deficiency in past conventional bracing practices relative to their application to modern homes. These changes required significant concessions by interests represented on the *IRC* drafting committee. While imperfect, they represented a notable improvement to wall bracing requirements for conventional construction as a whole.

Later in the *IRC* drafting process and as the process continued to attract broader public input, concerns were effectively voiced that the draft bracing provisions did not strictly conform to recommended seismic provisions of the National Earthquake Hazard Reduction Program (NEHRP). In the end, the conventional bracing provisions for the *IRC* were redrafted to conform more closely to the NEHRP provisions and only a few features from earlier draft bracing provisions were retained. Features retained from the earlier draft included seismic bracing amounts (now Table R602.10.1) and the continuous structural sheathing approach (now Section R602.10.5).

Unfortunately, the wind bracing table in the earlier draft of *IRC* bracing provisions was removed from consideration and wind speed limits were more or less arbitrarily added to the seismic bracing table (Table R602.10.1). In addition, all conventional bracing methods (except Method 1 let-in bracing) were recognized as in the past for use in high seismic regions. While these *IRC* drafting decisions may have been considered advantageous for seismic performance in some regards and questionable in others (Crandell and Kochkin 2003), they clearly resulted in wind bracing requirements in the *IRC* that are dubious and potentially unsafe, especially for large multi-story homes.<sup>2</sup>

### Derivation of the Continuous Structural Sheathing Method

The continuous structural sheathing bracing method (or “R602.10.5 Method”) was conceived and developed, not to increase the strength of homes, but to provide equivalent bracing performance with less bracing. Thus, it was intended to address a problem in modern housing construction where increased use of windows and doors of larger sizes created difficulties in providing adequate space for

traditional bracing methods (e.g., let-in braces or 4-ft. brace panels). It was also intended to be used with other code-compliant bracing methods on other braced wall lines provided that:

1. any wall line using the R602.10.5 method had a minimum 2-ft. sheathed corner return at each end per *IRC* Figure R602.10.5 and
2. other braced wall lines in the same building were compliant with provisions applicable to the bracing method used.<sup>3</sup>

The continuous sheathed bracing method in the *IRC* is based on the PSW design method with some notable differences relative to its current codified form. These are:

- *PSW Shear Reduction Factor (F)* – This factor is the foundation of the PSW design method (in its empirical form), and it accounts for the reduction in shear capacity of a perforated shear wall relative to the same wall without the presence of openings (solidly sheathed throughout). The empirical form of the shear reduction factor used to develop  $C_o$  factors in the *Wind and Seismic* standard is  $F = r/(3 - 2r)$  where “ $r$ ” is a parameter based on geometric properties of a wall line that relate to bracing strength and stiffness (e.g.,  $r = 1$  represents a shear wall without perforations whereas  $r = 0.2$  represents a shear wall with a substantial number of openings and little structurally sheathed surface area).

However, various tests of perforated shear walls reported in Crandell and Kochkin (2003) demonstrate that a more accurate (less conservative) prediction is achieved on average by the form  $F = r/(2 - r)$ . This form was used in developing the *IRC* bracing amounts for intermittently spaced wood structural panels in accordance with Method 3 (wood structural panel sheathing, per *IRC* Section R602.10.3) and continuous wood structural panel sheathing (*IRC* Section R602.10.5).

The only analytical difference between these two bracing methods is in the treatment of opening sizes. For Method 3, wall areas between intermittent brace panels are assumed to be openings that occupy 100 percent of the wall height. These wall portions between braces were conservatively assumed to provide no shear resistance (i.e., act like a bare frame without gypsum finishes or other components that are known to contribute to the lateral resistance of light-frame construction).

For continuous structural sheathing, the size of openings may vary from typical window clear opening height (e.g., 67% of the wall height) to that for a door (e.g., 85% of the wall height) and sheathing is applied continuously between and around wall openings. These assumptions permitted the use of the PSW design method to back-calculate a length of bracing required for walls with maximum opening heights corresponding to these conditions. Thus, the bracing adjustment factors of 0.9 and 0.8 in Section R602.10.5 reflect the effect of limited

<sup>2</sup> Work is currently underway within the ICC Ad Hoc Committee on Wall Bracing to address the concern with wind bracing amounts in the *IRC* ([www.iccsafe.org/cs/cc/ahc-wb/index.html](http://www.iccsafe.org/cs/cc/ahc-wb/index.html)).

<sup>3</sup> This original intention has been restored and clarified in the recent ICC code development cycle for the *2007 IRC Supplement* and the future *IRC 2009* edition. Similar action has already been taken in a number of states by appropriately amending Section R602.10.5 of the 2003 or 2006 *IRC*.



opening heights on the performance of a perforated shear wall relative to a perforated shear wall with opening heights of 100% (i.e., bracing Method 3 in the *IRC*).

- *Overturning Restraint at the Ends of a PSW* – In the *Wind and Seismic* requirements for PSWs, a hold-down bracket is required for full restraint at each end of the wall line. However, testing (Crandell and Kochkin 2003) clearly demonstrated that substantial capacity of a perforated shear wall could be maintained by use of corners to “partially restrain” the ends of the wall line. To account for the effect of using corners as overturning restraints, the unit shear value for Method 3 and R602.10.5 bracing in the *IRC* was based on tests of a 12-ft.-long wall with a minimum 2-ft. corner return (Dolan and Heine 1997). This resulted in a unit shear capacity of 634 plf being used as the basis for evaluating Method 3 and R602.10.5 bracing in the *IRC* with the PSW design method.<sup>4</sup> In more recent studies, better methods of analyzing the effect of “partial” end restraint on the shear capacity of perforated and segmented shear walls have been developed and are summarized elsewhere (HUD 2001b).
- *Uplift Restraint along the Bottom Plate of a PSW* – In the *Wind and Seismic* version of the PSW design method, the bottom plate of the wall must be anchored against a uniform vertical uplift force equivalent to the maximum horizontal unit shear force in the wall line. But, the previously mentioned testing included conventional bottom plate connections (e.g., two 16d pneumatic nails at 24 in. oc and anchor bolts at 6 in. oc). These connections are not consistent with the *Wind and Seismic* uplift anchorage requirement, yet the tested PSWs still performed as predicted with uplift restraint equivalent to roughly 50 percent of the unit shear strength of the wall system. Because the *Wind and Seismic* provisions are conservative and came after the *IRC* bracing provisions, the *IRC* relies on the uplift restraint provided by conventional bottom plate connections.
- *Aspect Ratio of PSW Segments* – In the *Wind and Seismic* version of the PSW design method, the aspect ratio of PSW segments is limited to 2:1 (height:length). A greater aspect ratio is permitted (up to 3-1/2:1) with a reduction in the nominal unit shear capacity used to determine PSW capacity. This reduction applies even if only one segment has an aspect ratio greater than 2:1 when it is included in the determination of total length

of shear wall segments in a PSW. In the *IRC*, PSW segments are permitted to be as narrow as 4:1 (with limits on the height of openings adjacent to the PSW segment) with no penalty. This difference is based on testing (Crandell and Kochkin 2003).

