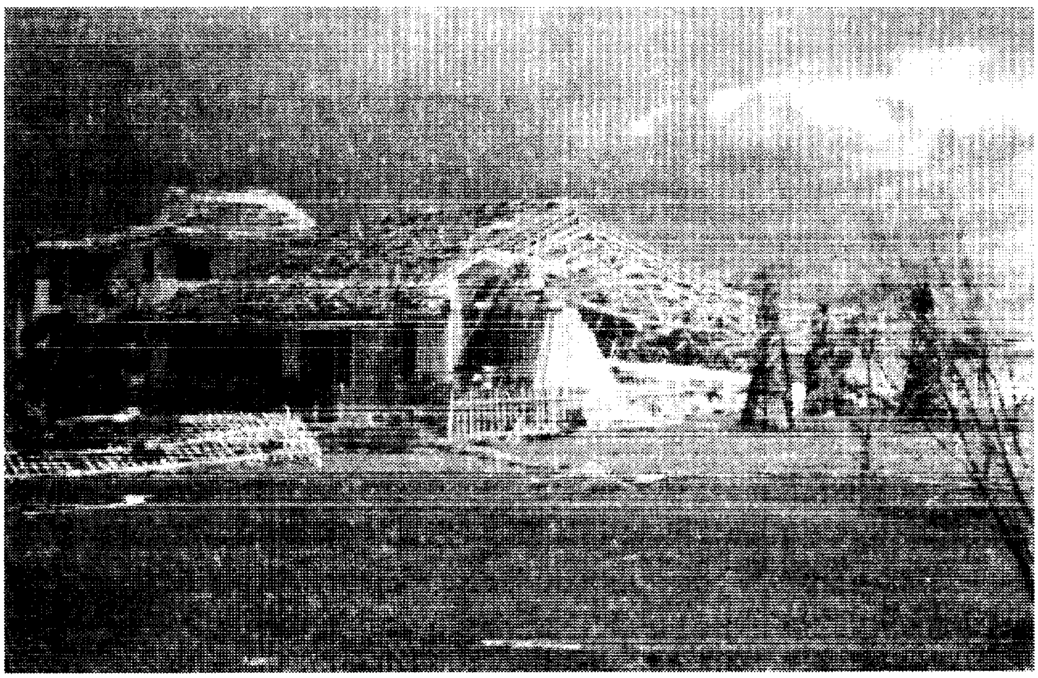
NISTIR 6224

A Summary of the Structural Performance of Single-Family, Wood-Frame Housing



Building and Fire Research Laboratory
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United States Department of Commerce
Technology Administration
National Institute of Standards and Technology

Cover photo: Roof structure failure in Hurricane Andrew. Courtesy of the Federal Emergency Management Agency.

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ABSTRACT

A review of the structural performance of single-family, wood-frame houses is presented. The performance of these structures in selected earthquakes and hurricanes is summarized. In general, wood-frame houses performed well with relatively few instances of structural failures and without serious loss of lives. Failures mainly resulted from poor construction practices and noncompliance with building codes. The issue of repair costs due to non-structural damage suggests that different performance objectives are needed for residential structures.

The review of experimental research covers studies on full-scale houses, shear walls and intercomponent connections. The database for full-scale house tests is sparse while there are numerous studies of shear walls and only a limited number of studies on intercomponent connections. Test methodology for shear wall tests has evolved from prevalent use of ASTM Standard E72 to more frequent use of ASTM Standard E564-76. Within the last ten years there have been adaptations of E564 for cyclic and dynamic test procedures: however, the wide variation in test procedures for cyclic and dynamic tests points to a need to standardize these procedures.

The development of analytical procedures to predict component and structural behavior under different types of loading is also presented. It is found that while significant progress has been made in recent years in analytical modeling of shear walls and horizontal diaphragms, only limited progress has been made on the modeling of a complete house and intercomponent connections. Most of these analytical procedures are used primarily as research tools rather than design tools.

Based on this review, a multi-year research program is proposed to determine the baseline performance of single-family houses.

Keywords: Analytical tools; building technology; computer modeling; earthquakes; experiments; houses; hurricanes; performance; single-family houses; structural behavior; wood-frame construction.

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1.0 INTRODUCTION

The National Institute of Standards and Technology (NIST) project, *Structural Performance of Housing Systems*, is a research component under the Building and Fire Research Laboratory's (BFRL), Performance Standards System for Housing (PSSH) major objective. PSSH supports industry coordinated research and the development of national and international performance standards to guide the design, specification, evaluation, and acceptance of innovative housing products and systems. The *Structural Performance of Housing Systems* project was initiated in response to the *Residential Sector Strategic Approach*, a report prepared by the National Association of Home Builders (NAHB) Research Center. This report addresses the National Construction Goals established by the Subcommittee on Construction and Building of the National Science and Technology Council. In addition, this research project and PSSH support the Office of Science and Technology Policy's program *Partnership for Advancing Technologies in Housing* (PATH). PATH is a joint effort between the private and public sectors with objectives which include: increased durability, lower construction costs, reduced disaster losses, and acceleration of the development and market acceptance of new housing technologies.

Total construction in the United States in 1997 amounted to $577 billion or approximately 7% of the Gross Domestic Product. Approximately 30% of this amount was spent on new residential (single-family) construction. Wood-frame construction constitutes the majority of single-family houses as wood is an economical construction material. In general, wood-frame houses perform well under gravity loads. Substantial damage and economic losses, however, have resulted from natural disasters such as earthquakes and hurricanes. Therefore, significant economic and societal benefits may be realized by the development of methodologies to better predict and evaluate the performance of single-family, wood-frame houses subjected to extreme lateral loads. In spite of the extensive use of wood, wood-frame houses are being designed "on a basis of ignorance rather than on the basis of knowledge and understanding" (Diekmann, 1994).

In the past, the performance of single-family, wood-frame houses has been assessed using the performance of individual components determined analytically or experimentally. However, very few studies have been conducted on complete houses. Analytical and experimental studies on complete housing units are necessary to understand the load path through a structure, the load distribution among various components, the contribution of the transverse walls and interior partitions to the lateral load resisting capacity, and possible torsional effects due to unsymmetric configurations.

The objective of the NIST project is to establish the baseline structural performance in terms of strength and ductility of single-family, wood-frame houses subjected to lateral loads. The baseline performance is necessary for the development of performance based design criteria for residential houses currently being developed by the ASTM Subcommittee E6.66, Performance Standards for Buildings. In addition, the baseline performance will serve as the benchmark for comparing the performance of houses constructed with non-traditional materials or different construction technologies. The use of materials such as concrete, light gage steel, or composites in residential housing, which is currently very limited, may result in improved performance.

The NIST project will initially develop baseline performance for wood-frame houses using an analytical model. The analytical model will include the interaction of various components such as walls, roofs, floors, intercomponent connections, etc. Where necessary, experimental work will be conducted to verify or provide data for the analytical model. Once the performance criteria for typical traditional housing units are established, the model will be used to examine houses constructed with non-traditional construction materials or techniques.

The scope of the project includes: 1) conducting a literature review to summarize the structural performance of wood-frame houses and housing components and to identify research needs, 2) conducting 3-dimensional (3-D) analytical studies of complete houses, 3) performing component tests to provide data as needed for the analytical studies, and 4) performing studies to determine the feasibility of using non-traditional construction materials.

This report summarizes a literature review on single-family, wood-frame houses. The opinions, findings, conclusions, and recommendations are those of the cited authors unless stated otherwise. The performance of such houses in past earthquakes and hurricanes is presented in Chapter 2. Chapters 3 and 4 summarize the experimental and analytical research, respectively, that have been conducted on different structural components such as walls, floors, roofs, intercomponent connections, and complete housing units. A summary is presented in Chapter 5 which also includes recommendations for further analytical and experimental work and a multi-year plan for the completion of the NIST project.

2.0 PERFORMANCE IN PAST EARTHQUAKES AND HURRICANES

2.1 INTRODUCTION

Four recent events – the 1989 Loma Prieta and 1994 Northridge earthquakes, and the 1989 Hugo and 1992 Andrew hurricanes – have shown that wood construction performed well in terms of life safety. Buildings that have performed well have had simple rectangular configurations, continuous floors, and small window and door openings. However, because of the large number of single-family, wood-frame houses in the United States, damage to these structures in extreme load events constitute a large percentage of the economic loss. To reduce earthquake and hurricane damage to single-family, wood-frame structures, a better understanding of the performance and behavior of these structures in such events is essential.

Even though building codes contain wind and seismic provisions, the structural behavior of houses subjected to lateral loads is not fully understood and many uncertainties still exist. Although earthquakes and hurricanes are catastrophic events, they do provide the engineering community with rare opportunities to learn more about the behavior of full-scale structures subjected to lateral loads. Damage surveys after such events greatly increase the existing knowledge of structural behavior and this is reflected in revisions to the building codes as shown in Table 2.1 (note the increase in seismic design coefficient and decrease in the allowable shear loads after significant earthquakes).

Table 2.1 History of Code Changes in the Los Angeles City Code (after Holmes and Somers, 1996).

Year of Code	Seismic Design Coefficient[1]	Allowable Shear Loads (lb/ft)		
		Plywood[2]	Stucco[3]	Drywall[4]
1956	0.092	355	200	-
1962	0.133	355	200	125
1966	0.133	355	200	125
1970	0.133	360	200	125
1972	0.133	360	200	125
1976	0.186	360	180	125
1980	0.186	360	180	125
1985	0.140[7] / 0.186[8]	360[5]	180	125
1991	0.138[7] / 0.183[8]	360[5]	180	62.5
1994[6]	0.138[7] / 0.183[8]	200	90	30

[1] Seismic design coefficient (base shear/building weight) for a two-story wood building
[2] 1-inch[1] top grade Douglas Fir, 8d nails at 102 mm (4 in) blocked, 2 framing
[3] 1 inch Portland cement plaster with metal lath
[4] 1-inch gypsum wallboard, blocked with nails at 178 mm (17 in) or unblocked with nails at 102 mm (4 in)
[5] 6.3 kN/m (432 lb/ft if face grain laid across studs and stud spacing not exceeding 406 mm (16 in)
[6] Post-Northridge earthquake
[7] Seismic design coefficient for a two-story plywood building
[8] Seismic design coefficient for other type of two-story building
Note: 25.4 mm = 1in, 1 lb/ft = 14.59 N/m

A brief background discussion on lateral loads and types of failures and wood properties is presented in Section 2.2. Summaries of the performance of wood-frame structures in selected earthquakes and hurricanes are presented in Sections 2.3 and 2.4. In cases, where terminology or description was not precise or was unclear, an interpretation was made to clarify the situation. The reader is referred to the cited reference for more details or further clarification. Findings, conclusions, and recommendations are those of the authors of the cited references based on their observations during their damage assessments. Sections 2.5 and 2.6 summarize conclusions and recommendations gathered from the various references.

2.2 BACKGROUND

2.2.1 Earthquake vs. Hurricane: Loads and Failures

Although both hurricane and earthquake loads are considered lateral loads, the manner in which they are transmitted to the structure is different. Wind forces cause the roof and walls to be loaded first and these loads have to be transferred to the foundation with failures first occurring at the weakest link. Wind loads on low-rise wood structures may be treated as static

[1] All dimensional lumber is described with nominal dimensions in U.S. Customary units and a hard conversion to SI units.

loads as most of the energy from the wind speed fluctuations is distributed at frequencies lower than the natural frequency for wood structures [1.2 Hz to 18 Hz (Foliente, 1997)]. On the other hand, seismic motion is transmitted from the foundation to the structure, creating inertial forces which have to be transferred back to the foundation. Unlike wind loading, seismic loads are considered dynamic loads as the predominant frequencies of earthquake motion is within the range of those for low-rise wood structures as shown in Figure 2.1.

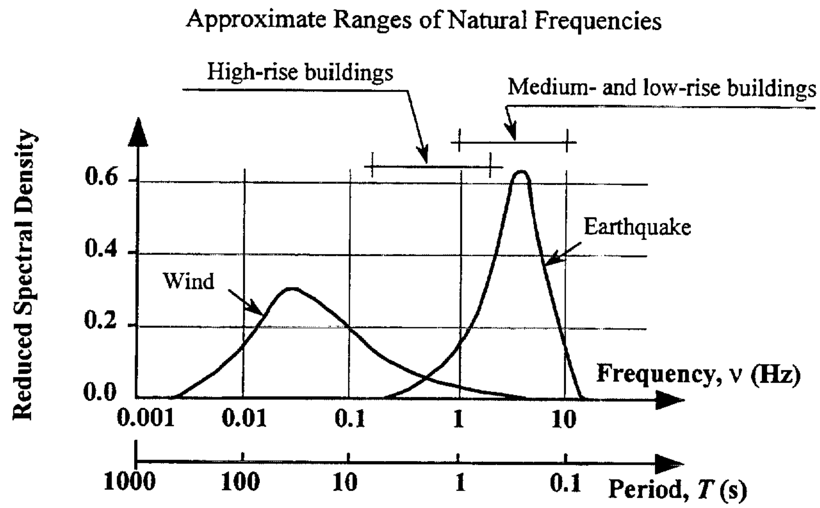

Figure 2.1 Ranges of Natural Frequencies (after Foliente, 1994).

