Whole structure testing and analysis of a light-frame wood building

Phase 1 – Test House Details and Preliminary Results

Greg C. Foliente
CSIRO

Bo Kasal
North Carolina State University, Raleigh, NC

Phillip Paevere, Lyndon Macindoe, Rod Banks, Mike Syme and Craig Seath
CSIRO

Michael Collins
North Carolina State University, Raleigh, NC

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Please address all enquiries to: The Chief, CSIRO Building, Construction and Engineering P.O. Box 56, Highett, Victoria 3190 Australia
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Whole structure testing and analysis of a light-frame wood building

Phase 1 - Test House Details and Preliminary Results

Executive Summary

Most people in the US rely on their houses to provide them safety and protection against natural disasters. However, the public’s level of confidence has been badly shaken as recent hurricanes and earthquakes brought extensive damage to woodframe residential buildings and inflicted emotional, social and economic difficulties on affected people and communities.

The long-term goal of this project is to make houses more affordable and safer for people than they currently are through efficiency in the structural system. A path towards this goal is the development of tools and procedures that can be used to establish the structural performance criteria for low-rise residential buildings and to cost-optimize house construction, i.e., find the cheapest design for a given performance level. Following this path, this project aims to develop experimentally validated whole building models of a simple one-story house and develop simplified analysis and design procedures.

Phase 1 of the project involves design and construction of the test house, development of an extensive instrumentation system, preliminary testing of walls and of the entire house and preliminary modeling and analysis. Phase 2 involves continuation of the testing program and development of a preliminary concept for a simplified “system-based” engineering design method for shear walls in wood buildings subjected to wind load.

This report describes developments and accomplishments in Phase 1 of the project. Here, we review the literature on house testing and modeling, and describe our strategy to develop improved tools for performance analysis of light-frame buildings under wind and earthquake loads. We also present details of the experimental program, test house and instrumentation, the current progress in the development of whole building models, and preliminary results of both experiments and analysis.

Most analytical and experimental work in the literature focuses on understanding the response behavior and mechanism at the sub-system level such as shearwalls. Recommendations arising from studies at this level typically ignore systems effects, and the fact that the actual forces that the sub-system sees depend primarily on the geometric and structural characteristics of the whole building. This project will provide us information and data that will be useful in filling critical gaps in existing knowledge.

At the end of Phase 1, we can conclude that:

• Measuring the loads at the base of the house with three-directional load cells provide very useful information that could contribute to our understanding of load sharing and
distribution in a wood-frame building. Although further test data are needed, we can say that given the preliminary results that we have obtained, this is achievable.

- It is highly feasible that the three-dimensional model of the house can be experimentally validated with the test data that we would obtain from testing of the full-scale house, and of its constituent components and materials.

With these promising developments, we have started work on Phase 2 of the project.
Introduction

The vast majority of people in the US live in woodframe buildings. They rely on these structures for safety and protection against natural disasters. However, the general public’s level of confidence has been shaken as recent hurricanes and earthquakes brought extensive damage to light-frame buildings and inflicted emotional, social and economic difficulties on affected people and communities.

A light-frame wood building is an assemblage of several components or sub-assemblies with repetitive members such as walls, floors and roof systems connected by inter-component connections such as bolts, metal straps or proprietary connectors forming a three-dimensional indeterminate structural system. Because very little is known about how the load is shared and distributed in such a complex structural system, simplifying assumptions are made in analysis and design. This can result in either over- or under-designed elements, resulting in either over-conservative (and therefore uneconomical) or less safe structures. Shear wall forces in a light-frame wood building, for example, may be over-predicted by 130 percent or under-predicted by 60 percent using current design procedures (Kasal and Leichti 1992).

Fig. 1 shows a framework for analytical and experimental studies of structural performance of light-frame buildings. Most analytical and experimental work in the literature focus on understanding the response behavior and mechanism at the sub-system level such as shearwalls – Level 1 in Fig. 1 (Foliente and Zacher 1994). Recommendations arising from studies at this level typically ignore systems effects, and the fact that the actual forces that the sub-system sees depend primarily on the geometric and structural characteristics of the whole building – Level 2 in Fig. 1. For example, Level 1 tests do not always take into account the effect of boundary conditions on the results. Boughton’s (1988) results from tests of a full-scale house (Level 2) and isolated wall components (Level 1) have shown that boundary conditions in the wall test “can influence the stiffness, ultimate load and failure characteristics of components under test”. Full-scale whole house testing is needed to prove the validity of isolated wall test results and their interpretations (Boughton 1988; Foliente and Zacher 1994).

Unfortunately, the relatively fewer experimental studies on whole building response under simulated wind and/or earthquake loads have not provided the right kind and/or amount of data needed to:
1. assess the validity of observations and design recommendations based on isolated wall component tests; and
2. develop and validate whole building analytical models.

Results of the test conducted by Phillips et al. (1992) have been used in experimentally validating the whole house model by Kasal et al. (1994) of a simple box-like structural system under static loading. More realistic models, and more and better test data, are now needed to improve our understanding of the behavior of light-frame buildings under natural hazard loads.
The long-term goal of this project is to make houses more affordable and safer for people than they currently are through efficiency in the structural system. A path towards this goal is the development of tools and procedures that can be used to establish the structural performance criteria for low-rise residential buildings and to cost-optimize house construction, i.e., find the cheapest design for a given performance level. Following this path, this project aims to develop experimentally validated whole building models of a simple L-shaped one-storey house and develop simplified analysis and design procedures. Thus, after experimentally validating our whole building models, we intend to:

1. Investigate the load distribution, load-sharing mechanisms and load re-distribution in light-frame buildings;
2. Investigate the sensitivity of global building response to changes in component properties of the house (e.g., effects of different materials/systems, different designs or construction details, mistakes, etc.); and
3. Develop improved design procedures for light-frame wood buildings against lateral loads.