### Derivation of the Minimum Bracing Amounts (*IRC* Table R602.10.1)

The following summarizes key features of the analysis used to derive the seismic bracing amounts in *IRC* Table R602.10.1. These analysis features are employed in example calculations in the next section. As mentioned previously, the *IRC* bracing amounts in Table R602.10.1 were not originally intended to address bracing amounts for lateral wind forces (a separate wind bracing table was originally proposed).

- Allowable Stress Design (ASD) Safety Factor = 2.0

*Note: This safety factor is taken to be representative of traditional design practice and is applied to the average ultimate (nominal) tested shear wall strength.*

- Deflection Limit = unspecified (see note below)

*Note: Deflection limits were not specifically evaluated. Appropriate deflection criteria, particularly when applied to small deflections, are very sensitive to variations in boundary conditions represented in any particular index test as well as wall segment aspect ratio and actual end-use boundary conditions imposed by a real building system. However, in comparison to test data serving as the basis for the continuous sheathed method, an approximate 0.5 to 0.7 percent drift limit at ASD load (50% of ultimate load) may be considered as representative. This range is reasonably consistent with historic deflection criteria for light-frame wall bracing which permitted up to 0.65 percent drift at 50 percent of ultimate capacity (FHA 1949) or as much as 1 percent drift at approximately 90 percent of ultimate capacity (NBS 1948).*

- Nominal Unit Shear Wall Value = 634 plf

*Note: See previous section for the basis of this value and the note with R-factor below. This value assumes the presence of 1/2-in. gypsum wall board (GWB) on the interior face of the braced wall. Because this value is based on sequential phased displacement (SPD) cyclic testing, however, the necessity of requiring GWB in actual practice for wind or seismic loading is uncertain. In cyclic testing, GWB tends to contribute little to the peak strength of the wall assembly and, therefore, the peak resistance would be similar with or without GWB; however, GWB does contribute to energy dissipation during a seismic event.*

*Also, this value represents the nominal shear capacity of a partially restrained shear wall assembly with a minimum 2-ft. corner return at the ends. Tested 12-ft.-long shear walls with corner returns provide about 90 percent of the unit shear strength of fully restrained shear wall segments*

<sup>4</sup> These tests included 1/2-in. gypsum wall board on the interior fastened at 7 in./10 in. oc (edge/field) using 5d cooler nails and 7/16-in.-thick oriented strandboard on the exterior fastened with 8d common nails at 6 in./12 in. oc with Spruce-Pine-Fir framing at 16 in. oc. The tests also used the sequential phased displacement cyclic test protocol which resulted in a limited contribution from gypsum panels at peak load. Response of the corner-restrained walls was noted as being very ductile.

with hold-downs at each end (Dolan and Heine 1997). As a basis for design, this corner-restrained unit shear strength has been found to produce conservative predictions relative to actual performance in whole building tests (Paevere 2002, Kasal et al. 2004, Simpson 2007). This value is also reasonably consistent with nominal shear strength of other IRC bracing methods as reported in numerous sources (Crandell 2006).

- Seismic Response Modifier (R):  $R = 5.5$  for wood structural panel bracing (Method 3 and R602.10.5) and  $R \sim 4$  for other bracing methods (both are based on ICBO 1997)

*Note: A seismic response modifier (R-factor) of 5.5 was originally applied to all bracing methods, but a lesser unit shear resistance value (by a factor of about 0.7) was used for bracing methods other than Method 3 or R602.10.5 (wood structural panel methods). Subsequent review of relevant test data for IRC bracing methods has shown that the shear strength of other bracing methods are all similar or greater than Method 3 or R602.10.5 due to fastening requirements unique to each method (Crandell 2006). Therefore, the 634 plf nominal unit shear value can be taken to represent the design shear resistance value for all IRC bracing methods (except Method 1 let-in bracing).*

Also, an R-factor of about 4.0 (slightly less than  $R = 4.5$  per the UBC-97) should be taken to represent the seismic response of bracing methods other than Method 3 or R602.10.5 bracing in the IRC. Thus, the IRC bracing amounts for Method 3 (wood structural panels) and other bracing methods are different in proportion to the difference in R-factors (e.g.,  $5.5/4 \sim 1.4$  so roughly 40% more bracing is required in the IRC for bracing methods other than Method 3).

It should be noted that current code provisions for engineered light-frame shear walls call for an R-factor of 6.5 for wood structural panel bracing and an R-factor of 2.0 for other bracing methods (ASCE 2005, ICC 2006b). The latter value may be considered as a generally conservative “one-size-fits-none” value in terms of the variation in response characteristics of the various bracing methods and materials it purports to address.

- Seismic Base Shear:  $V = [1.2 S_{DS}/R] \times [W] \times [1/1.4]$

*Note: This equation is in accordance with the simplified method included in the IBC 2000 (refer to the IBC 2000 for parameter definitions). The 1/1.4 factor provides a conversion from strength-based seismic force level to ASD seismic force level. The 1.2 factor conservatively accounts for the use of a tributary mass vertical force distribution rather than a lumped-mass or “pendulum effect” approach of vertical force distribution used in the equivalent lateral force procedure.*

- Vertical Distribution of Base Shear: Story shear forces based on tributary mass at a given story level

*Note: This vertical seismic force distribution approach is consistent with the use of the simplified seismic base shear equation. It should be noted that one-half of the partition wall weight at a given story level under consideration was not included in the total supported seismic weight attributed to that story because partition walls were considered to contribute at least enough lateral resistance to the story under consideration to offset the weight they contributed to story shear forces under consideration. The full weight of partitions in stories above the story level under consideration was included and no resistance or seismic energy dissipation provided by those partition walls was accounted for directly.*

- Horizontal Force Distribution: Story shear forces were distributed on the basis of tributary area except when more than two braced wall lines exist in a given plan direction. Then, the forces are considered to be distributed based on an assumption of equivalent relative stiffness (i.e., total story shear force is distributed equally to each wall line).