While some types of failures are common to hurricanes and earthquakes, others are not because of the differences in loading. Hurricane damage results from wind loading and/or wave action (sites along the water front) which usually cause foundation and roof failures. In addition, water damage from flooding and torrential rains usually associated with hurricanes significantly contribute to the economic losses in hurricanes. Damage from earthquakes generally results from either insufficient lateral support of cripple walls (short stud walls between the foundation and the first floor but may be as high as one story, see Figure 2.2), inadequate lateral resistance of shear walls, poor soil conditions or a combination of these conditions. Failures that are common to earthquakes and hurricanes are failures of intercomponent connections or connections between the different components (e.g. wall-to-wall, roof-to-wall, wall-to-foundation), and intracomponent connections or connections within subassemblages.

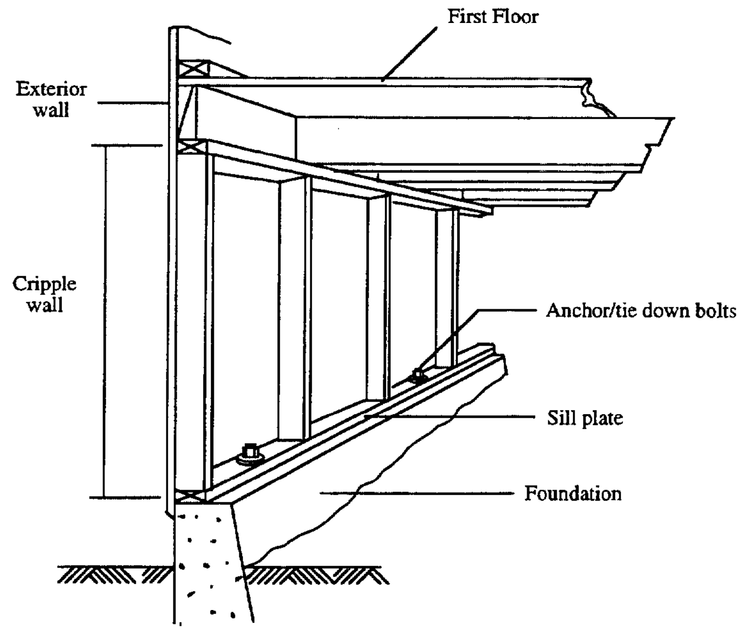

Figure 2.2 Cripple Wall (after Benuska, 1990).

The preceding paragraphs provide a very brief overview on lateral loads and types of failures, but they illustrate the importance of a continuous load path and the important role of adequate connections. These concepts are further reinforced in the findings by the various damage assessment teams presented later in this chapter.

2.2.2 Wood Properties

The strength of wood varies depending on its species, density, grade, type and direction of loading, duration of loading, moisture content, etc. Properties of wood that are of primary interest in the design of structures are usually the strength and modulus of elasticity. Wood is an anisotropic material with vastly differing properties in its three orthogonal axes. The axial tensile strength of wood (parallel to grain) is up to 50 times greater than the strength in its transverse (perpendicular to grain) direction (Tsoumis, 1991). The axial tension strength varies from 50 MPa to 160 MPa (7,250 psi to 23,200 psi) and the transverse tensile strength ranges from 1 MPa to 7 MPa (145 psi to 1015 psi). The axial compressive strength of wood is up to 15 times greater than the compressive strength in the transverse direction. Axial compressive strength of wood ranges from 25 MPa to 95 MPa (3,625 psi to 13,775 psi) and the transverse compressive strength ranges from 1 MPa to 20 MPa (145 psi to 2900 psi). The modulus of elasticity along the grain varies from 6,890 MPa to 13,780 MPa (1 to 2 x 10^6 psi) (Soltis et al., 1981).

Soltis et al. (1981) characterized wood as a viscoelastic material with good damping characteristics. They also reported that wood can resist short duration loads up to twice its design strength. As compared to the full design load for a 10-year duration, ultimate strengths are approximately 60% higher for 5-minute load duration and 90% higher for a 5-second load duration. Building codes allow a 33% increase in allowable stress for seismic loads which is based on the full design load for a 10-year duration.

Similarly, Liska and Bohannan (1973) noted that wood structures perform well under shock and fatigue loading because their mechanical joints can absorb short duration shock loads approximately twice the magnitude of their design loads. Further, wood is less sensitive to repeated loads than crystalline structural materials. This property, therefore, makes wood a good material for resisting wind and seismic loading. A wide variety of mechanical fasteners may be used to connect wood elements, and these fasteners allow slight deformations of the fasteners without decreasing the integrity of the connection. These fasteners can, therefore, be designed as energy dissipating devices and used to reduce the energy imparted to the structure.

2.3 EARTHQUAKE PERFORMANCE

Houses constructed prior to 1940 (pre-adoption of nominal seismic resistant standards) had wood floors supported on substructures which had little or no lateral bracing (Jephcott and Messinger, 1991). These houses lacked mechanisms to transfer horizontal forces from the structure to the plain or unreinforced masonry foundation. However, some wood houses built prior to the inclusion of seismic requirements survived strong earthquakes (e.g. 1906 San Francisco earthquake, Magnitude 8.3 and the 1964 Alaska earthquake, Magnitude 8.6). Newer houses are also susceptible to earthquake damage as evidenced by the damage in more recent earthquakes such as the 1994 Northridge earthquake (Magnitude 6.8). Poor construction practices and inadequate detailing and inspection were cited as possible reasons. Wood buildings have generally performed well in earthquakes mainly due to their light mass, redundancy from non-structural elements such as partitions and ceilings which add to the building's stiffness, ductility, and energy absorption of joints when properly connected (Foliente, 1995). Connection failures and noncompliance with good construction practices or building codes have generally been responsible for most structural failures.

Wood structures are divided into two categories in the Uniform Building Code, UBC (ICBO, 1997a):

1. Non-engineered or conventional structures: one-, two- or three-story, single-family, houses; apartments; or condominiums. These structures are constructed using the "General Construction Requirements" and the "Conventional Construction Requirements". These prescriptive requirements are based on engineering judgment and past experience.
2. Engineered structures: light-frame structures of unusual size, shape or split level, buildings with concrete or masonry walls, public schools, and hospitals.

The performance of non-engineered, single-family, wood-frame houses in selected seismic events are summarized in the following sections.

2.3.1 Anchorage, Alaska - March 27, 1964

The Anchorage, Alaska earthquake of March 27, 1964 measured 8.6 on the Richter scale (Anderson and Liska, 1964). Reported ground subsidence and uplift of approximately 300 mm (1 ft) to several meters occurred. Anderson and Liska found that well constructed wood buildings sustained minimal damage. In general, damage was caused by the structure not acting as a unit due to failed connections or by inadequate lateral resistance.

Although ground subsidence left several houses partially supported, the wood-frame houses were rigid enough not to sustain significant deflection or damage. In areas of subsidence, concrete or masonry foundation walls were destroyed whereas the upper wood-frame stories sustained little damage.

Wood floor systems performed well with little damage[2]. Even when the basements or foundation walls were partially destroyed, the wood floors were strong enough to support the weight of the house in such situations. In some instances, insufficient fastenings of the floor to the first-story wall allowed the floor to pull away from the wall. In contrast to wood floor systems, concrete slab floors were usually destroyed when the ground under the structure subsided.

Proper construction of walls (braced wall frames, floor and roof systems properly fastened together and well nailed sheathing) minimized the damage to structures and provided the necessary strength for them to resist earthquake forces and landslides. In general, plywood sheathing performed well and the lateral resistance of plywood was found to be better than that of fiberboard. Fiberboard sheathing [20 mm (25/32 in) thick] when properly installed performed well except in high stress areas where shear failures of the sheathing between studs occurred. Improper installation of the fiberboard caused nailhead pullthrough or shearing at the nails which led to severe racking of the walls. Horizontal wood sheathing with let-in bracing was common in older houses and performed well as did wood sided walls, panelized walls with plywood or paper-overlaid plywood, and wood sheathing with a stucco (Portland cement plaster) overlay. Single covering material such as drop siding without sheathing did not have sufficient lateral strength to resist the applied forces. Masonry veneers, brick and concrete block, failed in some houses, but the wood-frame and sheathing beneath it were not damaged. Proper connections between interior partitions (non-structural) and the exterior walls would have added to the stiffness of the structure. Windows and openings near corners also reduced the rigidity of the structure significantly.

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[2] The term "performed well" was commonly used in many of the cited references but was not explicitly defined. It is assumed in this report to mean that the element or system performed its intended function in the majority of cases with failures being uncommon.

Roofs were found to perform well even when subjected to severe ground subsidence. Damage to chimneys varied from minimal to total collapse.

Based on the observed damage, Anderson and Liska (1964) suggested the inclusion of an energy dissipating connection between the basement wall and the floor system to reduce the forces transferred from the basement (foundation) to the house (superstructure). They, also, noted that the integrity of the structure resulted from good corner connection of exterior walls, proper nailing of siding and sheathing in addition to the use of both materials (siding and sheathing), and adequate connections. Nail connections offer adequate stiffness and energy dissipation and were better than rigid mortar joints.

The findings of Soltis, et al. (1981) were similar to those of Anderson and Liska (1964). Failures were due to the houses not acting as integrated units or to inadequate lateral bracing. Houses which sustained damage had inadequate corner connections, insufficient ties between the floors and walls and between the roof systems and the walls, lack of sheathing, and wall openings especially near the corners.

2.3.2 San Fernando, California - February 9, 1971

The San Fernando earthquake occurred on February 9, 1971 and it measured 6.6 on the Richter scale. The estimated building damage (both commercial and residential excluding mobile homes) was in the order of $167 million (1971 dollars) for the city of Los Angeles and Los Angeles County (Lew et al., 1971). There were approximately 300,000 wood-frame dwellings in the San Fernando Valley of which about 5% were located in the region of heaviest shaking (Steinbrugge et al., 1971).

A survey of 12,000 single-family wood-frame houses was conducted by the Pacific Fire Rating Bureau (Steinbrugge et al., 1971). Most of the dwellings were constructed within the two decades prior to the earthquake. Typical types of foundations were either slab on-grade or continuous concrete foundation around the perimeter with concrete piers in the interior with the former being more common. The majority of the houses were single-story. The construction characteristics of the sampled houses are given in Table 2.2.

Table 2.2 Construction Characteristics of Sampled Wood-Frame Dwellings (after Steinbrugge et al., 1971).

Dwelling Age	Pre-1940	5%
	1940-1949	38
	Post-1949	57
Dwelling height	One story	94
	Two story	2
	1 & 2 story (one part one story, other part 2 story)	3
	Split level	0.5
Floor construction	Wood Joist	40
	Slab on-grade	60

The survey showed that within the region of most intense shaking, 25% of the wood-frame dwellings sustained losses greater than 5% of the dwelling's value with the remainder sustaining smaller losses. The number of houses with damage above the 5% threshold is equivalent to 1% of all the wood-frame dwellings in the San Fernando Valley. The various levels of damage to different components of the houses are given in Table 2.3. As seen in Table 2.3, most of the damage was non-structural. There was no difference in performance between houses with slab on-grade floors and those with wood joist floors. One-story houses performed better than two-story houses with 1 & 2 story (see Table 2.2 for definition) houses performing the worst among the three types. McClure and Messinger (1973) also found this to be true. For this earthquake, houses built after 1940 performed better than those built before 1940. Approximately 30% of the chimneys sustained damage with 14% sustaining moderate to severe damage and the rest sustaining slight damage. The good chimney performance was attributed to the use of reinforcing steel.

Table 2.3 Type of Damage of Surveyed Houses (after Steinbrugge et al., 1971).