We believe that our findings will contribute to the basic understanding of the response of low-rise, light-frame structures to natural hazard lateral loads and to the development of safer and more affordable housing to the benefit of the public.

This report describes Phase 1 results of this project. This includes our strategy to develop improved tools for performance analysis of light-frame buildings under wind and earthquake loads that can be used to improve current design procedures. Details of the experimental program, test house and instrumentation are presented. We also present the current progress in the development of whole building models, and preliminary results of both experiments and analysis. In Phase 2, we will develop a preliminary concept for a simplified “system-based” engineering design method for shear walls in wood buildings subjected to wind load.

**Literature Review**

**Experimental Testing**

While disaster reconnaissance studies have been a primary means of validating and improving design methods, these have proven to be a slow process and lack the
quantitative aspects needed to improve engineering tools. Furthermore, data collected from these efforts do not provide information on processes leading to failure. Laboratory and field tests are useful because they allow close monitoring of structural behavior and response under controlled loading conditions, and provide information that can be used directly to improve new and existing products and design procedures.

Mathematical models for analysis of light-frame structural systems are also needed to extend the usefulness of experimental data, predict structural behavior under specified natural hazard loads and conduct parametric response studies. These models are useful in research and can assist in the development and/or calibration of code requirements and design procedures. But because models are generally much simpler than the actual mechanisms they represent, experimental testing is, in turn, needed in the development, refinement and validation of analytical models. Thus, integration of results of laboratory testing and analytical modeling of light-frame buildings is needed to provide the necessary technical basis for developing rational design procedures.

**General Behavior of Light-Frame Systems**

Foliente and Zacher (1994), Barton (1997) and Gad (1998) have reviewed the literature on experimental testing of light-frame wood and steel structural systems. Most available experimental cyclic test data are based on tests of connections and sub-assemblies such as shear walls and diaphragms. A common observation from these tests is that the hysteresis trace of a sub-system or sub-assembly is governed by the hysteretic characteristics of its primary connection. Thus, it is important to characterize the hysteretic behavior of the connections in order to characterize the behavior of light-frame structural systems.

Fig. 2 shows typical hysteresis data from cyclic tests of connections and subassemblies in light-frame construction. Several characteristic features of cyclic response of these systems can be noted (Foliente 1995):  
1. nonlinear, inelastic load-displacement relationship without a distinct yield point, progressive loss of lateral stiffness in each loading cycle (or stiffness degradation),  
2. degradation of strength when cyclically loaded to the same displacement level (or strength degradation), and  
3. pinched hysteresis loops.

In some tests, the presence of initial slackness due to shrinkage, clearances at fastener holes or deformation at supports has been observed.

**Previous Work on Whole Building Testing**

During the last two decades, several attempts have been made to investigate the behavior of the whole building (Tuomi 1980). Vibration tests of full-size structures are extremely expensive and only few such tests have been reported (Foliente and Zacher 1994). The experimental work of Tuomi and McCutcheon (1974), Sugiyama et al (1988), Boughton (1988), Reardon (1991) and Phillips (1990) involved static loading. All of the performed tests were very limited in scope and results are mostly restricted to observation of global response and typical failure modes. Some of these test results are reviewed in Kuo (1989) and Kasal et al (1994a).

Phillips (1990) recorded the reaction forces beneath individual walls in the plane of loading to address the problem of load sharing within a nonlinear system. Although only static cyclic load was applied, the experiments offered important insight into load sharing in a full-scale building with regular geometry. This is, perhaps, the most comprehensive experiment documented so far. Properties of individual members, connections and sub-
assemblies of the experimental house were established prior to the entire building test. This makes the test suitable for validation of an analytical model (Kasal et al. 1994a).

More recently, tests of full structures were reported by Japanese and Australian researchers (Suzuki et al. 1996; Reardon and Henderson 1996; Barton 1997; Gad 1998; Gad et al. 1998). Suzuki et al. (1996) and Gad et al. (1998) used an impact hammer to estimate the natural frequencies and modal shapes of three different wooden houses and a cold-formed steel-framed house, respectively. None of the reported experiments contained instrumentation to measure forces between components, which makes it impossible to determine the load distribution within the structure.