*Note: The original bracing analysis was based on a 25 ft. by 25 ft. building module resulting in the bracing amounts determined for a 12.5 ft. tributary span between exterior braced wall lines. But, for a 25 ft. by 50 ft. building with three braced wall lines perpendicular to the long direction, the tributary braced wall line spacing is 17 ft. (assuming equal stiffness force distribution to three equally spaced wall lines). It is noted that under a strict tributary area force distribution method the exterior wall lines would experience 12.5 ft. tributary spacing and the interior wall line a 25 ft. tributary spacing (or twice that assumed in the analysis).*

With the equal stiffness force distribution assumption, the interior wall's tributary portion of the building length is 17 ft. which is greater than assumed by a factor of  $17/12.5 = 1.36$ . Thus, the IRC bracing provisions effectively rely on a design adjustment factor that may be attributed to system effects not considered in the analysis. The factor may be considered consistent with the past practice of using a 1.33 wind/seismic allowable stress adjustment factor.

- Baseline Building Geometry: Braced wall lines spaced at 25 ft. oc, stud wall heights at 10 ft., and roof eave overhang at a nominal 1 ft.

*Note: In the lower seismic design categories (A–C), the baseline braced wall line spacing in the IRC 2006 is 35 ft. oc even though the bracing amounts were originally evaluated assuming a 25 ft. oc braced wall line spacing as described above. In the lower seismic hazard conditions, this situation created an additional non-conservative bias from a purely analytical standpoint, but the resulting bracing amounts were considered to be consistent with*

historically (and politically) accepted norms for conventional construction.

- Baseline Dead Loads (seismic weights):
  - Roof/ceiling (12 psf)
  - Walls (8 psf)
  - Interior partitions (6 psf, floor area)
  - Floors (10 psf)

*Note: The 8 psf wall and 12 psf roof dead loads are not consistent with the 15 psf maximum dead load permitted in the IRC. This difference is attributed to a failure to coordinate analysis assumptions with scope limits introduced to the IRC draft at a later time. However, this non-conservative difference may be considered to be offset by conservative omissions in the IRC (e.g., failure to fully consider system effects caused by both structural and non-structural components in a light-frame buildings' lateral force resisting system).*

*Also, no snow load was included in the seismic analysis, so the ground snow load scope limit for the IRC conventional woodframe bracing provisions is theoretically 40 psf based on the criteria that only 20 percent of the roof snow load be considered when roof snow loads exceed 30 psf. However, this scope limitation was not defined in the IRC.*

*Finally, the 6 psf partition load (based on floor area) represents a maximum value from a review of a limited number of representative modern house plans (HUDa 2001). Actual partition wall loads varied by story level, but rarely exceeded 5 psf (floor area basis).*

It should be noted that Method 1 (wood 1 by 4 let-in) bracing was not specifically analyzed and remains in the IRC on the basis of successful practice with special limitations on use. This bracing method in combination with horizontal board sheathing, served as the basis of minimum performance criteria for wall bracing in early evaluation criteria (FHA 1949). Subsequent review of test data has shown that a Method 1 brace can provide peak shear resistance (for a 45° angle application and 16 in. oc stud spacing) similar to a 4-ft. Method 3 wood structural panel brace (Crandell 2006). Today, its application in the IRC is intended to be used in combination with a 1/2-in. GWB finish on the interior face of the wall, not horizontal board sheathing on the exterior face of a wall.

### Example Calculations

Using the approach described above, Examples 1 and 2 demonstrate how seismic bracing amounts in Table R602.10.1 of the IRC were derived.

### Discussion

According to the rational analysis and judgments employed, the IRC seismic bracing amounts turned out to be similar to historically accepted practice for minimum bracing amounts required in low seismic hazard areas. However, the same analysis method and judgments demon-

strated the need to increase bracing amounts in higher seismic hazard areas. For example, in Seismic Design Category D2 (corresponding roughly to legacy code Seismic Zone 4), bracing amounts for two-story construction increased from 25 percent (CABO 1995, ICBO 1997) to 55 to 75 percent in the IRC – a two- to three-fold increase over past minimums.

The analysis approach and judgment employed, despite any specific deficiency, provided results that were consistent with accepted norms and experience in lower hazard conditions and provided a dramatic increase in bracing requirements in higher seismic hazard areas. This outcome is consistent with the fundamental need to calibrate an engineering analysis approach to successful experience (refer to Background and Fig. 1). This approach also had political appeal because it provided a reconciliatory position between existing engineering provisions and conventional construction practices.

The acceptability of the IRC bracing amounts is also supported by the results of a scientific damage assessment of conventional woodframe homes following the Northridge Earthquake (HUD 1994, HUD 1999). The key finding here is that there were no identified collapses of buildings that at least met the IRC wall bracing provisions. For example, the minimum bracing amount for single-story Portland cement stucco clad homes per the IRC bracing provisions is 40 percent of the total length of each braced wall line. For homes with this amount of bracing, the probability of experiencing cracking in stucco exterior and gypsum panel interior finishes was about 50 percent in the Northridge Earthquake. For a roughly 300- to 500-year return period event experienced by the sampled housing stock, this level of performance (damage probability) seems acceptable and further increases in bracing amounts would be faced with severely diminished returns (Crandell and Kochkin 2003). Also, if the relatively few conventional woodframe buildings that collapsed or experienced significant structural damage (e.g., less than 2% of the single-family housing stock and often with extenuating circumstances) had been constructed in accordance with the modernized IRC wall bracing provisions, these cases of severe damage would have been significantly lessened and the loss of life (16 in one apartment building alone) could have been avoided.

Finally, the following statements from FEMA-232 give additional confirmation that the IRC bracing provisions for conventional woodframe construction provide adequate performance (FEMA 2006):

*“The reason one- and two-family houses tend to perform adequately in earthquakes even when designed to minimum code forces are because houses often are stronger than recognized in code-level design...In residential construction, the finish materials and nonstructural partitions often add significantly to the strength provided by required bracing materials...” [p. 11]*



### Example 1 – 35 ft. by 70 ft. Two-Story Home in Seismic Design Category C

**Objective:** Evaluate *IRC* wall bracing amount for Method 3 (wood structural panel) bracing for transverse seismic story shear force on the bottom story. Transverse story shear force is shared by three braced wall lines spaced at 35 ft. oc. The home is assumed to have 10-ft.-high walls on both stories and nominal 1-ft. roof overhangs at eaves only.