Construction Component	Level of Damage[1]			
	None	Slight	Moderate	Severe
Foundation	91.9 %	5.8 %	1.6 %	0.7 %
Damage to frame	78.8	16.0	3.3	1.9
Interior finish - plaster	4.2	78.4	11.1	6.3
Interior finish - gypsum board	12.1	78.0	6.5	3.4
Exterior finish - stucco (plaster)	20.7	74.1	4.0	1.2
Brick chimney damage[2]	67.6	16.1	6.6	7.4

[1] Slight - minor cracking of stucco finish along discontinuities or at window corners or enlargement of old cracks
Moderate - damage that is not slight nor severe
Severe - separation of wood-frame from foundation, roof or walls at incipient failure, loose interior plaster or spalled plaster with extensive cracking of walls and ceiling, interior gypsum board had to be replaced or retaped at joints.

[2] 2.3% of chimneys had total brick damage and were not repairable.

As described by Lew et al. (1971), most single-family residences were one-story with the roof sheathing either of plywood or 25 mm (1 in) wide boards. Roof coverings varied and included built-up asphalt impregnated asbestos material, asphalt shingles, wood shingles or shakes, and clay tile. Exterior wall cover was either cement plaster, wood siding, or plywood. Either gypsum or plaster on gypsum board lath was used in the interior.

Failures occurred most frequently in split level or irregularly shaped houses due to inadequate connections at changes in roof or floor elevations. Failures also occurred due to large door openings for garages. Damaged masonry chimneys were also found to be common.

McClure and Messinger (1973) also found that most of the damage was non-structural damage. Other findings by McClure and Messinger included: 1) wood buildings performed well to meet life safety requirements, 2) overall building damage was due to inadequate bracing of the walls due to excessive door and window opening, 3) exterior finish materials listed in order of better performance - plywood, vertical and horizontal siding, brick and stone veneer, and stucco, and 4) interior finish materials listed in order of better performance - plywood, gypsum board, and gypsum lath and plaster. They recommended that the lateral bracing of houses over one-story should not be provided by the use of wall finish materials combined with conventional 1 x 4 nominal let-in bracing.

2.3.3 Loma Prieta, California - October 17, 1989

The Loma Prieta earthquake occurred on October 17, 1989 in the San Francisco Bay region and it measured 7.1 on the Richter scale (Lew, 1990). Property damage was estimated at over $6 billion and over 12,000 people were displaced from their homes.

A survey of the damage to wood-framed structures was conducted by a group of three engineers from the American Plywood Association (APA) (Tissell, 1990). Their main findings were:

1. Damage was caused by failure of cripple walls. The failures of cripple walls were the result of inadequate nailing of plywood sheathing. When adequate nailing was provided, no failure was observed.
2. Lack of connection between the major framing members and the foundation was the cause of failure of two severely damaged houses.
3. Damage caused by soft stories was observed in the Marina District. The phenomenon of soft stories, first observed in this earthquake, results from garage door or large openings on the ground floor of apartment buildings and houses which reduces the lateral resistance of that story. The reduced lateral resistance causes severe racking to occur or increases lateral instability.
4. Chimney damage was common. Chimneys were typically unreinforced and not sufficiently tied to the structure.
5. Upward ground movements caused doors to be jammed and damage to basement floors.
6. Post-supported buildings were damaged because of inadequate connections of the floor to the post foundation and unequal stiffnesses of the posts due to unequal heights. Houses where the poles were diagonally braced were not damaged.

Some of conclusions from the APA survey were:

1. All wood-frame buildings damaged were either built before 1973, had critical construction features with inadequate connections, or did not comply with minimum code requirements.
2. Structures built to current code provisions performed well with no code deficiency observed. However, greater attention is needed in the design and installation of connections and fasteners and to structural continuity and integrity.

Corbeen (1996) also found that low-rise, wood-frame structures built according to modern building codes performed well. Structures with problems were built prior to the 1949 UBC requirements and lacked connections between roof, walls, floors, and foundation. Significant cracking of stucco and brick veneer finishes was also observed. Corbeen proposed the following retrofits:

1. Install energy dissipating assemblies in the lower story to absorb the imparted energy. One simple and inexpensive way of increasing energy dissipation is to increase the number of nails. This, however, would require inspection for quality assurance.

2. Replace brittle finishes with more flexible finishes such as paneling or siding. The use of flexible fastenings to attach gypsum board to framing would reduce damage to the boards.

Research is, however, needed to investigate the use of fiber mesh over gypsum board surface to reduce damage and to investigate the use of reinforced gypsum board to make it less brittle.

2.3.4 Northridge, California - January 17, 1994

An earthquake with a magnitude of 6.8 struck the Northridge community in the San Fernando Valley on January 17, 1994. The effects of this earthquake were felt over the entire Los Angeles region. Approximately 65,000 residential buildings were damaged with 50,000 of those being single-family houses (HUD, 1995). The estimated damage based on insurance payouts was over $10 billion (Holmes and Somers, 1996) for single- and multi-family residences.

A damage survey was conducted by the National Association of Home Builders (NAHB, 1994) for the U. S. Department of Housing and Urban Development (HUD). Two categories of buildings were examined: 1) single-family, detached buildings and 2) single-family, attached and multi-family, low-rise (two stories or less) buildings. The following discussion will only consider the first type of buildings and any reference to houses or buildings refers to single-family, detached buildings. The survey was conducted so that a statistical sampling was possible. The surveyed area consisted of randomly selected postal regions within a 16.1 m (10 mi.) radius of the epicenter.

Of the houses examined (341 total out of 183,514 in the surveyed area), about 90 percent were built before the 1971 San Fernando earthquake and approximately 60 percent were built during the 1950s and 1960s. Most of the houses were built to prescriptive specifications known as the LA City "Type V" construction common in the 1950s and 1960s. This type of construction is now only used for non-habitable buildings such as detached garages. This type of construction allowed several methods to provide the necessary bracing, and in practice, stucco was commonly used to provide the lateral resistance. Also, the use of wood roof rafters and plaster in the interior was common.

Of the sampled houses, all of the houses had exterior wood-framing and were typically one story (79%). An exterior finish of stucco was typical and two-thirds of the houses had an attached garage. Homes on crawlspace foundations outnumbered homes on slab on-grade by 2 to 1. Crawlspace foundations of full height concrete or masonry walls were typical and cripple wall foundations were much less common. Roof sheathing using boards was more typical than using plywood or oriented strand board (OSB). Approximately, half of the houses were rectangular in shape and the other half was irregular. The characteristics of the surveyed houses are given in Table 2.4.

Table 2.4 Characteristics of 341 Surveyed Houses (after NAHB, 1994).

Year Built	1970 or before	88%
	1971 or later	12
Stories	One	79%
	Two	18
	One-and-a-half	1
	Three or more	2
Shape	Rectangular	41
	Irregular	59
Exterior Finish	Stucco mix	50
	Stucco only	45
	Wood siding	5
Interior Finish	Plaster	60
	Gypsum board	26
	Other	1
	Unknown	13
Exterior Framing	Wood	99
	Other	1
Wall Sheathing	None	80
	Plywood	7
	Unknown	13
Roof Framing	Woof rafter	87
	Wood truss	5
	Other	5
	Unknown	3
Roof Sheathing	Board	69
	Panel - Ply or OSB	16
	Other	3
	Unknown	12
Foundation	Crawlspace - stem wall	68
	Crawlspace - cripple wall	3
	Slab on-grade	34
	Other	5

The overall performance of single-family houses was found to be good - even those built to the less stringent prescriptive Type "V" requirements. Damage to structural elements such as foundations, wall framing, and roof framing occurred for only a small percentage of the houses and the damage sustained was minimal. A small number of houses (2% of the housing population) with moderate or major damage were located in regions where ground fissures or ground settlement occurred or on hillsides where damage occurred in the foundation. It was also observed that houses on hillsides with columns supporting the house on the downhill side and concrete foundation on the uphill side were prone to collapse. Fissures and ground settlements caused cracks in the slab on-grade foundations. Also, inadequate bracing of cripple walls allowed racking to occur.

Damage to the interior and exterior finishes was more common and occurred in about 50% of the houses. More resilient finishes such as wood panel or lap board siding sustained less or no damage as compared with more brittle finishes such as stucco. Exterior finishes of houses with slab on-grade foundations were damaged in 30% of the cases while the occurrence increased to 60% for houses on crawlspace foundation. It was suggested by the authors that one possible reason for this difference because houses with slab on-grade foundations are situated on the ground and were newer than houses with crawlspaces.

Walls with large openings were damaged due to racking, and openings near wall corners increased the potential for damage. Wall damage was rated in the low category and was limited to 2% of all walls surveyed. Among the surveyed houses, structural damage to the roof, other than damage caused by masonry chimneys, was rare (less than 1%). Damage to masonry chimneys was common and varied from minimal to collapse. Chimney movement caused localized damage to the interior, exterior, and roof framing. Prefabricated wood chimneys with metal flues performed well. The rare occurrences of structural roof damage were due to alterations which modified, removed, or overloaded rafter ties.

The general finding by NAHB was that single-family houses built after the mid-1970s performed well unless they were located on sites with poor soil conditions.

Another damage survey conducted by the Residential Buildings Cripple Wall Subcommittee of the City of Los Angeles, Structural Engineers Association of Southern California (SEAOSC) (1994) found that cripple wall bracing failures occurred due to inadequate connection of the exterior finish which served as the lateral bracing. Single-story and multi-story residences with heavy tile or concrete tile roofing sustained significant damage. Houses whose lateral bracing consisted of horizontal wood siding or stucco sustained the most damage. A major cause of damage to stucco sheathed buildings was insufficient embedment of stucco in the wire lath.

The survey also found that damage to houses not bolted to the foundation was caused by sliding of the structure and failure of the cripple wall bracing system. When sill bolts were present but wall bracing failed, some sill plates split longitudinally from the bolt. This splitting was likely caused by oversized bolt holes.

To minimize seismic damage, the SEAOSC Cripple Wall Subcommittee (1994) recommended some changes in the code for new construction:

1. Include reinforcement in top and bottom of footing.
2. Sill bolt requirements:
   a. Decrease prescriptive bolt spacing and increase bolt size for 2 and 3 story wood construction.
   b. Require minimum 229 mm (9 in) and maximum 305 mm (12 in) edge distance for bolt at end of sill plate.
   c. Require square plate washers instead of round cut washers to reduce sill plate splitting.
3. Stucco requirements:
   a. Prohibit staples for attaching self-furring wire lath.
   b. Prohibit stucco as bracing for cripple walls for new construction over one story high.
   c. Limit the maximum shear strength of stucco to 1.3 kN/m (90 lb/ft). The value should be verified by conducting dynamic testing of walls braced with stucco using various lath and attachment methods.
4. Increase minimum length of individual panels of cripple wall bracing to make the prescriptive conventional bracing requirements consistent with the analytically determined requirements for seismic zone 4.

Other recommended code changes (Holmes and Sommers, 1996) based on the observed damage in the Northridge earthquake were:

1. Require footing reinforcement for single-family houses and duplexes - regardless of soil type.
2. Determine experimentally the cyclic strength of plywood, stucco, and drywall to verify the values given in the building codes. Until such values are available, recommended values were:
   a. 1.3 kN/m (90 lb/ft) for stucco. A 50% reduction of the code specified value.
   b. 0.4 kN/m (30 lb/ft) for drywall. Code specified shear strengths vary from 0.7 kN/m to 1.1 kN/m (50 lb/ft to 75 lb/ft)
   c. 2.9 kN/m (200 lb/ft) for 10 mm (3/8 in) thick plywood. Prohibit use of plywood thinner than 10 mm (3/8 in). Code specified values vary from 3.4 kN/m to 8.9 N/m (230 lb/ft to 610 lb/ft) for panels applied directly to framing (ICBO, 1997a).
   d. 25% reduction of strengths for plywood thicker than 10 mm (3/8 in).
3. Set maximum stud spacing to 406 mm (16 in) for plywood shear walls.
4. Enforce code-required use of common nails. Box nails which have lower capacities than common nails were usually used in place of common nails.
5. Address nail-overdriving problem through inspection, training, and use of attachments to nail guns to prevent overdriving. 50% of plywood shear walls were observed to have overdriven nails.
6. Restrict maximum height-to-width ratio of plywood-sheathed walls to 2:1

7. Limit column drift to 1/200 and use realistic k[3] factors (2.1 for vertical cantilever case) to check column stability criteria and restrict the cantilevered length to 25% of the distance between the lines of lateral resistance. These requirements would alleviate the stability problem with soft stories.

In contrast to the surveys presented above which focused on structural damage and life safety issues, Russell (1996b) presented the case of post-1950 buildings which do not have the same weaknesses as the pre-1940 ones and were considered to have met the life safety requirements after an earthquake but sustained sufficient nonstructural damage to render repairs uneconomical. The performance in the 1994 Northridge earthquake of a two-story dwelling built in 1958 with an addition added in 1972 was presented as a case study.