**Research Needs and Opportunities**

1. Previous tests are limited to failure observations and determination of global behavior. Even if a lot of information was collected, test results alone have limited engineering use. Experimentally validated mathematical models for analysis of light-frame structural systems are needed to extend the usefulness of experimental data and to perform parametric and sensitivity studies.
2. No conclusions about load path within the structure [with the exception of Phillips’ (1990) test on a house with regular plan and elevation] can be drawn. Load sharing and distribution need further study, especially on a house with irregular plan and elevation [e.g., Reardon and Henderson’s (1996) test house].
3. Little is known about specific properties of materials, connections and sub-assemblies used to construct the full-building experimental models. This makes it difficult to use the information for validation of analytical models. Most testing in the past was planned and conducted without consideration of analytical modeling requirements.
4. Full-scale response data of light-frame buildings under cyclic loading need to be obtained (from laboratory tests and field measurements of instrumented buildings), compared with component test results and used in validation and refinement of analytical models.
Modeling and Analysis

Previous Work on Modeling


Many hysteresis models have been proposed for general construction and for light-frame structures; these have been reviewed by Fardis (1991) and Foliante (1997b), respectively. For light-frame structures, it is important that the four basic hysteresis characteristics observed in tests and listed earlier are properly modeled. The Bouc-Wen-Baber-Noori-Foliante (BWBNF) model (Fig. 3a) and the Kasal-Xu model (Fig. 3b) meet this requirement. The BWBNF model has been successfully used in time history and zero-mean random vibration analyses of single-degree-of-freedom (SOF) systems (Foliante, 1995a; Foliante et al 1996a; 1996b) and can be used for non-zero-mean random vibration analysis and seismic damage analysis. The Kasal-Xu model, a nonlinear hysteretic spring, has been validated via comparison with limited experiments and can be attached to a general-purpose Finite Element (FE) software (Kasal and Xu 1996; 1997). These two phenomenological hysteresis models are used in this project.

![Hysteresis models](image)

(a) BWBNF model
(b) Kasal and Xu model

Models of subassemblies such as shear walls, roofs and floors are relatively well documented and static and dynamic models are available (Dolan 1989; Kasal et al. 1994; White and Dolan 1994; Barton 1997). Models were mostly verified for in-plane shear loads. Polensek’s tests (1975) were used by Kasal and Leichti (1992) to verify analytical models of walls subjected to combination of axial load and uniformly distributed pressure.

Most of the available models of the whole structure focused on quasi-static behavior (Gupta and Stalnaker 1991; Kasal 1992). Those that can be used in dynamic analysis [e.g, models proposed by Yoon and Gupta (1991), Chehab (1982) and Lee (1987)] have major drawbacks. Significant simplifications were made to reduce the large number of degrees of freedom or to model system behavior. For example, pinching, slip and stress and stiffness degradation typical in connections with dowel-type fasteners (nails, screws or bolts) were ignored thus yielding inaccuracies in calculating cyclic or dynamic response. Tarabia (1994) developed dynamic models but offered no experimental validation of the

Load Sharing

Significant load sharing takes place in light-frame buildings (Phillips 1990; Phillips et al. 1992; Ohashi and Sakamoto 1988). This is a function of the relative stiffness of individual sub-structures and components as well as inter-component connections (Kasal et al. 1994b). Since the stiffness in nonlinear systems is a function of the load, the stiffness relationships and subsequent load distribution within the nonlinear structure can change during the course of loading. Moreover, the load-history dependent properties of connections in light-frame building — such as strength and stiffness degradation — can significantly affect this force distribution.

Research Needs and Opportunities

1. One of the main challenges is the development of structural models for irregularly shaped light-frame structure, that incorporate elements (of sub-systems and/or connections) capable of nonlinear, non-conservative response with pinching (or slip) and stiffness and strength degradation. The currently available models have limited capabilities. Experimental validation of whole structure models is insufficient.
2. Valid or reliable specification of input values for the elements and the hysteresis models that comprise the structural model has been identified as a problem (Gupta and Moss 1991; Foliente 1997a). System identification techniques need to be applied to obtain valid hysteresis model input values in a consistent and systematic manner.
3. Commonly used software programs (e.g., ANSYS, ETABS/SAP, DRAIN, etc.) will be more useful than they are now if they have specific hysteretic elements applicable to light-frame systems in their element libraries. These programs provide a wide variety of features that can be used for routine static and dynamic analysis. In addition, many analysts and engineers are already familiar with them.
4. Model parameters (related to material types and system configuration of the actual structure) that significantly affect response should be identified. Various types of parametric and sensitivity studies (e.g., influence of material and joint characteristics, material or system substitution) should be conducted (Stalnaker and Gramatikov 1991). Experimentally validated analytical models are needed to obtain meaningful results from such studies.

Both Kasal and Xu’s (1997) model and the BWBNF (Foliente 1995) model demonstrate the basic characteristics of hysteretic joints and subassemblies in light-frame buildings, and are currently being incorporated into common structural models. The former has been incorporated into ANSYS and the latter has been used in random vibration analysis and preliminary seismic reliability analysis (Foliente et al. 1999; 2000). To address the need to obtain valid input values for analysis, system identification techniques have been used to determine the parameters of Kasal and Xu’s model and are being used to determine the parameters of the BWBNF model (Foliente et al. 1998a; 1998b; Zhang et al. 1999). This project focuses on experimental validation and calibration of the structural models under development and investigation of load sharing mechanisms and distribution.
Light-frame wood buildings in the US may be non-engineered (commonly called "conventional" construction), fully engineered or mixed (i.e., combined conventional and engineered construction) (Cobeen 1997; Foliente 1998). For the most part, conventional construction provisions have little or no direct relations with engineered design provisions. Combined conventional and engineered construction, which is increasingly practiced in California and other US states, results in significant variations in design practice even in the same locality (Cobeen 1997).