- *IRC* Requirements

Seismic Design Category = C

Bracing Amount Required (Table R602.10.1) = 30%<sup>5</sup>

Length of Transverse Braced Wall Lines (BWL) = 35 ft.

Length of Bracing Required for Each Transverse BWL<sup>5</sup> = (35 ft.)(30%) = 10.5 ft.

- Tributary Weight on Bottom Story BWLs

Roof ( $W_r$ ) = (70 ft.)(35 ft. + 2 ft.)(12 psf) = 31,080 lb.

Snow ( $W_s$ ) = 0 (max. ground snow load less than 40 psf)

Floor ( $W_f$ ) = (70 ft.)(35 ft.)(10 psf) = 24,500 lb.

Int. Walls/Partitions ( $W_p$ ) = (70 ft.)(35 ft.)(6 psf) = 14,700 lb. (upper story only)

Ext. Walls ( $W_w$ ) = [10 ft. + 1/2(10 ft.)][2(35 ft. + 70 ft.)](8 psf) = 25,200 lb.

TOTAL ( $W$ ) =  $W_r + W_s + W_f + W_p + W_w$  = 95,480 lb.

- ASD Story Shear Force

$V = [1.2 S_{DS}/R] \times [W] \times [1/1.4] = 1.2 [(0.5 g)/5.5](95,480 \text{ lb.})(1/1.4) = 7,447 \text{ lb.}$

- ASD Shear Force (per each of three transverse BWLs uniformly spaced at 35 ft. oc)

$V_{BWL} = 1/3(7,447 \text{ lb.}) = 2,482 \text{ lb.}$

- Braced Wall Line (BWL) ASD Shear Strength

$V_{\text{Allowable}} = (v)(L)(F_{PSW})$

$v = 634 \text{ plf}/2 = 317 \text{ plf}$  (nominal ultimate unit shear strength divided by 2.0 safety factor)

$L = 35 \text{ ft.}$  (length of BWL)

$F_{PSW} = r/(2 - r)$  (empirical PSW shear strength reduction factor)

$r = 1/[1 + \alpha/\beta]$

$\alpha = \Sigma(A_o)/[H \times L]$

$\Sigma(A_o) = \text{wall opening area}^6 = [35 \text{ ft.} - 10.5 \text{ ft.}](10 \text{ ft.}) = 245 \text{ ft}^2$

$\alpha = 245 \text{ ft}^2/[(10 \text{ ft.})(35 \text{ ft.})] = 0.7$

$\beta = \Sigma(L_i)/L$

$\Sigma(L_i) = \text{sum of braced wall panel segments} = 10.5 \text{ ft.}$

$\beta = 10.5 \text{ ft.}/35 \text{ ft.} = 0.3$

$r = 1/[1 + 0.7/0.3] = 0.3$

$F_{PSW} = 0.3/(2 - 0.3) = 0.176$

$V_{\text{Allowable}} = (317 \text{ plf})(35 \text{ ft.})(0.176) = 1,953 \text{ lb.}$

$V'_{\text{Allowable}} = 1,953 \text{ lb.} (1.3) = 2,539 \text{ lb.} > 2,482 \text{ lb., OK}^7$

<sup>5</sup> The actual bracing amount also must comply with maximum 25 ft. oc spacing of braced wall panels which, for some braced wall lines, may result in a greater amount of bracing than required by the minimum percent of bracing.

<sup>6</sup> For analysis of intermittent bracing methods, such as Method 3 in this example, the area between braced wall panels was assumed to be occupied by full-height wall openings (i.e., devoid of any framing, sheathing, or other components that contribute to shear strength of a BWL). The continuous sheathing bracing amount adjustment factors in Section R602.10.5 of the *IRC* (e.g., 0.8 and 0.9 factors) were computed using the same analysis method, but with limits placed on the opening height between braced wall panels which increases the sheathed area of a braced wall line (i.e., decreases  $\Sigma(A_o)$ ) due to sheathing located above and below wall openings.

<sup>7</sup> The 1.3 factor is a whole building system strength adjustment. Refer to Addendum A to this article which serves to justify such an adjustment factor, if not one of much greater magnitude. The factor addresses many complex effects that are known to have a significant impact on performance of whole light-frame buildings. For example, a factor of 1.1 addresses load path through out-of-plane walls observed in whole building testing (Paevere 2002). A factor of 1.2 collectively addresses the contribution of non-structural and structural components of the building and actual boundary conditions to bracing elements that tend to increase performance, but which are ignored in the analysis. The net factor is taken here as  $1.2 \times 1.1 = 1.3$ .

## Example 2 – 25 ft. by 50 ft. Two-Story Home in Seismic Design Category D2

**Objective:** Evaluate IRC wall bracing amount for Method 3 (wood structural panel) bracing for transverse seismic story shear force on the bottom story. Transverse story shear is shared by three braced wall lines spaced at 25 ft. oc. The home is assumed to have 10-ft. high walls on both stories and nominal 1-ft. roof overhangs at eaves only.

- IRC Requirements

Seismic Design Category = D<sub>2</sub>

Bracing Amount Required (Table R602.10.1) = 55%

Length of Transverse Braced Wall Lines (BWL) = 25 ft.

Length of Bracing Required for Each Transverse BWL = (25 ft.)(55%) = 13.75 ft.

- Tributary Weight on Bottom Story BWLs

Roof ( $W_r$ ) = (50 ft.)(25 ft. + 2 ft.)(12 psf) = 16,200 lb.

Snow ( $W_s$ ) = 0 (max. ground snow load less than 40 psf)

Floor ( $W_f$ ) = (50 ft.)(25 ft.)(10 psf) = 12,500 lb.

Int. Walls/Partitions ( $W_p$ ) = (50 ft.)(25 ft.)(6 psf) = 7,500 lb. (upper story only)

Ext. Walls ( $W_w$ ) = [10 ft. + 1/2(10 ft.)][2(25 ft. + 50 ft.)(8 psf)] = 18,000 lb.

TOTAL ( $W$ ) =  $W_r + W_s + W_f + W_p + W_w = 54,200$  lb.

- ASD Story Shear Force

$V = [1.2 S_{DS}/R] \times [W] \times [1/1.4] = 1.2 [(1.17 g)/5.5](54,200 \text{ lb.})(1/1.4) = 9,918$  lb.