The dwelling was conventionally framed. The exterior of the dwelling was Portland cement plaster (stucco) and the interior and ceilings were gypsum lath and plaster. The continuous perimeter foundation was unreinforced concrete. The dwelling did not have cripple walls and the first floor joists were supported directly on the sill plate which was bolted to the foundation. The dwelling had a vertical irregularity as the second story was recessed from the line of the first story exterior walls and was enclosed by the roof. This resulted in the lateral loads being resisted by gypsum lath and plaster instead of plywood in one direction. Also, the roof had unequal slopes. The foundation of the 1972 addition was reinforced.

The dwelling was located 9.65 km (6 miles) from the epicenter and instrument stations around the dwelling recorded strong ground motions with large vertical accelerations. The structural damage to the house was considered moderate.

Settlement of one side of the foundation of approximately 25 mm (1 in) caused severe damage to the exterior and interior plaster wall finishes. These finishes served as lateral bracing and were considered to have sustained the main structural damage. Cracks in the exterior plaster occurred at the foundation sill plate. A permanent racking of approximately 13 mm over 2.4 m (1/2 in over 8 ft) of the exterior wall was noted. Interior plaster cracking occurred most often at the corners of doors and windows and at the edges of the gypsum lath panels. Cracks also occurred in the first floor ceiling and splitting of the first-story plywood subfloor was observed. Both unreinforced and reinforced foundations sustained severe vertical cracks. The chimney partially collapsed but this was attributed to poor construction practice and inadequate inspection.

The house was considered to be well within the life safety criteria of the building code. However, the estimate for repairs which included work on the foundation made the repairs uneconomical. The option to retrofit existing houses similar to the one in this case study was also estimated to be uneconomical. Based on the findings, Russell (1996b) made the following recommendations:

[3] k is the effective column length for calculating column buckling.

1. Codes should address damage control by including limits on drift and deflections for wood-frame houses.
2. Determine acceptable performance of Portland cement plaster and gypsum drywall (most commonly used finish materials) under cyclic loading.
3. Require tests of questionable soils to determine their dynamic characteristics.
4. Require reinforcement in the foundation for houses located in seismic zones.
5. Improve quality of construction by making contractors and building inspectors aware of the significance of all aspects of the installation of materials that could affect the performance of houses under lateral loads.

The potential benefits and effectiveness of retrofit are illustrated in another case study by Russell (1996a). The retrofitted structure was a one-story, conventionally framed, wood house built in 1911. The house was situated on alluvial soil and located 22.5 km (14 miles) from the epicenter. Lateral bracing consisted of wood siding on the exterior and wood lath and plaster on the interior. The house was supported on a continuous unreinforced concrete perimeter foundation and concrete piers. Between the first floor and the foundation was a 305 mm (12 in) cripple wall.

The retrofit, completed three months prior to the Northridge earthquake, consisted of:

1. Anchoring the exterior wall sill plates to the concrete foundation.
2. Bracing exterior cripple walls with 10 mm (3/8 in) plywood.
3. Providing supplemental connections at each floor joist to the top of its supporting posts.
4. Bracing of the brick chimney with metal straps.
5. Reconstructing two brick columns under the front porch.

These measures were taken to prevent cripple wall collapse, sliding of the exterior wall on the foundation, and chimney damage. Also, when the roofing was replaced, 19 mm (3/4 in) plywood sheathing was added on top of the roof rafters. The effectiveness of this retrofit could not be determined as the nailing pattern was unknown. Another improvement was the addition of 51 mm x 152 mm (2 in x 6 in) ceiling joists at 0.4 m (16 in) spacing with particleboard attached on the attic side. This improvement was made when the ceiling plaster was replaced after it was damaged in the 1971 Sylmar earthquake.

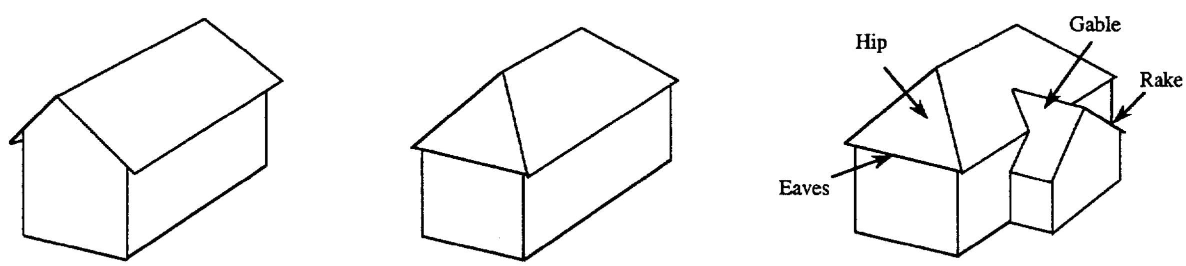
Other than the damage to the chimney, the building sustained only minor damage such as a few interior and exterior cracks, minor displacement of furniture and spillage of contents in cabinets. The only recommended action was to replace the chimney with a metal fireplace and flue ($6,700) and to provide a more symmetrical distribution of the cripple wall bracing. Damage to similar buildings in the neighborhood included cripple wall failures which led to demolition and rebuilding of the house in two cases, damage to front porch roof support posts, and partial collapse of all unreinforced masonry chimneys.

The performance of this older house illustrated that strengthening measures at a relatively minor cost ($3,200) could prevent costly earthquake damage. Although recommended code documents for strengthening older houses are available (ICBO, 1997b, FEMA, 1997), their implementation by homeowners would require mandatory compliance or voluntary efforts encouraged by incentives such as reduced insurance premiums, tax deductions, etc.

2.4 HURRICANE PERFORMANCE

Hurricanes rarely cause major structural failures in engineered buildings and such failures, while not widespread, mostly occur in non-engineered buildings. A common conclusion based on observed hurricane damage to non-engineered buildings is that the majority of the problems are related to failures of the roof system. This is because roofs are generally subjected to larger loads than other components and not much attention is given to the engineering of roofs (Smith and McDonald, 1991). Roof damage leads to more costly damage as a result of interior water damage.

The most common types of roofs in the United States are the gable and hip roofs (Figure 2.3). Most roof damage starts at the corners and edges facing the wind and is caused by increased suction and uplift forces at abrupt changes in geometry (Watford, 1991) as shown in Figure 2.4. Earlier building codes did not include the effects from wind gusts and high suction around building corners and roof edges (Sparks et al., 1994). Modifications to the building codes have eliminated these deficiencies, but the issue of near ground wind speeds are still not addressed.



a. Gable b. Hip c. Intersection

Figure 2.3. Roof Types.

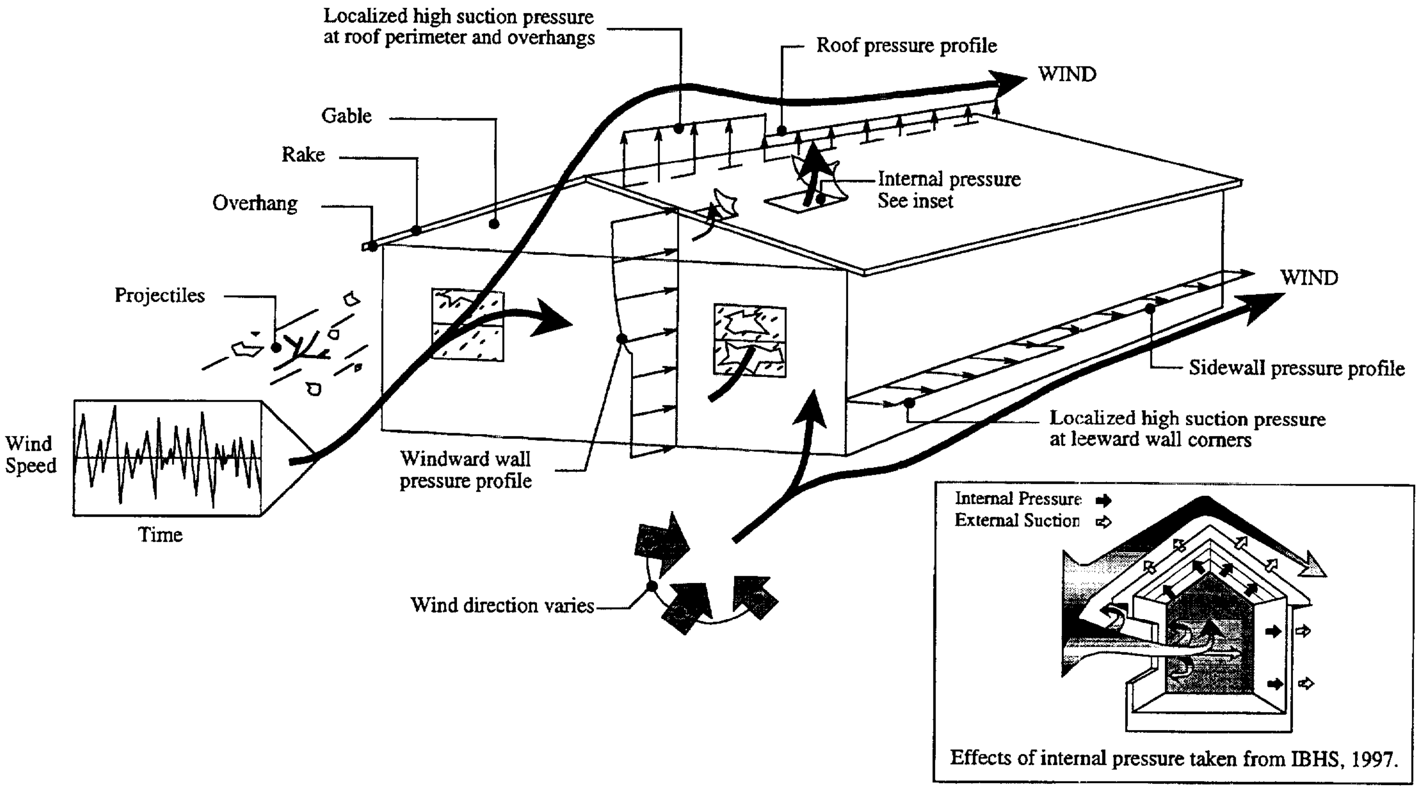


Figure 2.4 Wind Effects on Low-Rise Buildings (after NAHB, 1993).

Besides increasing water damage, breached envelopes such as broken windows increase the internal pressurization and thereby, the suction forces on the roof. The phenomenon of internal pressurization can effectively double the wind loads on the leeward and sidewalls, and the uplift load on the roof. In urban coastal areas, it was found that wind damage to building envelopes contributed to more than half of the insurance losses in a major hurricane (Sparks et al., 1994).

Many factors affect wind loading on a structure with the main factors being the surrounding terrain and wind speed (wind pressure is proportional to the square of the wind speed). A relationship between loss and wind gradient speed was developed by Sparks et al. (1994) based on data from Hurricanes Hugo and Andrew. They found that damage increased linearly for wind speeds ranging between 40 m/s (90 mph) and 70 m/s (160 mph). Between 70 m/s (160 mph) and 80 m/s (180 mph), loss of roof sheathing and broken windows and doors result in a sudden rise in damage. The large increase in damage is due to water damage once the roof covering and sheathing are lost. Sparks et al. (1988) attempted to assess the structural damage of existing buildings in a typical design hurricane - fastest-mile wind speed of 45 m/s (100 mph) at an elevation of 10 m (33 ft) in open terrain. Fastest-mile wind speed is based on the time for a mile of wind to pass an observation point. The findings were based on wind tunnel tests and are shown in Table 2.5. It was found that occurrences of roof failures were likely for wind speeds ranging from 22 m/s to 27 m/s (50 mph - 60 mph); however, the failure speed is a function of wind direction, roof shape, type of roof-to-wall connection, and size and location of wall openings. The following sections summarize the performance of wood-frame houses in four hurricanes. The damage assessments from more recent disasters such as Hurricanes Opal in Florida and Fran in the Carolinas, both occurring in 1996, are not presented as the damages from these two hurricanes were due mainly to flooding, wave action, erosion, and scour and not due to wind. The emphasis of the following sections will be wind damage associated with hurricanes.

Table 2.5 Risk of Structural Damage in a Design Hurricane (after Sparks et al., 1988).