In Australia, deemed-to-comply provisions for light-frame wood construction are given in a set of span tables and supporting specifications for various members of the house. Used widely by builders throughout Australia, including regions with high winds, these provisions have been developed based on accepted engineering design standards (MacKenzie 2000) - the center and left blocks in the diagram in Fig. 4 show this development process. This is a rational approach, and has also been applied (although to a less extent, compared to the Australian practice) in the development of the *Wood Frame Construction Manual* (WFMC-SBC) for one- and two-family dwellings in high wind areas in the southeastern US (AFPA 1995).

Figure 4. Framework for development of improved design procedures (Foliente 1998)

Fig. 4 shows an ideal way of developing improved design methods, both for engineered construction and conventional (or deemed-to-comply) construction. Improvements are made based not only on calibration of design methods to field observations and historical performance (e.g., Crandell and McKee 2000), but also on improved building performance models and state-of-the-art analysis techniques (right-most block in Fig. 4). The present work is part of a larger effort (Kasal et al. 1999; Foliente at al. 1999; 2000) towards this goal.

Key areas where improvement is critical to the development of better performance analysis tools include:

- Consideration of system effects, and sources of uncertainties and variability;
- Development of improved structural models and experimental validation of whole building models;
- Generation (from experiments) and identification of quality model input data; and
- Reliable methods of model calibration based on field data.

Field Code Changed
Experimental Testing Program

Overview of Test Program

Because previous full-scale light-frame building tests that we are aware of do not provide enough data that can be used for experimental validation of a detailed three-dimensional model of a light-frame house, we built and are, currently, testing a full-scale house to:

- Obtain data for whole house model validation;
- Improve our understanding of load sharing and distribution within a light-frame wood building; and
- Compare the performance of walls tested as part of the house and those tested in isolation.

In this report, we describe the basic details of the test house and some preliminary test results related to the second and third objectives.

The full-scale whole house testing program includes:

- Non-destructive dynamic testing – the natural frequencies and other dynamic characteristics of the house are obtained by measuring the acceleration response from impact tests using an instrumented hammer and a swept sine-wave excitation using a controllable vibrator.
- Pseudo-elastic tests – a point load is applied laterally, at the ceiling or roof level, at various locations in the house to determine the extent and the distribution of the load within the structure. To avoid irrecoverable damage, the total house displacement in these tests is targeted to be less than ±2mm.
- Ultimate load test – a lateral cyclic load will be applied statically (in line with the long direction) until failure. Complete hysteresis response of the house and the wall components will be obtained.

The main shear-resisting walls in the house will be tested individually (to destruction) as isolated or free-standing walls. Coupon tests of materials and typical details used in the house will also be conducted; these include:

- materials tests
  - bending properties of sheathing panels
  - modulus of elasticity of all framing materials
- sheathing joints
  - plywood to stud
  - gypsum to stud
- inter-component connections
  - wall to foundation
  - wall to wall
  - ceiling to wall

Test House Details

The test house design specifications, as shown in Fig. 5, were supplied by the NAHB Research Center Inc., with slight modifications to the materials used (Australian equivalents to US materials were used) and the floor plan (the size of the rooms were adjusted to match the strong floor anchorage layout in the CSIRO Structures Laboratory).

The framing details, wall numbering notation and overall dimensions are shown in Fig. 5. Walls W1, W2, W3 and W4 are in line with the direction of the applied loading. Detailed
material specifications are given in Appendix A. Photographs of the house at various stages of construction are shown in Fig. 6 and a complete set of CAD drawings for the house is given in Appendix D.

The entire house is fully supported by load cells that take measurements in three principal directions and fully instrumented with displacement transducers. The load cells are located under the bottom plate at approximately one metre spacings (3.3 ft) as well as at all corners and openings, making it probably the first test house to be instrumented in this manner, and to this extent. The load cells were developed and assembled at CSIRO especially for this project, and have been extensively tested and calibrated. The instrumentation system is described in more detail in the following section.

**Instrumentation Details**

**Load Cells**

The load cells measure the distribution of forces throughout the house by measuring the reaction forces under the bottom plates. They are located under the bottom plate at approximately one metre (3.3 ft) spacings as well as at all corners and openings. The entire system of load cells for the house is shown in Fig. 7a. Each load cell unit comprises of three identical uni-directional load cells coupled together to measure forces in three principal directions, X, Y and Z (directions shown in Fig. 5). The three dimensional load cell units are formed from three identical shear beam load cells. A single calibration factor could be used to achieve an accuracy of ± 1%. The load cell units are held in place and attached to the strong floor of the laboratory via a heavy metal frame. Two load cell units at a corner, and the hold-down frame are shown in Fig. 7b. The cells are connected to the bottom plate via a steel channel as shown in Fig. 7c. The pin connection between the channel and the load cell prevents moments being applied to the load cell.

**Displacement Gauges**

Three types of digital displacement gauges are used to measure displacements:
- 300 mm (11.8 in) digital calipers for all in-plane (X) displacements at the top plate
- 150 mm (5.9 in) digital calipers for all out-of-plane (Y) and vertical (Z) displacements
- 25mm (1 in) digital gauges for all bottom plate displacements

Each of the displacement gauge types is shown in Fig. 8.