- ASD Shear Force (per each of three transverse BWLs uniformly spaced at 25 ft. oc)

$V_{BWL} = 1/3(9,918 \text{ lb.}) = 3,306$  lb.

- Braced Wall Line (BWL) ASD Shear Strength

$V_{\text{Allowable}} = (v)(L)(F_{PSW})$

$v = 634 \text{ plf}/2 = 317 \text{ plf}$  (nominal ultimate unit shear strength divided by 2.0 safety factor)

$L = 25 \text{ ft}$  (length of BWL)

$F_{PSW} = r/(2 - r)$  (empirical PSW shear strength reduction factor)

$r = 1/[1 + \alpha/\beta]$

$\alpha = \Sigma(A_o)/[H \times L]$

$\Sigma(A_o)$  = wall opening area = [25 ft. – 13.75 ft.](10 ft.) = 113 ft<sup>2</sup>

$\alpha = 113 \text{ ft}^2/[(10 \text{ ft.})(25 \text{ ft.})] = 0.45$

$\beta = \Sigma(L_i)/L$

$\Sigma(L_i)$  = sum of braced wall panel segments = 13.75 ft.

$\beta = 13.75 \text{ ft.}/25 \text{ ft.} = 0.55$

$r = 1/[1 + 0.45/0.55] = 0.55$

$F_{PSW} = 0.55/(2 - 0.55) = 0.38$

$V_{\text{Allowable}} = (317 \text{ plf})(25 \text{ ft.})(0.38) = 3,012$  lb.

$V'_{\text{Allowable}} = 3,012 \text{ lb.} (1.3) = 3,916 \text{ lb.} > 3,306 \text{ lb., OK}$

*Note: The conservative bias in the above examples is a result of a general rounding-up of the bracing percentage values in IRC Table R602.10.1. The larger conservative bias in Example 2 can be primarily attributed to a discrepancy in the treatment of arbitrary braced wall line spacing limits or baselines introduced to the IRC for different seismic design categories, but not necessarily in coordination with the analysis approach shown above.*

*“To date, story failure has only been observed in houses that would not meet the current IRC bracing requirements or would fall outside the scope of the IRC.” [p. 34]*

### Conclusions

1. The IRC wall bracing amounts in Table R602.10.1 of the code provide adequate seismic performance based on available technical data as well as experience in major disasters such as the Northridge Earthquake.

2. The IRC wall bracing amounts in moderate and high seismic hazard regions are a significant improvement (2- to 3-fold increase) compared to minimums in prior codes.

3. The IRC wall bracing amounts for seismic resistance (Table R602.10.1) are based on a rational analysis approach involving a number of judgments to reconcile engineering methods with conventional construction experience. However, a number of improvements should continue to be sought including:



- a. Improving the understanding and treatment of variations in seismic response for various conventional bracing methods.
  - b. Providing a consistent braced wall line spacing criteria (or baseline) across all seismic hazard conditions (remove arbitrary limits in favor of a consistent performance-based solution).
  - c. Increasing the understanding of differences between actual performance of conventional buildings and codified engineering theory used to predict performance (e.g., system effects) or “irregularity limits” intended to control performance.
4. The *IRC* bracing provisions lack a similarly rationalized set of bracing requirements and bracing amounts to ensure adequate wind resistance consistent with past conventional construction practices under conditions where performance has been acceptable.
  5. The continuous sheathed bracing method (R602.10.5 method), as first introduced in the *IRC* and justified using a modified version of the PSW design method to account for corner-restrained (or partially restrained) ends of a wall line, represents an initial attempt to rationalize the overturning load path and lateral resistance of conventional woodframe construction.

### Recommendations

*Engineering Provisions for Lateral Design* – A generalized mechanics-based analysis method for shear walls is needed to accurately predict strength of fully restrained and partially restrained shear wall segments in wall assemblies with varying degrees of wall openings. The codification and application of such a design method should also include consideration of whole-building effects that may still be ignored in the analysis, but which provide a non-negligible contribution to the lateral strength and stiffness of whole light-frame buildings.

*IRC Bracing Provisions* – Most importantly, a wind bracing table is needed to address a deficiency in the *IRC* provisions that carries potential safety implications, particularly for large modern homes. Work should also continue to improve existing seismic bracing provisions. Such work is in progress by an ICC-appointed Ad Hoc committee on wall bracing as well as a technical working group organized by the Building Seismic Safety Council (BSSC). The ICC Ad Hoc committee has worked diligently to correct, simplify, and clarify *IRC* bracing provisions for future editions of the code.