	Type 1			Type 2		
	Class A	Class B	Class C	Class A	Class B	Class C
Sheltered Secured	Low	Low	Low	Low	Low	Low
Sheltered Unsecured	Low	Low	Low	Medium	Low	Low
Open Secured	Low	Low	Low	Medium	Low	Low
Open Unsecured	High	Low	Low	High	Medium	Low
Severe Secured	Low	Low	Low	Medium	Low	Low
Severe Unsecured	High	Low	Low	High	High	Low/Medium

Type 1 - Hip roofs with slopes greater than 25°
Type 2 - All other roofs
Class A - Ordinary toe-nailed connections
Class B - Light-duty hurricane anchors
Class C - Heavy-duty hurricane anchors
Sheltered - Wooded areas, densely packed subdivisions and centers of towns
Open - Flat open country with few obstructions
Severe - Flat areas adjacent to the sea
Secured - Windows protected against damage, porches and carports secured against uplift forces
Unsecured - All other buildings with porches and carports or with windows exceeding 5% of the wall area

2.4.1 Hurricane Camille - August 19, 1969

As reported by Dikkers, et al. (1971), Hurricane Camille hit the Mississippi-Louisiana Gulf coast on August 17, 1969. According to the National Hurricane Center, Camille was the second most intense hurricane to hit the U. S. in the 20th century and was a Category 5 hurricane based on the Saffir-Simpson scale. The estimated damage was $1.42 billion (1969 dollars) with 248 fatalities (Dikkers et al., 1971). It was reported that approximately 5,600 houses (excluding mobile homes) were destroyed, 14,000 houses sustained major damage, and 34,000 houses sustained minor damage.

Houses close to the coast were damaged by the storm surge, flooding and water borne debris. Wood-frame houses sustained relatively little wind damage and performed well when constructed in accordance with code provisions. To reduce damage, it was important that buildings be properly anchored to their foundations, and walls, floors, and roofs be adequately tied together. The majority of damage was from wind borne missiles and falling trees. In instances of severe damage, the damage was caused mainly by roof failures. Wind-resistant asphalt shingles generally performed well.

Soltis (1984) reported that damage in this hurricane was due mainly to combined wave and wind action and inadequate connections between the structures and foundation. Damage was also caused by inadequate or improper ties between the walls and roofs. Where proper ties were provided, damage was confined to the roof. In addition, the shape of the roof was reported to influence the severity of damage. Hip roofs sustained less damage compared to gable roofs.

2.4.2 Hurricane Hugo - September 22, 1989

Hurricane Hugo struck the South Carolina Coast on September 22, 1989 and was classified as a Category 4 hurricane with an estimated damage to property from flooding and wind of over $7 billion. A statistical analysis of wind damage conducted by Watford (1991) showed that of insured damage to buildings, 75% of the claims were for conventional single-family dwellings which accounted for 55% of the total payments. On average, direct wind damage payments amounted to about 7% of the insured value of the structure. In 95% of the cases, roof damage accounted for over 95% of the direct wind damage. It was found that houses built after 1971 performed better than those built before 1971 as post-1971 houses were required to comply with the Standard Building Code in order to be insured by the Windstorm and Hail Underwriting Association. However, as noted by Watford, the improved performance may also be due to the fact that the post-1971 houses were newer and in better condition than the pre-1971 houses. Damage to hip roofs was less than the damage to gable roofs with flat roofs sustaining the most damage among the three roof types. Insufficient data were available to verify the expectation of more roof damage as the slope of the roof decreased.

In low-rise structures, the walls and roof constitute a major part of the lateral load resisting system and the failure of either of these components leads to collapse. In his survey of the damage, Sparks (1990) found that damage to roof coverings and wall cladding occurred when the fastest-mile wind speed exceeded 27 m/s (60 mph). Major structural damage such as loss of roof structure, collapse of single-story masonry buildings, complete destruction of mobile homes and extensive damage to older pre-engineered metal buildings and wood-framed construction occurred when the fastest-mile wind speed exceeded 38 m/s (85 mph). The extent of the damage was influenced by the upwind terrain with structures in open terrain and along the coast sustaining more damage than the more sheltered or inland structures. South Carolina is a heavily wooded state, and trees were both an advantage and a disadvantage as they provided shelter for the structures but they also damaged the structures when they fell as the fastest-mile wind speed was about 27 m/s (60 mph).

Sparks (1990) found that shelter from trees contributed significantly to the good performance of one- and two-family dwellings. Major structural damage was caused by loss of roofs which led to the collapse of walls, shear failures, and foundation failures. Roof failures were not as prevalent in this hurricane as in past hurricanes. This was attributed to the steep-pitched roofs and the use of hurricane anchors. However, roof failures occurred in highest wind regions even when hurricane anchors were used and especially when windows were damaged. Many houses lost shingles which caused water damage to the interior. These two types of apparently minor damage resulted in a tenfold increase in the cost of repairs.

Foundation failures were more common in Hurricane Hugo than in previous hurricanes with instances of the structure floating away from its foundation (Muerden, 1991 and Manning and Nichols, 1991). Many buildings in South Carolina were supported on concrete block piers which rested on small spread footings with shallow embedment depths. Unreinforced masonry piers with a height to width ratio of 10, if filled with concrete, were allowed by the Standard Building Code. These piers failed as did lightly reinforced piers. Rogers (1991) noted that pier failures resulted from insufficient lateral resistance and undermining of the footings from erosion.

Some shear failures occurred in two-story buildings due to inadequate bracing. Conventional bracing proved to be inadequate in buildings with few interior crosswalls and large openings in the exterior walls.

Murden (1991) found that major damage occurred in regions where fastest-mile wind speeds exceeded 38 m/s (85 mph) where structural collapse, roof loss, destruction of mobile homes, and extensive damage to wood-frame construction occurred. Fastest-mile wind speeds of 31 m/s (70 mph) caused damage to roofing systems and wall cladding. As in previous hurricanes, the major economical losses were a result of water damage to the interior of houses caused by loss of roof covering and sheathing, damaged roofs, and damaged windows and doors.

Similar to Sparks'(1990) findings, Murden (1991) also found that one- and two-family houses performed well. As observed in past hurricanes, roof failures constituted the majority of structural failure and was a result of inadequate connections between the roof and the exterior wall (Manning and Nichols, 1991). The loss of walls was usually the result of loss of roofing. The roof and wall failures initiated at the corners and eaves of the building (Curry, 1991). Sheet metal, whether used in siding or roofing, performed poorly (Miehe, 1991). However, as compared to data from previous hurricanes, there was less roof failures in this hurricane. The improved roof performance was attributed to quality of design and installation of the roofing system with no one roofing system being better than another (Cook, 1991), adequate use of hurricane anchors, and steeper roofs (Murden, 1991). Also, good roof performance was attributed to compliance with the local building code and federal government flood plain requirements (Manning and Nichols, 1991). In general, hip roofs performed better than gable roofs as observed in previous storms. Chimney damage was widespread with no difference in performance between masonry and wood-frame construction (Miehe, 1991). As observed by Smith and McDonald (1991), the performance of asphalt shingles was highly variable as it depended on the wind resistance of the shingle and the performance of the adhesive backing. Smith and McDonald also found that the use of underlayment proved beneficial in reducing damage as it provided secondary protection once the roof covering was lost. They also noted that metal roofing clips attached with nails failed because nails have low pull-out resistance and were prone to dynamic loading fatigue.

While there was a decreased number of roof failures, other failures such as shear failures became more common. These shear failures resulted from the trend towards more open floor plans and larger internal spaces and windows near corners.

2.4.3 Hurricane Andrew - August 24, 1992 and Hurricane Iniki - September 16, 1992

Damage from Hurricane Andrew (Category 4) which hit South Florida on August 24, 1992 was estimated at $20-25 billion and the damage from Hurricane Iniki (Category 3) which struck the Hawaiian Island of Kauai on September 16, 1992 was estimated at $1.2 billion. In both events, most of the damage was due to water damage to the interior of the house. Significant structural damage occurred in less than 20% of the homes surveyed in both hurricanes. Damage assessments from Hurricane Andrew will be presented first followed by those for Hurricane Iniki.

The survey and data collection conducted by NAHB Research Center (1993) for HUD were statistically based so that conclusions drawn could be extrapolated to the housing population in the respective regions. The houses in Florida were typically one-story with masonry walls and gable roofs. Roof coverings were shingles over plywood sheathing connected to wood trusses. Foundations were mainly slab on-grade and the houses were mostly 5 to 25 years old. The building characteristics of the surveyed houses are given in Table 2.6.

Table 2.6 Building Characteristics of Surveyed Homes in Dade County (after NAHB, 1993).

Number of stories	One story	80%
	Two stories	18
	One and a half	2
Roof type	Gable	79
	Hip	16
	Gable-on-hip	4
	Other	1
Roof framing	Wood truss	83
	Wood rafter	17
Roof sheathing	Plywood	89
	Board	6
	OSB	3
	Other	1

Roof damage constituted 97% of all the structural damage. The wood trusses did not fail but the loss of a few panels of sheathing initiated eventual collapse or failure of the roof. This was the result of the reliance on sheathing and ceiling drywall to provide the lateral resistance of the roof structure. Loss of the roof sheathing was due to inadequate fastenings (not following recommended nail spacings or fasteners missing framing) and insufficient anchoring at rake overhangs. Most of the roof sheathing was plywood (90%) with the rest being either board or OSB. There was no indication that one type of sheathing performed better than another. Gable ends did not usually have secondary bracing but the existence of secondary bracing prevented damage to the roof framing in only a few cases and was found to be more critical for taller roofs with larger gables.

Building height (one- versus two-stories) influenced the extent of damage with greater damage occurring in the two-story houses. Greater damage was expected for two-story houses as one-story houses had less windows and exposure area. There was no clear relationship, however, between the building height and the extent of roof damage.

There was statistical difference in the performance of gable roofs and that of hip roofs with the former sustaining more damage. Possible reasons for better performance of hip roofs were:

1. Framing geometry of hip roofs makes them better for lateral load resistance.
2. Hip roofs are more aerodynamic and therefore, reduce the wind load on the roof.
3. Increased skill to frame hip roofs which may result in better workmanship.
4. The surveyed houses with hip roofs had more wind resistant roof coverings and had more window protection.

Water damage to the interior was due mainly to the loss of roof covering and was the major contributor to property loss and cost of repairs. In terms of projectile damage and wind uplift, it was found that flat tile roof systems performed the best, followed by contoured tile, and then by composition shingles. The waterproof underlayment beneath all types of tiles provided additional water protection. Hurricane shutters also reduced water damage and structural damage from internal pressurization.

Wall damage was infrequent and was typically minor as most (99%) of the surveyed houses had concrete block and stucco (CBS) walls on the first story. Of the two-story houses, only 17% had wood framing on the second floor. Damage of the CBS walls was due to insufficient reinforcement for continuous load paths or insufficient overlap at corners. Wood-frame wall damage was due to improper connections to the top plates at the corner joint and to the reliance on the roof structure to provide lateral support to resist severe wind loads. No foundation damage occurred due to wind forces.

Based on their observations of the damage sustained in Hurricanes Andrew and Iniki, recommendations by NAHB (1993) to improve the hurricane resistance of single-family houses include:

1. Improve compliance with wind-resistant construction practices.
2. Improve building code requirements to address major factors contributing to damage - roofing, roof sheathing, and window protection.
3. Conduct research to improve understanding of extreme winds and near-ground wind effects.

Findings similar to those in the NAHB (1993) report were observed by Oliver and Hanson (1994) in their damage survey after Hurricane Andrew. Breach of the building envelope occurred significantly more often in wood-frame buildings than in masonry buildings. Breaches were caused by improper attachment of the roof sheathing to the top chord of the roof truss and lack of bracing of the roof truss especially at the gable end. These breaches led to instability and eventual failure of the roof systems. Braced truss roof systems, secondary bracing, and the use of hip roofs instead of gable roofs were recommended by Oliver and Hanson.

A survey of exterior wall damage from Hurricane Andrew conducted by Sanders (1994) found that damage of wood components other than by missile impact was not common. Failure of wood structures was usually caused by failure of the connections. Shear walls of Masonite®[4] or exposed plywood with textured surfaces performed poorly because of inadequate strength of Masonite® and deterioration of poorly maintained textured plywood.

Sanders (1994) indicated that inadequate connections of the top chords and lack of lateral support of the bottom chords of the gable end truss were the main causes of failure. He found that hurricane straps were used but not shear connectors. Top chord connections failed because they could not resist the combined lateral and uplift loads.

[4] Certain trade names and company products are mentioned in the text or identified in an illustration in order to adequately specify the experimental procedure and equipment, or material used. In no case does such an identification imply recommendation or endorsement by the National Institute of Standards and Technology, nor does it imply that the products are necessarily the best available for the purpose.