**Data Acquisition**

The load cell system is connected together by four separate RS485 lines which are then read into the serial port of an IBM compatible PC via RS232 converters. The digital displacement gauges are multiplexed onto eight additional RS232 serial ports on the computer. The entire instrumentation system, data visualization and storage is managed by a Microsoft EXCEL based application running under a Windows NT4.0 operating system.
(a) Wall framing only

(b) Roof framing and wall sheathing

(c) Floor plan and wall numbering

Figure 5 – Details of test house (dimensions in mm)
(a) Wall framing only

(b) Plywood-sheathed walls and roof trusses

(c) Completed house without gypsum board lining

Figure 6 – Photographs of test house during construction
(a) Grid of load cells supporting entire house

(b) Two load cell units supporting corner of house

(c) Load cell units and pin connections to supporting channel

Figure 7 – Photographs of load cell system
(a) Measurement of bottom plate displacement

(b) Measurement of large in-plane displacement using 300mm digital calipers

(c) Displacement gauge support frames for in-plane and out-of-plane displacements

(d) Attachment of displacement gauge supports to house support frame

Figure 8 – Photographs of displacement measurement system

**Preliminary Wall Test Results**

Four wall tests have been conducted on wall types similar to those which are used in the test house. The main purpose of these experiments was to trial-run the experimental equipment and to obtain some preliminary data on the capacity of the walls in the house. The four tests conducted were:

1. Monotonic test conducted on wall W2 sheathed with 13mm (0.5 in) gypsum attached by self-drilling screws @ 300mm, (11.8 in) and 7mm (0.27 in) bracing ply (instead of 9.5mm [0.37 in]) attached with hand-driven 30x2.8mm (1.2 x 0.1 in) galvanized clouts (instead of machine driven nails) at 150 / 300 mm (5.9/11.8 in) spacing.

2. Static-cyclic test conducted on wall W2 sheathed with 13mm gypsum attached by self-drilling screws @ 300mm (11.8 in), and 7mm (0.28 in) bracing ply (instead of 9.5mm (0.37 in)) attached with hand-driven galvanized clouts (instead of machine driven nails) at 50 / 300 mm (1.97/11.8 in) spacing.
3. Monotonic test on a sheathed 2.4x2.4m (8 x 8 ft) wall segment sheathed with 13mm (0.5 in) gypsum attached by self-drilling screws @ 300mm (11.8 in), and 7mm (0.28 in) bracing ply attached with hand-driven galvanized clouts at 150 / 300 mm (5.9/11.8 in) spacings.

4. Monotonic test on a sheathed 2.4x2.4m (8 x 8 ft) wall segment with 13mm (0.5 in) gypsum attached by self-drilling screws @ 300mm (11.8 in), and 9.5mm (0.37 in) ply attached with 2.87 x 50mm (0.11 x 1.96 in) D-head machine driven nails at 150 / 300mm (5.9/11.8 in) spacings

A summary of the test results and observations are given in Table 1 and Fig. 9 below.

Figure 9 - Load - displacement curves for preliminary wall tests

Table 1 - Summary of preliminary wall testing results

<table>
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<th>Deflection (peak load)</th>
<th>Comments</th>
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<tr>
<td>1</td>
<td>+22 kN (+5,000 lb)</td>
<td>+60 mm (+2.4 in)</td>
<td>Plywood and stud layout in this test slightly different to wall in house</td>
</tr>
<tr>
<td>2</td>
<td>+30.7 kN (+6,900 lb)</td>
<td>+35 mm (1.4 in)</td>
<td>Plywood and stud layout modified to match in-house wall, fastener spacing of 50/150 (1.97/11.8 in) used. The cyclic loading was not applied evenly (wall was released too fast)</td>
</tr>
<tr>
<td>3</td>
<td>+29.6 kN (+6,650 lb)</td>
<td>+60 mm (+2.4 in)</td>
<td>7mm (0.27 in) plywood and clouts at 150/300 (5.9/11.8 in) used for bracing</td>
</tr>
<tr>
<td>4</td>
<td>+25.5 kN (+5,730 lb)</td>
<td>+45 mm (+1.77 in)</td>
<td>9.5mm (0.37 in) plywood and machine nailing used for bracing. Many of the machine-driven nails had missed the stud.</td>
</tr>
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</table>
Preliminary Coupon Test Results

At the time of preparation of this report, a series of gypsum board to stud joint tests had been carried out. The results of these tests are given in Appendix C.

Preliminary Whole House Test Results

Dynamic Testing

At the time of preparation of this report, the house had undergone two series of tests. The first was a non-destructive dynamic impact test. The natural frequencies of the house in the North-south (long) direction and the East-West (short) direction were obtained by measuring the acceleration response from impact tests using a rubber tipped hammer. The house was instrumented with five accelerometers and was excited from three different locations to ensure that the first (fundamental) mode was excited. To ensure repeatability, the test was repeated six times for each excitation. The excitation and accelerometer locations are shown in Fig. 10. and the time history responses and autospectra are shown in Fig. 11. The three excitation locations for each direction produce consistent values for the fundamental frequency of 13.6 Hz in the North-South direction and 14.8 Hz in the East-West direction.

Table 2. Fundamental frequencies of house from dynamic testing

<table>
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<th>Direction</th>
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<tr>
<td>North-South</td>
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<tr>
<td>East-West</td>
<td>14.8</td>
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</table>

Pseudo-Elastic Testing

The second test was a preliminary pseudo-elastic test to determine load distribution. A point load of approximately one ton (in both the push and pull directions) was applied at the ceiling level on Wall W2. The displaced shape and the distribution of the load throughout the house are shown in Fig. 12. A free-standing test of Wall W2 has also been conducted. The stiffness obtained for Wall W2 in the free-standing test, and whole house tests are compared in Table 3, the values quoted are based on the secant stiffness at one millimeter displacement.