### References

- AF&PA. 2005. Special Design Provisions for Wind and Seismic. 2005 NDS Supplement. American Forest & Paper Association, American Wood Council, Washington, DC.
- APA. 2007. A Review of Large Scale Wood Structural Panel Bracing Tests. APA – The Engineered Wood Association, Tacoma, WA. Draft 4/13/2007. [www.wallbracing.org/news.cfm?newsid=11](http://www.wallbracing.org/news.cfm?newsid=11) (4/30/07).
- ASCE. 2005. Minimum Design Loads for Buildings and Other Structures (ASCE 7-05). American Society of Civil Engineers, Reston, VA.
- CABO. 1995. CABO One and Two Family Dwelling Code. The Council of American Building Officials (CABO), Falls Church, VA.
- Cobeen, K., J. Russell, and J.D. Dolan. 2004. Recommendations for Earthquake Resistance in the Design and Construction of Wood-frame Buildings, Parts I and II, Consortium of Universities for Research in Earthquake Engineering (CUREE), Richmond, CA.
- Crandell, J.H. 2006. *IRC Bracing Methods, Relevant Test Data, and Recommended Design Values for Wind Bracing Analysis of Conventional Wood-Frame Homes*. Draft, rev. 9/23/06. [www.foamsheathing.org](http://www.foamsheathing.org) (4/28/07).
- Crandell, J.H. and V. Kochkin. 2003. Common engineering issues in conventional construction. *Wood Design Focus*. 13(3): 13-23.
- Dolan, J.D. and C. Heine. 1997. Sequential Phased Displacement Tests of Wood-framed Shear Walls with Corners. Report No. TE-1997-003. Brooks Forest Products Research Center, VPI&SU, Blacksburg, VA.
- FEMA. 2006. Homebuilder's Guide to Earthquake-Resistant Design and Construction. FEMA-232. Federal Emergency Management Agency, Building Seismic Safety Council, National Institute of Building Sciences, Washington, DC. June 2006.
- FHA. 1949. A Standard for Testing Sheathed Materials for Resistance to Racking. Technical Circular No. 12. Underwriting Division, Federal Housing Administration, Washington, DC. Oct. 5, 1949.
- HUD. 2001a. Review of Structural Materials and Methods for Home Building in the United States: 1900 to 2000. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through [www.huduser.org](http://www.huduser.org)).
- HUD. 2001b. Model Guidelines for Design, Fabrication, and Installation of Engineered Panelized Walls. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through [www.huduser.org](http://www.huduser.org)).
- HUD. 2000. Residential Structural Design Guide, 2000 Edition. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through [www.huduser.org](http://www.huduser.org)).
- HUD. 1999. Evaluation of Housing Performance and Seismic Design Implications in the Northridge Earthquake. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through [www.huduser.org](http://www.huduser.org)).
- HUD. 1994. Assessment of Damage to Residential Buildings Caused by the Northridge Earthquake. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through [www.huduser.org](http://www.huduser.org)).
- ICBO. 1997. Uniform Building Code – Volume 2. International Conference of Building Officials, Whittier, CA.
- ICC. 2006a. International Residential Code (IRC). International Code Council, Inc. (ICC), Falls Church, VA.
- ICC. 2006b. International Building Code (IBC). International Code Council, Inc. (ICC), Falls Church, VA.
- Kasal, B., M.S. Collins, P. Paevere, and G.C. Foliente. 2004. Design models of light frame wood buildings under lateral loads. *J. Struct. Eng.* 130(8): 1263-1271.
- Line, P. 2002. Perforated shear wall design. *Wood Design Focus*. 12(2): 3-10.
- Line, P. and B.K. Douglas. 1996. Perforated shear wall design method. *In: Proc. of International Wood Engineering Conf.*, New Orleans, LA, Oct. 28-31, Vol. 2, pp. 345-352.
- NBS. 1948. *Strength of Houses: Application of Engineering Principles to Structural Design*. U.S. GPO, Washington, DC.
- Ni, C. and E. Karacabeyli. 2002. Capacity of shear wall segments without hold-downs. *Wood Design Focus*. 12(2): .
- Paevere, P.J. 2002. Full-scale testing, modeling and analysis of light frame structures under lateral loading. PhD thesis, Dept. of Civil and Environmental Engineering, The Univ. of Melbourne, Parkville, Victoria, Australia.
- Simpson. 2007. A collection of 15 different 3D and 5 different 2D sub-structure reports. Simpson Strong-Tie. [www.strongtie.com/news/industry/wall-bracing/3D-test/](http://www.strongtie.com/news/industry/wall-bracing/3D-test/) (4/30/07).

## ADDENDUM A

### Comparison of Design vs. Whole House Test



**Objective:** Compare predictions from various methods of lateral analysis to actual results from a whole building test program. Direct shear resistance in the longitudinal (N-S or X) direction of load application will be considered.

**Given:** Refer to **Figure A1** for an illustration of the light frame structure as tested and floor plan layout.

#### Exterior Side Walls W1, W2, and W4 (Fig. A1) N-S Wall Construction (parallel to load direction)

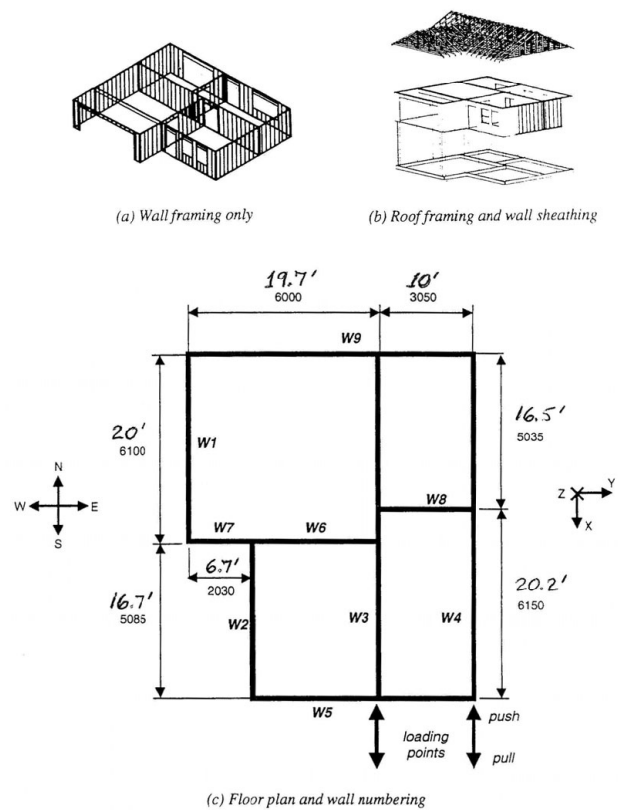
- 8-ft. wall height with various openings sizes and panel widths from 1.7 ft. to 4 ft. (segment aspect ratios of 4.7:1 to 2:1)
- 2 by 4@16 in. oc radiata pine (similar to southern pine)
- 3/8-in. plywood structural sheathing with 0.113 in. by 2 in. D-head Senco nails at 6 in. oc edges and 12 in. oc field of panel (continuous sheathing above windows and doors)
- 1/2-in. gypsum wall board (horizontal unblocked) with #6 by 1.2-in. gypsum screws at 12 in. oc at stud locations and 16 in. oc on plates

#### Exterior End Walls W5 and W9 (Fig. A1) E-W Wall Construction (perpendicular to load direction)

- No openings
- 4-ft. wood structural panels at ends only
- 1/2-in. gypsum wall board (GWB) on interior
- Same as exterior side walls in other respects

#### Interior Walls W3, W6, and W8 (Fig. A1)

- 1/2-in. gypsum panels on both sides installed same as on interior side of exterior walls
- interior walls nominally connected to bottom chords of trusses only (no blocking) and lapped with exterior wall top plates



**Figure A1.**—Floor plan of whole wood-frame building tested for lateral shear strength, stiffness, and force distribution to all wall lines and segments.