Based on the observed damage, Sanders found that failures in wood-frame buildings resulted from:

1. Discontinuities in load path.
2. Errors made during construction.
3. Inadequate connection strength.
4. Use of Masonite® or wood that is not protected from weathering.

Wolfe et al. (1994) examined the wind resistance of conventional frame structures in Dade County. They found that failures were due to inadequate connections and missile damage rather than inadequate code requirements. These failures occurred at the:

1. Attachment between roof covering to roof sheathing.
2. Attachment of roof sheathing to roof framing.
3. Rake overhang features.
4. Attachment of interior walls to exterior walls - although interior walls were usually considered non-structural, they contributed to the lateral resistance.

Composition shingles (asphalt or fiberglass) were warranted for wind speeds of 27 m/s (60 mph). Improved performance can be obtained by sealing the edges at the roof eaves and rakes with roofing cement and by using 6 nails per shingle instead of 4. Elastomeric and foam roofing products are available which provide a continuous membrane and have better wind resistance.

They also found that taping of windows prevented shattering of glass but not glass breakage, and inadequate fastening of plywood over windows caused the plywood to come off and become flying debris. Issa et al. (1994) found that storm shutters reduced window damage and thus damage to the interior of the house. The most common storm shutters were removable galvanized steel or aluminum storm shutters. Issa et al. (1994) found that anchoring of these shutters with lag bolts to the external wall is preferred over nailing, and airtight shutters were more effective.

In the aftermath of Hurricane Andrew, Khan and Suaris (1994) examined the design and construction deficiencies of structures and adherence to the South Florida Building Code (1988). For single-story framed construction, the building code required board or plywood storm sheathing on all exterior walls. However, in some cases, only hardboard siding was used with no sheathing and this practice led to collapse of the house. Siding products such as Masonite® and Thermax® approved for use by the Dade County Building and Zoning Department have much less racking shear resistance than plywood [e.g. The shear strength of a 12 mm (15/32 in) thick plywood is 2 to 3.6 times greater than that of Masonite®]. Furthermore, the more stringent nailing and stud spacing requirements for these products were usually not followed by contractors which aggravated the situation.

Khan and Suaris also observed other violations of the building codes such as

1. Improperly constructed corner studs.
2. No overlapping of plates (e.g. top plates) at intersections.
3. Inadequately nailed connections.
4. Improper splicing and notching of members.
5. Missed or missing hurricane straps in stud plate connections and sill plate anchors to foundation.

Some wood end-gables did not have the bracing to resist the lateral forces. Asphalt shingles and tiles were commonly used as roof coverings but performed poorly as they were not rated for wind speeds of 53 m/s (120 mph) -- shingles were approved by the Building and Zoning Department. Also, poor performance resulted from poor construction practices such as:

1. Tile installation - inadequate nailing and sloppy placement of mortar beds.
2. Roof sheathing - inadequate nailing/stapling, missing targets.

Khan and Suaris found that OSB sheathing panels did not perform as well as plywood as they seemed to disintegrate/curl at the edges under cyclic loading and/or moisture penetration. In addition, sheathing staples did not perform as well as nails when subjected to combined tension and shear forces.

In two-story composite or framed construction, the main deficiency was improper connection of the second story stud wall to the first story concrete block stucco (CBS) or stud walls below.

Contrary to the finding by Khan and Suaris (1994), Keith (1994) in his damage surveys of Hurricanes Andrew and Iniki found that plywood and OSB roof sheathing performed equally well. The three building types, wood, masonry/wood, or masonry, sustained similar sheathing failures. He found that the majority of wood structural sheathing failures in South Florida was due to inadequate connections. The most common failure was the loss of gable-end walls which was associated with the loss of plywood or OSB sheathing immediately adjacent to the gable-end wall. Diagonal cross-bracing of roof trusses was lacking in this failure mode and loss of sheathing was due to improper nailing. Improper connection of the gable-end truss to the framing also contributed to the problem. Construction practices that aggravated the problem included:

1. Connection of the gypsum wallboard ceiling to steel channels or wood furring strips rather than to roof framing or the perimeter (exterior) walls. This reduced the lateral resistance of the roof structure and removed the lateral resistance of the end wall at the "hinge point" between the gable-end wall and the gable-end truss.
2. New construction procedures which incorporated a non-structural rake-end roof overhang. This overhang was attached to the gable-end truss with minimal nailing and the sheathing was attached to this overhang with minimal or no nailing to the gable-end

truss. Peeling of the roof sheathing back to the second or third truss due to uplift occurred during the hurricane.

3. Use of tie-down straps in lieu of other fasteners such as shear connectors or lateral bracing.

In masonry and wood structures, all collapsed gable ends were caused by improper attachment of the gable-end truss to the masonry wall, loss of lateral bracing at the connection of the top of the masonry wall and the bottom of the gable end, or loss of bracing at the top of the gable end.

In Kauai, wood construction performed well. Similar to findings by others, hip roof construction performed well as did gable roofs which were properly braced at the gable-end to end-wall connection and had properly attached sheathing. The "good performance" of the gable roofs in Hurricane Iniki as compared to their performance in Hurricane Andrew was attributed in part to lower wind speeds.

OSB panels were not widely used in Hawaii. The performance of plywood sheathing was similar to that in South Florida - failures occurred due to improper attachments. Other roof sheathing systems include tongue-and-groove (T & G), and 25 mm (1 in) nominal spaced board roof sheathing. Most wood sheathing failures occurred in the T & G boards due to inadequate fastening of the roof deck to the framing members or lack of tie downs of the roof framing members to the rest of the structure. Substantial eave overhangs also added to the uplift forces on the roof sheathing.

Keith's (1994) conclusions were as follows:

1. Roof sheathing performed well if attached in accordance with code requirements.
2. Hip roofs performed satisfactorily. Gable roofs performed satisfactorily if
   a. Gable-ends were properly braced and the sheathing was properly attached at the gable-end
   b. The gable end-to-end wall was properly connected.
3. Overhangs are subjected to higher uplift forces and should be appropriately designed.

As reported by NAHB (1993), the houses in Kauai were typically one-story with approximately 90% wood-frame construction. In general, older wood-frame houses were of single-wall construction (42%), typical of the local construction standard, and newer houses were of conventional wood-frame construction (48%). Single-wall construction consisted of 25 mm (1 in) tongue-and-groove boards placed vertically and nailed to the top and bottom plates without studs (Keith, 1994). The tongue-and-groove boards served as sheathing/siding for uninsulated load bearing walls. Roof types varied widely with a combination hip/gable (see Figure 2.3c) being common. The most common roof coverings were composition shingles and metal with each being equally prevalent. Roof sheathing was typically plywood or corrugated metal and the wood trusses were built on site. Wood post foundations were typical. Newer houses had conventional framing and slab on-grade foundations. The building characteristics of the surveyed houses are given in Table 2.7.

Table 2.7 Building Characteristics of Surveyed Houses in Kauai (after NAHB, 1993).

Number of stories	One story	81
	Two stories	17
	Other	2
Roof types	Gable	49
	Hip	38
	Gable-on-hip	9
	Other	4
Roof framing	Wood truss	66
	Wood rafter	30
	Other	4
Roof sheathing	Plywood	60
	Metal	33
	Other	7
Roof Covering Material	Composition	40
	Metal	32
	Wood	14
	Gravel	7
	Other	7
Exterior Wall Construction	Conventional Wood	48
	Single-Wall Wood	42
	Concrete Masonry Unit (CMU)	5
	Other	3
Foundation Type	Slab on-grade	37
	Wood post & pier	34
	CMU Pier	20
	CMU Perimeter	7
	Other	3

As in Florida, most of the structural damage in Kauai was sustained by roofs with 64% of the surveyed homes sustaining extensive roof covering damage. Damage was due to inadequate connections of truss members and sheathing to framing members, and in older homes, damage was due to discontinuous load paths. Hip roofs sustained less damage than gable roofs with the source of the problem being the attachment of the roof sheathing at the gable end. Water damage was a large factor in the overall losses and resulted from failure of the roof covering and damaged windows. Composition shingles performed better than corrugated metal roofs in terms of wind resistance and damage. This was because shingled roofs were wood sheathed and the wood sheathing remained better fastened than corrugated metal; and many of the metal roofs were in the region of higher wind loads.

Most of the wall damage was classified in the lowest damage category - damage to one-third or less of the wall. Wall failures were infrequent and occurred in all types of walls with "single wall" construction resulting in structural failure in two cases (Keith, 1994).

The three main types of foundation were slab on-grade (37%), wood post and pier (34%), and concrete masonry piers (20%). The foundations typically sustained minimal damage with rare occurrences of overturning, uplift and sliding off post and pier foundations. These failures were due to inadequate design (inadequate bracing or load paths) or installation.

In contrast to Hurricane Andrew, the single-family houses in the coastal regions in Kauai were severely damaged by water from the storm surge or suffered significant structural damage from the impact of the surge. Houses with large openings which gave way, thereby reducing the force of the storm surge, sustained less structural damage. For houses in the coastal region and on top of steep slopes, two-story houses sustained greater damage than single-story houses. Another difference in Kauai was the island topography which created localized high winds.

It was found that the characteristics of houses which most influenced the severity of hurricane damage in Kauai were:

1. Opening protection (windows and doors)
2. Roof coverings
3. Roof sheathing attachment.

As in most hurricanes, water damage was the major factor in the loss of property and cost of repairs. Roof covering problems were associated with conventional composition shingles and metal roofing in Kauai. As is apparent, damaged windows and doors increase water damage. An inexpensive and effective method to prevent window damage is the installation of plywood covering.

2.5 SUMMARY

A review of the earthquake and hurricane performance of wood-frame, single-family houses is presented. In general, these types of houses have performed well as relatively few lives have been lost and the number of failures or collapses have been limited. This good performance is attributed to their light mass, redundancy from nonstructural elements, and ductility when adequate connections are provided. However, considerable damage was observed which led to significant economic loss.

2.5.1 Earthquakes

Beginning in 1940, seismic standards have been incorporated in building codes and since then, building codes have been modified to reflect better understanding of the performance of wood-frame houses in earthquakes based on damage sustained in such events. However, earthquake reconnaissances have found that both modern and older houses are vulnerable to seismic damage and that modern wood-frame structures do not seem to have better seismic resistance over earlier wood-frame structures (Corbeen, 1996). This is not an indication of inadequate building codes but is likely a result of widespread occurrence of construction flaws or inadequate quality assurance - missing fasteners, overdriving of shear wall nails, improper placement of hold down bolts, fasteners missing framing members, etc. The preference for more open interior spaces, hillside homes, soft stories, and more irregularly shaped structures may also be contributing factors.

In addition, some architectural characteristics of older buildings (those constructed prior to the inclusion of seismic provisions) such as more regular plan and smaller openings in walls, enhance their seismic resistance as compared to newer buildings. According to (Kicinski, 1995), older houses also benefit from:

1. Redundant load paths. In addition to main shear walls, older homes have many small rooms and interior walls.
2. Stronger framing lumber. Old growth, strong, full dimensional lumber was used.
3. Flexible structure with high damping. The use of a large number of nails results in a ductile structure with higher damping than other structural materials. Also, tongue-in-groove skip sheathing joints increases damping through friction.
4. Contribution to racking strength of interior wallboard or plaster and exterior siding of older homes.

However, in spite of these advantageous attributes, it was the found that older buildings performed poorly in the 1971 San Fernando and the 1989 Loma Prieta earthquakes and houses built in accordance with the 1973 UBC performed well (Jephcott and Messinger, 1991). The age of the houses and lack of maintenance may have contributed to the poorer performance of the older houses. In general, when subjected to seismic loads, poor performances of old and new wood-frame houses have resulted when the house did not respond as a unit due to discontinuous load paths. Common causes of failures (Foliente, 1995, Kicinski, 1995) which resulted in the lack of integrity were:

1. Insufficient or poorly detailed intercomponent connections (anchorage to foundation, wall-to-wall connections, wall-to-roof/floor connections, corner connections).
2. Inadequate bracing (cripple walls, total absence of shear walls, large wall openings, inadequate let-in bracing).
3. Component separation (masonry fireplaces and chimneys, masonry veneers, porch roofs and other overhangs, different house sections).
4. Non-uniform or irregular distribution of stiffness (split level house, setbacks) which leads to torsional movements.
5. More irregular plan and elevation (geometric irregularity) in newer houses. Anchorage and cripple wall failures were more common in older buildings and if these failures were ignored in newer buildings, failures would be attributed to more irregularity in the plan of the newer buildings.
6. Poorer detailing and quality of construction in newer buildings as compared to older buildings. This is because some failures in new buildings have been attributed to improper hardware installation. Adequate inspection of critical elements/connections is necessary.
7. Lack of continuous load path from roof to foundation.

   a. Lack of anchor bolts between walls and foundation.
   b. Unbraced cripple wall between house and foundation.
   c. Lack of properly constructed shear walls.
   d. Lack of proper method to prevent overturning of shear walls.
8. Poor soil conditions (liquefaction, excessive settlement) or location (building on hillsides).