The results in Table 3 and Fig. 12 indicate that significant load sharing occurs between the main shear resisting walls in the house. Wall W2 has an apparent stiffness when in the house of nearly double the value when free-standing. The load distribution shown in Fig. 12 supports this observation and also gives an insight into the load-sharing mechanism. Walls W3 and W4 are taking significant in-plane load even though the load is applied to Wall W2 only (note the x-direction reaction forces). The load has been transferred to Walls 3 and 4 primarily by the end walls, W5 and W9 (note the y-direction reactions), and the roof structural system. This observation holds true for loading in both push and pull directions. The data used for Fig. 12 is given in Appendix B, along with the distribution of gravity load throughout the house; note that the total mass of the house is 4.9 ton.

Table 3. Comparison of Wall W2 stiffness: free-standing vs. in-house.

<table>
<thead>
<tr>
<th>Loading Direction</th>
<th>Free-Standing Stiffness (N/mm)</th>
<th>In-House Stiffness (N/mm)</th>
<th>Ratio: In-House / Free-standing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Push</td>
<td>3300</td>
<td>5950</td>
<td>1.80</td>
</tr>
<tr>
<td>Pull</td>
<td>3200</td>
<td>5990</td>
<td>1.87</td>
</tr>
</tbody>
</table>
Figure 10. Excitation and accelerometer locations for dynamic testing on house

Figure 11. Time histories and autospectra from non-destructive dynamic tests on house
Figure 12. Summary of displaced shape and load distribution for elastic test on Wall W2
Whole Structure Modeling

Overview of Model Development

Two types of models are being developed:
1. Whole house Finite Element (FE) model of the building and its sub-systems
2. Shear-building model for stochastic analyses

The first is a three-dimensional model of the experimental house. This type of model allows us to investigate detailed response of local elements and details. We intend to use this model to study load sharing/interaction and distribution, to track the formation of local failure mechanisms in individual components, and to provide the needed behavior information for the shear-building and frame models, when experimental data are not directly available for the latter models.

The second is a "shear-beam" or, simply, a shear-building model, which is most commonly used in dynamic analysis. The model’s relative simplicity allows consideration of increased complexity on the loading side and in the nonlinear dynamic response. This model is not suitable to predict local or global failure mechanisms or damage levels in individual structural components, but is suitable for interpreting data measured from dynamic tests in the laboratory (e.g., shaketable tests) and from field measurements (e.g., instrumented buildings), and for conducting studies on the effect of soil conditions in the form of boundary conditions, higher mode effects, etc.

These two models can be used together in a hybrid method of analysis to take advantage of their complementary features (Kasal et al. 1999). Key to this is the use of system identification methods. Foliente et al. (1998) and Zhang et al. (1999) have presented various techniques and applications of system identification in inelastic dynamic analysis.

Whole House Finite Element Model

The detailed finite element model of the whole structure is based on the work of Kasal (1992) and Kasal et al (1994). The transformation of detailed sub-structure models into the global full structure model is based on energetically equivalent system approach rather than using sub-structuring. The approach is discussed in detail in the above publications.

The three-dimensional FE model of the house consists of a system of linear elements representing wood and wood composites, and nonlinear and non-conservative elements representing connections. The nonlinear elements are hysteretic, having load-history dependent behavior which includes stress and stiffness degradation (Xu and Kasal 1997). The full-structure model utilizes data obtained from cyclic connection tests and analyses of sub-structures with relatively detailed geometry and properties modeling.

Fig. 13a shows a three-dimensional FE model of the test house. A detailed three-dimensional model of Wall W2 in this house is shown in Fig. 13b with an energy-equivalent reduced degree-of-freedom (DOF) model shown in Fig. 13c. The former model includes material and connection properties and the analysis yields a load-deformation curve representing an in-plane shear behavior. Such behavior is essential in a light-frame structure subjected to a dynamic lateral load. Details of this model are described in the following section. Once each wall in the whole structure is analyzed and in-plane properties obtained, a system identification program is used to extract parameters of a diagonal spring representing the in-plane wall behavior. The system identification...
program is based on minimization of the error in dissipated energy during cyclic loading. This makes the simplified wall model (Fig. 13c) energetically equivalent to the detailed full model (Fig. 13b). The simplified model is non-conservative in shear but conservative in bending and axial directions. The bending and axial stiffnesses are equivalent to those of a full three-dimensional wall shown in Fig. 13b. The simplified wall model is the type used in the cyclic response analysis of the whole building (Fig. 13a).

Once the linear and nonlinear properties of main components and associated connections had been identified, a full structure model is assembled and subjected to a specified cyclic load. The goal is to obtain a load-deformation path at the roof/ceiling level (story shear). This load-deformation path is used to simulate the behavior of an entire story when subjected to a lateral cyclic load. The ceiling-level response of the house FE model under a simple “sawtooth” displacement history applied at the same level is shown in Fig. 13d (see inset plot for load history).

Once the FE model has been experimentally validated, the model parameters (related to material types and system configuration of the actual structure) that significantly affect response will be identified and various types of parametric and sensitivity studies (e.g., influence of material and joint characteristics, material or system substitution) will be conducted. Kasal et al. (1999) have demonstrated the application of a hybrid analysis method in determining the response of a one-storey wood-frame building to earthquake loads considering various sources of uncertainties.