#### Roof Construction

- 2 by 4 trusses (radiata pine) at 24 in. oc fastened to top plate with metal connector (angle clip)
- Sheathing same as walls fastened to trusses at 6 in. oc edges and 12 in. oc field; no fastening at boundary along eaves (no fascia board or band)

#### Wall Bottom Plate Anchorage

- 1/2-in. anchor bolts with plate washers at approximately 3.3 ft. (1 m) spacing
- No hold-downs or connectors

*Note: The purpose of this test program was primarily to investigate lateral force distribution, calibrate an FEA model, and recommend the best simplified model for general design purposes (flexible diaphragm vs. rigid diaphragm vs. relative stiffness, etc.). Testing was also conducted to failure.*

#### Find

- Estimate lateral shear capacity using code-prescribed design methods for total direct shear resistance of building in direction of loading (assume torsional response due to rigid diaphragm action is negligible – maximum displacement of diaphragm in Y-direction was only 16% of maximum displacement in X-direction of loading)
- Compare results to tested ultimate (peak) shear capacity of the building

## PART I – Predicted Capacity (Three Estimates)

### (A) Prediction #1: Strict Code-Compliant Analysis

#### Shear Wall Resistance

Code design values for shear walls are based on “fully restrained” shear wall segments or perforated shear walls fully restrained at ends with hold-down brackets. This building had no hold-down restraints. Therefore, in the absence of code-compliant design procedures to account for this wall system configuration the shear wall capacity is considered to be zero.

#### Diaphragm Resistance/Rigidity

Code design values for wood structural panel diaphragms are provided, but only for diaphragm construction conditions which require the use of a boundary member on all perimeter edges of the diaphragm. This diaphragm lacks a boundary member (no band or fascia for sheathing edge fastening along the N-S sides). Furthermore, the code does not provide a means to combine the GWB ceiling diaphragm with the WSP roof diaphragm. Therefore, the diaphragm shear capacity and stiffness is considered to be zero.

Assuming that there is no allowance for alternate means and methods of design or exercise of designer judgment, the code-predicted shear strength is:

PREDICTION #1: 0 lb. (“straw house” prediction)

### (B) Prediction #2: Evaluate N-S Walls as Segmented Shear Wall System

In this prediction, the code method for analyzing segmented shear walls will be assumed to apply (even though the segments are not fully restrained and all but one 2.7-ft. segment on the front side of the building exceed the 3.5:1 maximum segment aspect ratio limit). The diaphragm will be assumed to distribute load to each N-S wall line (including interior wall W3) in the direction of load such that all walls are loaded to their maximum capacity simultaneously (e.g., relative stiffness approach). This prediction would tend to give the greatest (most non-conservative) estimate of direct shear capacity due to the above assumptions.

#### Exterior Wall Nominal Unit Shear Capacity (N-S Walls W1, W2, and W4)

$$v_{w1} = 560 \text{ plf (3/8-in. plywood, 6d common nail)} \\ \text{(Wind \& Seismic Table A.4.3A)}$$

$$v_{w2} = 120 \text{ plf (1/2-in. GWB, unblocked, \#6} \\ \text{screw 8 in./12 in.) (Wind \& Seismic Table 4.3B)}$$

*Note: GWB nominal capacity value is for greater fastening than actually used.*

$$v_{wc} = 560 \text{ plf} + 120 \text{ plf} = 680 \text{ plf}$$

*Note: ASD reduction (safety) factor is not applied to give a nominal “ultimate” capacity estimate. Also, the tested ultimate capacity for this wall assembly type was 640 plf (8 ft. by 8 ft. restrained wall shear test as part of the building test program). The difference of 40 plf reflects the difference in GWB attachment used on the building relative to that used above to*

*determine a design value. However, it will be assumed that boundary conditions on the interior finish in a whole building compensate for this difference.*

#### Length of N-S Exterior Wall Segments

$$W1 = 2 \text{ ft.} + 2 \text{ ft.} = 4 \text{ ft. (Front Garage Opening Wall)}$$

$$W2 = 2.5 \text{ ft.} + 1.5 \text{ ft.} + 2 \text{ ft.} = 6 \text{ ft. (Front Entry Wall)}$$

$$W4 = 4 \text{ ft.} + 4 \text{ ft.} + 4 \text{ ft.} + 4 \text{ ft.} = 16 \text{ ft. (Rear Wall)}$$

#### Nominal (Unfactored) Shear Capacity of All N-S Segments in Exterior Walls Combined

$$\Sigma L = 4 \text{ ft.} + 6 \text{ ft.} + 16 \text{ ft.} = 26 \text{ ft.}$$

$$V_{\text{Ext Walls N-S}} = 26 \text{ ft (680 plf)} = 17,680 \text{ lb.}$$

#### Interior Wall Nominal Unit Shear Capacity (N-S Wall 3)

$$v_{wc} = 120 \text{ plf} + 120 \text{ plf} = 240 \text{ plf} \\ \text{(two side 1/2-in. GWB)}$$

*Note: Unit shear value for GWB may be as much as 40 plf, too high for each side for reasons given above.*

#### Length of N-S Interior Wall Segments

$$W3 = 19.8 \text{ ft.} + 12.3 \text{ ft.} = 32 \text{ ft.}$$

#### Nominal (Unfactored) Shear Capacity of N-S Interior Wall

$$V_{\text{Int N-S}} = 32 \text{ ft. (240 plf)} = 7,680 \text{ lb.}$$

$$\text{PREDICTION \#2} = 17,680 \text{ lb.} + 7,680 \text{ lb.} = 25,360 \text{ lb.} \\ \text{ (“brick house” prediction)}$$

### (C) Prediction #3: Evaluate N-S Walls as Perforated Shear Walls (PSW)

Use the same assumptions stated for Prediction #2 except do not count wall segments that do not meet the maximum aspect ratio limit of 3.5:1. This prediction should tend to give a more accurate prediction because the PSW method provides reduction in shear capacity due to openings in walls and partially unrestrained panels in the wall line. So, the primary assumptions are that: 1) all walls reach their ultimate capacity more or less simultaneously, and 2) the ends of the PSW wall lines are fully restrained by the building corners or intersections with other wall lines. In addition, the shear capacity of the GWB will be reduced by 40 plf to account for the lesser fastening actually used in the building (see previous note).