2.5.2 Hurricanes

From performances in past hurricanes, severe damage to residential wood structures have been mainly caused by roof damage due to inadequate anchorage of the roof framing to the wall, damage to the roof covering, and loss of roof sheathing. Foundation failure and inadequate anchorage of structure to the foundation have also been causes for failures. Water damage from roof failures was a major contributor toward the economic losses. Wood shingles and shakes have performed better than asphalt shingles, metal roofs and tile (Liska and Bohannan, 1973). Asphalt shingles are typically not designed for high winds (IBHS, 1997). Also, hip roofs were found to perform better than gable roofs.

In addition to increasing water damage, the internal pressure in a house is increased due to breaches in the building envelope from missiles, thereby contributing to roof failures. Properly installed plywood boards over windows or storm shutters, doors, and garage doors can alleviate this problem. In addition, improperly installed boards and shutters contributed to the damage from wind-borne debris. Therefore, to reduce damage from a hurricane, the tasks that a homeowner can do include reinforcing and protecting the roof, windows and doors. These tasks can be done in conjunction with other home improvements and would not be too costly. Other hurricane mitigation measures may be found in the literature (FEMA 1992, 1993).

2.5.3 Conclusions

In general, wood-frame houses performed well. However, failures have occurred in extreme load events such as earthquakes and hurricanes with the failures initiating from connection or anchorage failures. As stated by Foliente (1995), "Despite many of the things that are still little understood about earthquake response of wood buildings, if published recommendations on basic wood construction in seismic areas were followed, the damage potential to low-rise wood-frame buildings would be significantly reduced."

The general view held by the authors cited in this chapter is that the knowledge to mitigate hurricane and earthquake damage exists and most of the damage sustained from past events could have been prevented. However, the knowledge has to be transferred or incorporated into "non-engineered" buildings (Soltis, 1984). The damage from the deficiencies from non-engineered structures can be addressed through proper design, good construction practices, adequate inspection, and inexpensive retrofitting (Kicinski, 1995). The first three issues require that (Wallace, 1993):

1. The local authority adopts the appropriate building code: No adoption, no code, and no mitigation. As observed by Sparks (1990), much of the wind damage in Hurricane Hugo was avoidable, and "the state of knowledge of wind effects was such that no building constructed after the mid-1960s need have been damaged by the storm; but for reasons more political than technical, South Carolina found itself with a large population of buildings vulnerable to wind damage."

2. Effective enforcement be requisite as the building code is only as effective as its enforcement. Inspectors need to be properly trained.

3. Contractors and subcontractors be trained to properly install critical elements and be aware of the importance of maintaining a continuous load path.

To reduce the variability in quantifying the degree of damage , it is necessary to develop a standard evaluation form to assess the damage to single-family houses from hurricanes and earthquakes and also to assess the performance of structures that sustained little or no damage. A statistical approach such as that used in recent damage assessments needs to be utilized in future assessments to better determine if the damage is representative of the population of wood-frame houses. Procedures should be developed to assure compliance with building code requirements since compliance would have minimized the damage caused by hurricanes and earthquakes.

Although the principal intent of current building codes -- life safety -- is successfully met, disruptions of lives and commercial activities and the economic losses associated with these disruptions indicate that additional performance objectives may be necessary for residential structures. This was illustrated in a case study where the cost to repair nonstructural damage made the repair option uneconomical.

Finally, the building codes only impact new construction and major renovations. The issue of retrofitting existing buildings has to be addressed. As found by various reconnaissance teams, houses built after 1973 fared better than those built before 1973 in terms of seismic damage. As approximately 60% of U.S. housing (this includes multi-family and mobile homes) was built prior to 1970 (Bureau of Census, 1993), retrofit measures on houses in high seismic and hurricane prone regions may be judicious and as presented in the case study by Russell (1996a), more economical. Incentives such as reduced insurance premiums or income tax credits would encourage homeowners to undertake retrofit measures.

2.6 RESEARCH NEEDS

The relatively good performance of wood-frame single-family houses has been detrimental to the advancement of the state of knowledge since there has been no interest in acquiring more data by instrumenting wood structures to determine their behavior (Diekmann, 1994). Although the knowledge exists to reduce damages from earthquakes and hurricanes, only limited knowledge has been accumulated on earthquake or hurricane generated forces in wood-frame structures. If actual loads are not known, the reliability of analytical procedures cannot be determined nor can safety factors be assessed. More efficient use of wood is necessary as new, single-family, residential construction has been increasing at an average annual rate of about 5% in the period between 1993 and 1997 with an annual value of approximately $160 billion in 1997 (Bureau of Census, 1998). Ninety-nine percent of these houses are being constructed with wood.

Based on the need for more information on the behavior of wood-frame houses and the knowledge acquired from failures of such houses in past disasters, research in the following areas is required (Corbeen, 1996, Foliente, 1995, Jephcott and Messinger, 1991, Gupta, 1981):

1. Performance:
   a. Develop performance based design procedures for wood buildings based on displacement requirements. From the relatively small number of lives lost and collapses of single-family houses in earthquakes and hurricanes, it is evident that the principal intent of the building codes, life safety, is being met. However, the amount of economic losses and disruption of thousands of lives point toward a need to re-evaluate the performance goals for residential structures (see Section 5.1).

      The advantages of performance based design are:
      1.) More accurate prediction of structural performance which results in better and more economical designs.
      2.) Reductions in the cost of future rehabilitation.

   b. Review performance of existing structures. Integration of data from damage assessment surveys, testing, and analytical modeling to provide rational basis for design methodology.
   c. Accumulate database of damage data and standardize damage assessment forms. This would allow for damage classification and the eventual development of a damage index.
   d. Instrument wood-frame buildings to obtain strong motion data - none exists.
   e. Identify the desired location of failure in low-rise wood buildings and the acceptable failure modes.
   f. Detail of the ductile links to provide more predictable behavior by allowing a more ductile failure.
   g. Obtain reliable wind speed and pressure data.

2. Experimental research (see Chapter 3 for additional research topics):
   a. Increase knowledge on wood-frame construction subjected to dynamic cyclic loads - individual wood elements (tension and compression members), connections, and full-scale wood-frame structures.
      1.) Determine the hysteretic behavior and energy dissipation characteristics of wood buildings and their connections.

2.) Determine effects of load rate and damping.

b. Determine near-ground effects on wind speeds.
c. Determine the effects of topography on wind speeds.
d. Determine the relationship between the in-plane wall deformation of the lateral load resisting system and the acceptable deformation of the typical finishing materials. This information is necessary for the development of performance based design. The cost of repair of non-structural damage has been shown in some cases to be greater than the cost of rebuilding the structure.
e. Determine contribution of nonstructural elements to performance of wood structures.
f. Determine effects of load duration on the behavior of wood and wood-based products.
g. Develop means to minimize projectile damage.
h. Develop improved test methods to evaluate roof coverings.

3. Develop effective and economical methods for earthquake and hurricane mitigation (Litan et al., 1992):
   a. Upgrading of existing buildings.
   b. Changes to buildings codes so that new construction is designed to better resist lateral forces.
   c. Non-structural measures to reduce exposure of occupants and property to damage.

4. Develop construction methods that are wind or seismic resistant, can be easily inspected and are less sensitive to poor workmanship.

5. Perform non-destructive evaluation to determine compliance of existing buildings with building code and construction requirements.

3.0 EXPERIMENTAL STUDIES OF FULL-SCALE HOUSES AND STRUCTURAL SUBASSEMBLIES

3.1 INTRODUCTION

Wood is the most widely used construction material in North America. Optimal design of wood-frame structures can result in economic and ecological benefits. To apply performance based principles in the design of wood structures, it is important to understand the behavior of complete houses and their performance in earthquakes and hurricanes. Most structural failures in houses are attributed to these events and not to gravity loading. Full-scale testing is necessary to validate sophisticated and simplified analytical models of wood-frame structures. These arguments notwithstanding, there is only a sparse number of full-scale house experiments, laboratory-based or field-based, reported in the literature.

The reasons for limited tests of full-scale houses lie not only with the prohibitive cost of full-scale tests but with the unique properties of wood: higher variability than exhibited by most construction materials; mechanical properties which depend on the wood species and are non-homogeneous in three orthogonal directions; and load capacity that is affected by duration of loading. Hence, it becomes a difficult task to select a representative single housing unit for structural testing. In addition, it has been documented that for well constructed buildings, single-family, low-rise, wood-frame houses in the U.S have not sustained extensive structural damage during either hurricanes or earthquakes. Moreover, it is a widely accepted conclusion that the weak link in wood-frame, non-engineered structures is usually in intercomponent or intracomponent connections. Consequently, the vast majority of the wood research programs have focused on the performance of building subassemblies (floors, walls, roof/ceiling diaphragms) and intracomponent connections.

This chapter summarizes a number of experimental studies on wood-frame full-scale houses, shear walls, diaphragms, and connections performed during the past 30 years. Laboratory and field studies, involving monotonic, cyclic and dynamic loading procedures are included. While primary attention is focused on studies conducted in North America, selected studies carried out in Australia and Japan are also included.

3.2 FULL-SCALE HOUSE TESTING

3.2.1 Full Scale Test on a Two-Story House Subjected to Lateral Load (Yokel, Hsi, and Somes, 1973)

An existing two-story, wood-frame house with a partial brick veneer front at the lower story was subjected to racking loads in a field experiment. The single-family house was constructed in a housing development in Bowie, MD and slated for subsequent occupancy. The primary objective was to measure the lateral drift of a conventional wood-frame house under simulated wind load to determine whether the drift limitations prescribed in building codes for medium- and high-rise buildings were applicable to low-rise housing units. In addition, the dynamic response characteristics of the house were determined for a more accurate calculation of the effects of dynamic lateral loads such as those caused by earthquakes.

At the time of the test, there were no criteria limiting the lateral displacement of low-rise buildings subjected to wind and seismic loads. Rather, there were prescriptive provisions, in the Federal Housing Administration's Technical Circular 12 ("A Standard for Testing Sheathing Materials for Resistance to Racking"), governing the minimum lateral stiffness of shear walls oriented parallel to the direction of wind or earthquake force. The prescriptive code provisions for shear wall components were thought to be too simplistic to account for the complex interactions occurring between the structural and non-structural components.

There were no laboratory support facilities available and relatively rudimentary procedures were used to apply loads to the test surface of the house. Two basic experiments were conducted: first, concentrated, cyclic, static horizontal forces were applied to one face of the house to simulate wind loading, and the resulting horizontal and diagonal displacements were measured on the shear walls oriented parallel to the lines of force. The second experiment was carried out to determine the dynamic response of the house when subjected to impulsive loads.

In plan, the house measured 14 m x 8 m (47 ft x 26 ft). The front of the house contained a portico having a 102 mm (4 in) thick concrete floor slab resting on compacted fill. The portico slab abutted the front wall of the house, with its top surface located 0.9 m (2 ft 10 in) above the first floor level. Exterior walls were framed with nominal 51 mm x 102 mm (2 in x 4 in) studs spaced at 406 mm (16 in) on center. Interior and exterior sheathing consisted of gypsum wallboard. In addition to the single-wythe 102 mm (4 in) thick brick veneer, exterior siding consisted of asbestos shingles or 10 mm (3/8 in) thick beveled wood siding. Exterior wall studs were braced at all building corners with 25 mm x 102 mm (1 in x 4 in) wood let-in bracing installed at a 45 degree angle to the horizontal

The lower floor consisted of a 102 mm (4 in) thick concrete slab on-grade. The structural framing of the upper floor consisted of 51 mm x 203 mm (2 in x 8 in) wood joists spaced at 305 mm (12 in) on center, resting on bearing walls and intermediate supports.

The upper ceiling and the roof were supported by roof rafter trusses constructed with 51 mm x 102 mm (2 in x 4 in) wood members and spaced 0.6 m (24 in) on center. Roofing consisted of 13 mm (1/2 in) thick plywood sheets covered by asphalt shingles.

In the simulated wind loading test, four hydraulic jacks attached to steel beams were used to apply horizontal loads in a reasonable uniform distribution along the rear face of the house. Four concentrated loads, spaced 3.7 m (12 ft) apart, were first applied at the roof level and then at the second floor level.