The FE model was used to conduct preliminary analyses simulating the experimental setup. The structure was loaded at the ceiling level and displaced until a plateau in the
load-deformation curve was reached. The model was supported at locations of the load cells and reaction forces were calculated. The reaction forces and the load needed to fail the model were used in the design of the experimental setup and the placement of the load cells.

**Detailed Finite Element Model of Sub-Systems**

The sub-system models are based on work published by Kasal and Leichti (1992). In the original work, the models were massless and did not capture the hysteretic response to a cyclic load. The newly developed models, which include masses, can have load-history dependent behavior and bending stiffness is more accurately modeled by using stud bending properties rather than modeling it as an orthotropic continuum as was done in the earlier model.

An automated mesh generator developed in the past research was improved and individual nails can be modeled. The models are used to arrive from nail and material properties to the wall shear behavior. The analytical models were verified by comparing them with experiments performed by Karacabeyli (1997; 1999) in shear (Xu 1998). Verification of cyclic bending performance of the models is in progress (Collins and Kasal 1999).

The wall models are used to generate load-deformation characteristics of walls loaded in shear, bending or both. The output from wall analysis is used to generate input for the finite element model of the full experimental building. Load deformation characteristics of inter-component connections can be obtained either from experiments, detailed finite element models or both.

**Shear-Building Model**

The shear-building model is based on the assumptions that the girders on the floors (or the entire floor systems) are infinitely rigid as compared to the columns (i.e., in frame structures, rotation at the girder-to-column joint is suppressed), and that the rigid girders (or floor systems) remain horizontal during ground motion.

The differential hysteresis model, called the Bouc-Wen-Baber-Noori-Foliente (BWBNF) model, represents all the hysteretic response of the parts of the house between the roof and the foundation. The BWBNF model exhibits the following characteristics: (a) nonlinear, inelastic response, (b) stiffness degradation, (c) strength degradation, and (d) pinching (including slip). The shear building model that incorporates the BWBNF model has been previously used in nonlinear random vibration, Monte Carlo simulation and reliability analysis of structural timber systems (Foliente et al. 1998b, Foliente et al. 1999; 2000).

The model’s relative simplicity allows consideration of increased complexity on the loading side and in the nonlinear dynamic response. Overall dynamic responses to different ground motion inputs, recorded and/or simulated, are easily obtained. A shear-building model is not suitable to predict local or global failure mechanisms or damage levels in individual structural components, but is suitable for interpreting data measured from dynamic tests in the laboratory (e.g., shaketable tests) and from field measurements (e.g., instrumented buildings), and for conducting studies on the effect of soil conditions in the form of boundary conditions, higher mode effects, etc.

Results of shear-building model analysis of structures tested in the laboratory or field can be imposed on more detailed FE models to investigate local forces and response.
mechanisms. This will be done via reloading the more detailed models by boundary deformations and accelerations from the shear-building model analysis (Kasal et al. 1994; 1999). This will allow us to study load sharing/interaction and distribution, and to track the formation of local failure mechanisms in individual components of the full-scale buildings.

Summary and Conclusions

In this report, we reviewed the literature on house testing and modeling, and described our strategy to develop improved tools for performance analysis of light-frame buildings under wind and earthquake loads that can be used to improve current design procedures. We also presented details of the experimental program, test house and instrumentation, the current progress in the development of whole building models, and preliminary results of both experiments and analysis.

Most analytical and experimental work in the literature focuses on understanding the response behavior and mechanism at the sub-system level such as shearwalls. Recommendations arising from studies at this level typically ignore systems effects, and the fact that the actual forces that the sub-system sees depend primarily on the geometric and structural characteristics of the whole building. This project will provide us information and data that will be useful in filling critical gaps in existing knowledge.

At this point, we can conclude that:

- Measuring the loads at the base of the house with three-directional load cells provide very useful information that could contribute to our understanding of load sharing and distribution in a wood-frame building. Although further test data are needed, we can say that given the preliminary results that we have obtained, this is achievable.
- It is highly feasible that the three-dimensional model of the house can be experimentally validated with the test data we would obtain from testing of the full-scale house, and of it's constituent components and materials.

With these promising developments, we have started work on Phase 2 of the project. This involves continuation of the pseudo-elastic testing program and the destructive testing program. Obtained data will be used to develop a preliminary concept for a simplified “system-based” engineering design method for shear walls in wood buildings subjected to wind load.

References


**Acknowledgements**

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Appendix A Experimental Building Materials

Material Details

Framing:
Studs = 90x35mm MGP 10 (F5)* spaced @ 400mm centers
Lower Plate = 90 x 45mm MGP 10 (F5)
Top Plate = double 90 x 35mm MGP 10 (F5)
MGP 10 Characteristic Properties:
  Bending (90 x 35mm): 16 MPa
  Bending (90 x 45mm) 19 MPa
  Tension Parallel to Grain: 8.9 MPa,
  Shear: 5.0 MPa
  Compression parallel to grain: 24 MPa
  E: 10 000 MPa (short duration)
  G: 670 MPa (short duration)

* MGP 10 is similar to Southern Yellow Pine in density and general properties and is approximately equivalent to D-Fir standard grade.