#### Wall Line 1 (Garage Opening Wall)

$$\text{Max. opening height} = 5/6 h$$

$$\% \text{ Full-height sheathing} = \Sigma L_i / L = 2(2 \text{ ft.}) / 20 \text{ ft.} = \\ 0.2 = 20\%$$

$$C_o = 0.45 \text{ (Wind \& Seismic} \\ \text{Table 4.3.3.4)}$$

$$\text{Shear capacity, } V_1 = C_o v_{wc} \Sigma L_i = 0.45(680 \text{ plf} - \\ 40 \text{ plf})(4 \text{ ft.}) = 1,152 \text{ lb.}^*$$

\*USE  $V_1 = 0 \text{ lb.}$  (both segments have aspect ratio  $> 3.5:1$ )

#### Wall Line 2 (Front Entry Wall)

$$\text{Max. opening height} = 5/6 h$$



$$\% \text{ Full-height sheathing} = \Sigma Li / L = [0.8 \text{ ft.} + 1.6 \text{ ft.} + 1.1 \text{ ft.} + 2.7 \text{ ft.}] / 16.7 \text{ ft.} = 0.37 = 37\%$$

$$C_o = 0.52 \text{ (Wind \& Seismic Table 4.3.3.4)}$$

$$\text{Shear capacity, } V_2 = C_o v_{wc} \Sigma Li = 0.52(680 \text{ plf} - 40 \text{ plf})(2.7 \text{ ft.})^* = 899 \text{ lb.}$$

\*Only one segment has an aspect ratio less than 3.5:1.

#### Wall Line 4 (Rear Wall)

$$\text{Max opening height} = 5/6h$$

$$\% \text{ Full-height sheathing} = \Sigma Li / L = [4 \text{ ft.} + 4 \text{ ft.} + 4 \text{ ft.} + 4 \text{ ft.}] / 36.7 \text{ ft.} = 0.44 = 44\%$$

$$C_o = 0.52 \text{ (Wind \& Seismic Table 4.3.3.4)}$$

$$\text{Shear capacity, } V_4 = C_o v_{wc} \Sigma Li = 0.55(680 \text{ plf} - 40 \text{ plf})(16 \text{ ft.}) = 5,632 \text{ lb.}$$

#### Wall Line 3 (Interior Wall)

$$\text{Max opening height} = 5/6h$$

$$\% \text{ Full-height sheathing} = \Sigma Li / L = [19.8 \text{ ft.} + 12.3 \text{ ft.}] / 36 \text{ ft.} = 0.89 = 89\%$$

$$C_o = 0.86 \text{ (Wind \& Seismic Table 4.3.3.4)}$$

$$\text{Shear capacity, } V_3 = C_o v_{wc} \Sigma Li = 0.86(240 \text{ plf} - 80 \text{ plf})(32 \text{ ft.}) = 4,403 \text{ lb.}$$

#### Nominal (Unfactored) Shear Capacity of All N-S Walls

$$\text{PREDICTION \#3} = 0 \text{ lb.} + 899 \text{ lb.} + 5,632 \text{ lb.} + 4,403 \text{ lb.} = 10,934 \text{ lb. "stick house" prediction}$$

#### PART 2 – Comparison to Tested N-S Direct Shear Capacity of Building

PREDICTION #1: Code-Compliant Analysis = 0 lb.

PREDICTION #2: Segmented Shear Wall + Assumptions = 25,360 lb.

PREDICTION #3: Perforated Shear Wall + Assumptions = 10,934 lb.

TESTED PEAK SHEAR LOAD CAPACITY = 25,000 lb.  
(direct shear resistance in N-S direction of loading)

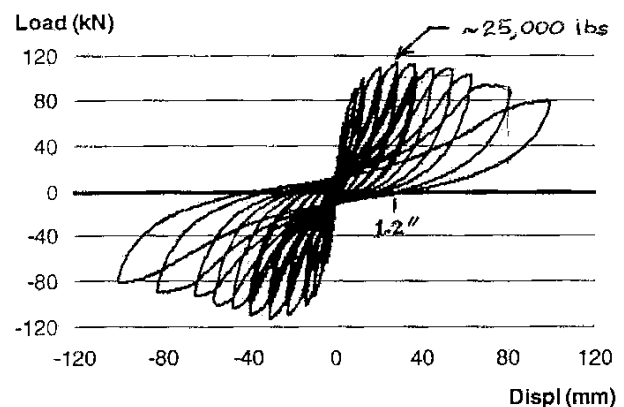
#### Conclusions

1. The code does not provide shear wall and diaphragm assembly design data or relevant design guidance for lateral analysis of conventional construction as represented by this whole building test and design example.
2. The most non-conservative set of analysis assumptions resulted in the most accurate estimate of total direct peak shear resistance of this whole building. The assumptions included:
  - assuming segmented shear wall behavior in the absence of hold-down restraints,
  - including all wall segments even of greater aspect ratio than permitted by code,

- assuming gypsum sheathing was fastened more closely than actually done, and
- assuming all walls reach their peak capacity at more or less the same time.

3. What was considered to be the most accurate prediction with the least number of non-conservative assumptions was too low by more than a factor of 2.
4. 10 percent of the 25,000 lb. total direct N-S shear force was actually transferred as out-of-plane forces through walls perpendicular to loading – this can not be accounted for by manual procedures for lateral force distribution except by way of a “system factor” adjustment.
5. The test building included no finishes components (e.g., windows, doors, siding, trim, etc.). These “non-structural” components, if considered, can increase shear capacity by as much as a factor of 2 and stiffness by more than 3-fold.

*Note: The building was actually tested using a cyclic (fully reversed) displacement pattern as shown in Figure A2 (X-direction wall reaction forces vs. X-direct displacement at centroid of roof diaphragm).*



**Figure A2.**—Cyclic displacement load pattern for whole house test.

#### References

- AF&PA. 2005. Special Design Provisions for Wind and Seismic. 2005 NDS Supplement. American Forest & Paper Association, American Wood Council, Washington, DC.
- HUD. 2001. Whole Structure Testing and Analysis of a Light-Frame Wood Building (Three Reports), prepared by NAHB Research Center Inc. for the National Association of Home Builders and U.S. Department of Housing and Urban Development, Washington, DC. (Research conducted by NC State University and CSIRO-Australia).
- Kasal, B., M.S. Collins, P. Paevere, and G.C. Foliente. 2004. Design models of light frame wood buildings under lateral loads. *J. Struct. Eng.* 130(8): 1263-1271.
- Paevere, P.J. 2002. Full-scale testing, modeling and analysis of light frame structures under lateral loading. PhD thesis, Dept. of Civil and Environmental Engineering, The Univ. of Melbourne, Parkville, Victoria, Australia.

Jay H. Crandell, PE., A-RES Consulting, West River, MD.