Plywood:
2400 x 1200 x 9.5mm F11 Bracing Ply (Laid Vertically)
Complies with AS 2269-1994, Plywood – Structural
F11 Plywood Characteristic Properties:
  Bending: 11.0 MPa, Tension: 6.6 MPa, Shear 1.80 MPa
  E:10500 MPa, G: 525 MPa

Gypsum:
1200 x 2400 x 13mm Gypsum Board (Laid Horizontally )
Complies with AS 2588-1998, Gypsum Board
Gypsum characteristic Properties:
  Breaking force perpendicular to wrapped edge : 490 N
  Breaking force parallel to wrapped edge : 200 N
  Minimum nail pull resistance = 270 N

Fixing Nails: For use in framing
3.05 x 75mm D-head machine driven (Senco nail gun)

Sheathing Nails: For use on plywood
2.87 x 50mm D-head machine driven (Senco nail gun)
Spaced @ 150mm on perimeter, 300mm internally

Sheathing Screws: For use on Gypsum wallboard
6 gauge x 30mm Needle-point Type 1 Gypsum board Screws (Self-Driven)
Spaced @ 150 mm on studs with 150mm clearance to top and bottom plates ceiling
### Figure B1 - Load cell location and x,y co-ordinates

![Load cell location and x,y co-ordinates](image)

### Table B1 - Experimental Data

<table>
<thead>
<tr>
<th>Load Cell Name</th>
<th>Co-Ordinates</th>
<th>X reactions</th>
<th>Y reactions</th>
<th>Z reactions</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>(mm) (mm) (N) (N) (N) (N) (N) (N) (N) (N)</td>
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<td></td>
<td></td>
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<td>Actuator</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
1 Ton Push - W2

Reactions:
X direction

Max = 760 N
Min = -40 N
Reactions:
Z direction

Max = 1110 N
Min = -1450 N

1 Ton Push - W2
Self Weight

Reactions:
Z direction

Max = 2650 N
Sum = -49050 N
Appendix C Preliminary Component Testing Results

Gypsum Board to Timber Screw Connections

Test Materials:
Gypsum Board: 13mm standard core
Timber: 90 x 95mm F5 Radiata pine stud sections
Screws: Bugle head 6 x 30mm

Loading:
Monotonic – 2mm/min

Test Configurations
Five specimens tested for each of the following configurations:
Configuration 1: Field connection – machine direction
Configuration 2: Field connection – across direction
Configuration 3: Along recessed edge (15mm from edge)
Configuration 4: Across recess edge (15mm from edge)
These configurations are shown in Fig. c1.

Figure C1 - Typical gypsum board sheet showing location of gypsum board sections tested and direction of loading.

Table C1 - Results of tests on field connections

<table>
<thead>
<tr>
<th>Test</th>
<th>Machine direction (Config. 1)</th>
<th>Across direction (Config 2)</th>
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<tbody>
<tr>
<td></td>
<td>$F_{\text{max}}$ (N)</td>
<td>$\Delta_u$ (mm)</td>
</tr>
<tr>
<td>1</td>
<td>510</td>
<td>10.8</td>
</tr>
<tr>
<td>2</td>
<td>494</td>
<td>5.0</td>
</tr>
<tr>
<td>3</td>
<td>533</td>
<td>9.0</td>
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<tr>
<td>4</td>
<td>501</td>
<td>17.0</td>
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<td>5</td>
<td>628</td>
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<tr>
<td>Average</td>
<td>533</td>
<td>10.0</td>
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</tbody>
</table>
## Table C2 - Results of tests on recessed edge connections

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<thead>
<tr>
<th>Test</th>
<th>Machine direction (Config 3)</th>
<th>Across direction (Config 4)</th>
<th></th>
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</thead>
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<tr>
<td></td>
<td>$F_{\text{max}}$ (N)</td>
<td>$\Delta u$ (mm)</td>
<td>$F_{\text{max}}$ (N)</td>
<td>$\Delta u$ (mm)</td>
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<tr>
<td>2</td>
<td>763</td>
<td>6.2</td>
<td>526</td>
<td>2.2</td>
</tr>
<tr>
<td>3</td>
<td>906</td>
<td>5.2</td>
<td>696</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>536</td>
<td>5.6</td>
<td>520</td>
<td>2.4</td>
</tr>
<tr>
<td>5</td>
<td>702</td>
<td>9.2</td>
<td>594</td>
<td>1.9</td>
</tr>
<tr>
<td>Average</td>
<td>745</td>
<td>6.4</td>
<td>580</td>
<td>2.4</td>
</tr>
</tbody>
</table>

Figure C2 - Definitions of $\Delta u$: Ultimate displacement at rupture (or slip) or at 0.8 $F_{\text{max}}$, whichever may occur first.

## Observations and summary of results

- There seems to be little difference between field connections loaded in the machine direction (config 1) and the across direction (config 2).
- Failure mode for field connections (config 1 & 2) is tearing slot in the gypsum board and bending of screws.
- Screws loaded along the edge in the recessed part (config 3) had higher load carrying capacity compared to corresponding field connections (745N compared to 533N).
- Connections in config 3 failed at lower displacement than corresponding field screws (config 1) (6.4mm compared to 10mm). The connections in config 3 failed in a unique way – shearing of the screws.
- Connections in config 4 failed at the lowest displacement due to tearing out of the gypsum board edge, however the maximum load was comparable to field connections.
Appendix D Experimental building CAD drawings
W2 Studs
Plaster begins 13mm from ends.
Truss joint
90mm X 35mm cutout on each end of both beams
Roof Ply 2