In order to measure the dynamic characteristics of the house, a steel pipe was inserted between one of the hydraulic jacks on the lower level and the corresponding loading plate. After a predetermined horizontal load was applied, the pipe was suddenly removed by a sharp hammer blow, thereby causing the house to oscillate laterally.

The dynamic tests were conducted for peak horizontal loads of 4.5 kN, 6.8 kN, and 9.0 kN (1000 lb, 1500 lb and 2000 lb) to determine the natural frequency and percent critical damping of the test house. Based on the test measurements, it was concluded that the fundamental frequency was approximately 9 Hz and the damping averaged about 6% of critical damping, ranging from 4% to 9%.

The drift measurements were plotted for the two exterior side walls and one interior shear wall. The plots of the upper and lower ceiling drifts indicated that the house translated as a whole from rear to front and rotated slightly about a vertical axis. Drift values for the largest displacement at the lower ceiling level were compared to the then required drift limit [i.e. height (h)/500] for medium- and high-rise buildings for a wind load of a 50-year mean recurrence interval. The maximum second-story drift of the house, measured for a simulated wind pressure of up to 120 Pa (25 lb/ft^2), was about 70% less than the h/500 drift limit prevalent in U.S. design practice for medium- and high-rise buildings.

Although the drift near the center of the house at the second story level was greater than that at the side walls on the same level, it could not be concluded that the difference was indicative of in-plane deformation of the floor-ceiling assembly. Thus, the lower ceiling level tended to act as a rigid diaphragm.

The upper ceiling diaphragm underwent significant in-plane deformation. Correspondingly, the racking distortion of the upper-story interior partitions (shear walls) exceeded the distortion of the exterior side walls.

3.2.2 Structural Test of a Wood Framed Housing Module (Yancey and Somes, 1973)

The test unit was a prototype factory-built housing module, fabricated as part of the U.S. Department of Housing and Urban Development program entitled "Operation Breakthrough." The full-scale, factory-built module was designated as a second-story front unit of a townhouse cluster (Figure 3.1). A second-story module was selected because its construction was thought to be the most critical for transportation. The module's construction, however, was thought to be representative of the wood-frame housing system in general.

First, a static monotonic racking load test was conducted to determine the stiffness of the housing module with respect to lateral load and to estimate the second floor drift in the actual building when subjected to wind forces. One thousand cycles of simulated horizontal wind force were applied to determine the reduction in lateral stiffness under repeated application of lateral loading. Reversed cyclic lateral loading was applied to observe the extent of damage to connections and exposed components under simulated earthquake loading. Finally, a static monotonic lateral load was applied to measure the maximum lateral load that the module could withstand.

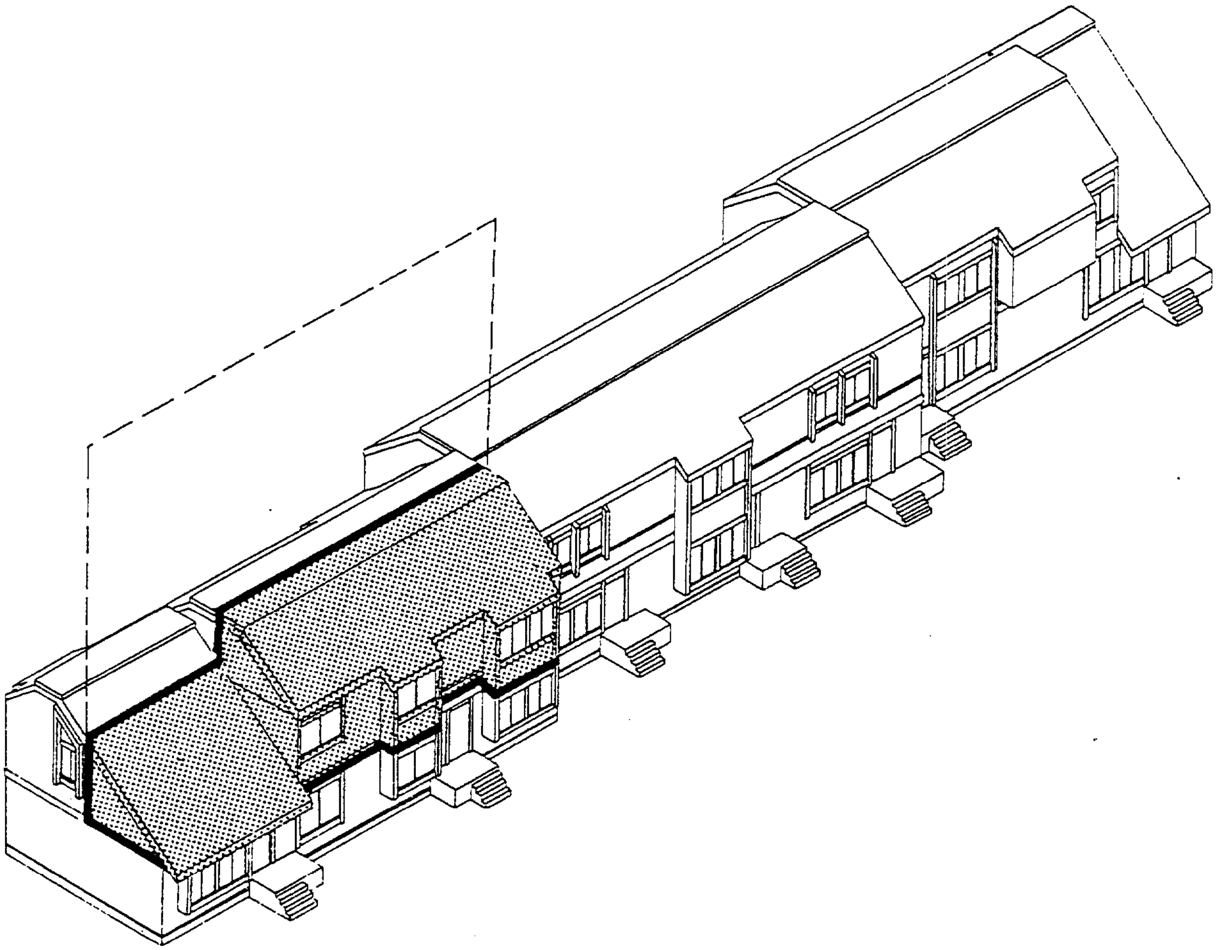


Figure 3.1 Isometric View of Typical Townhouse Cluster
(after Yancey and Somes, 1973).

As erected in the laboratory, the module was nominally 18 m (60 ft) long and 4 m (12 ft) wide. The height to the peak of the pitched roof was 5.8 m (15 ft 7 1/2 in). The ceiling and vertical framing were all nominal 51 mm x 102 mm (2 in x 4 in) lumber spaced 406 mm (16 in) on center. The ceiling and interior wall surfaces consisted of gypsum wallboard. Single 51 mm x 102 mm (2 in x 4 in) lumber formed the top and bottom plates for all vertical framing. The principal components of the floor were plywood subflooring-underlayment and 51 mm x 200 mm (2 in x 8 in) wood joists. Exterior wall surfaces consisted of 11 mm (7/16 in) hardboard siding backed by a layer of gypsum wallboard sheathing.

The module was seated upon a wood base, which was anchored to the laboratory test floor. To simulate the joint between the first and second story modules in the actual townhouse, the perimeter beam for the support assembly was chosen to match that specified for the first floor ceiling in the townhouse.

In the first racking test, service life racking, concentrated loads were applied to the front face of the module at the eave line. The positions of the loading rams were selected to simulate the racking effect of a uniform wind pressure distribution along the length of the front face (see Figure 3.2). Subsequent to the service life racking test, several floor tests were performed to determine the damping behavior under vibration of short duration and to measure the displacement under sustained load. Then the cathedral roof section was severed from the rest of the module. The remaining racking tests were performed on a nominal 12 m (40 ft) long structure.

For the one thousand-cycle and reversed cyclic load tests, two electro-servo hydraulic rams, located at the one-third points along the front of the truncated module, were used. In the one thousand-cycle racking test, the load varied with time in accordance with a half-sine wave. In the reversed-cycle racking test, five cycles of lateral load were applied in accordance with a sinusoidal forcing function. The three selected frequencies were 0.1 Hz, 0.5 Hz, and 1.0 Hz. The maximum amplitude of ram displacement was 4 mm (0.15 in) in one direction and 3 mm (0.10 in) in the opposite direction. As there were no available requirements for deflection limitation for a structure subjected to this type of loading, strictly visual observations were recorded. The five reversals of lateral load did not cause any apparent structural damage to the test module or to the horizontal joint at its base.

For the racking to capacity (failure) test, two additional hydraulic actuators were positioned at the ends of the truncated module, making a total of four loading points. The racking to capacity test was terminated when the bond between the base of the module and the wood support assembly was broken. There were also several local failures in the members comprising the support assembly, although the superstructure of the module could still resist additional lateral load.

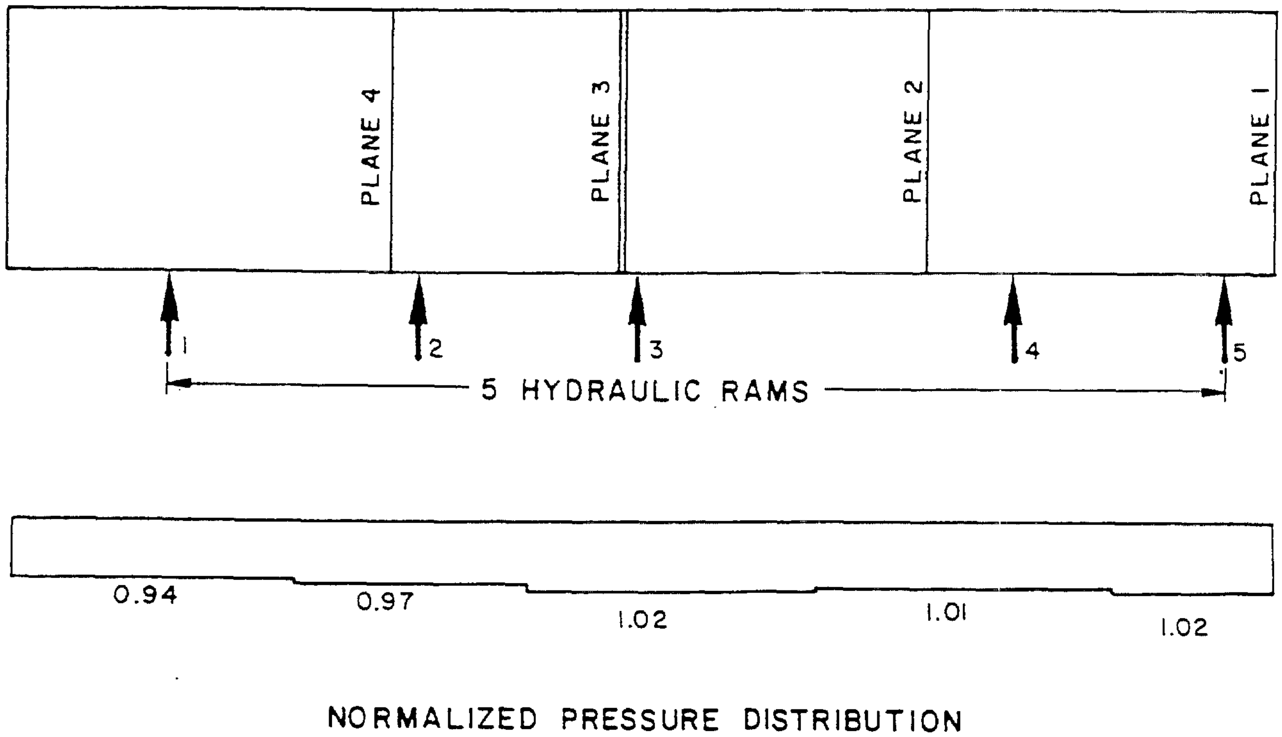


Figure 3.2 Normalized Pressure Distribution - Test 1
(after Yancey and Somes, 1973).

An analytical model (Figure 3.3) was derived to relate drift at the second floor to static wind pressure acting normal to the longitudinal walls of the erected townhouse. The equivalent lateral force at a given level is denoted by P in the figure. Derived drift was plotted versus simulated wind pressure for three of the four shear wall planes (Figure 3.4). The drift versus wind pressure curves were used to determine the maximum wind pressure corresponding to the conventionally accepted maximum allowable drift. It was concluded that the drift limit of height (h)/500 could be satisfied for wind pressures up to 1005 Pa (21 lb/ft^2).

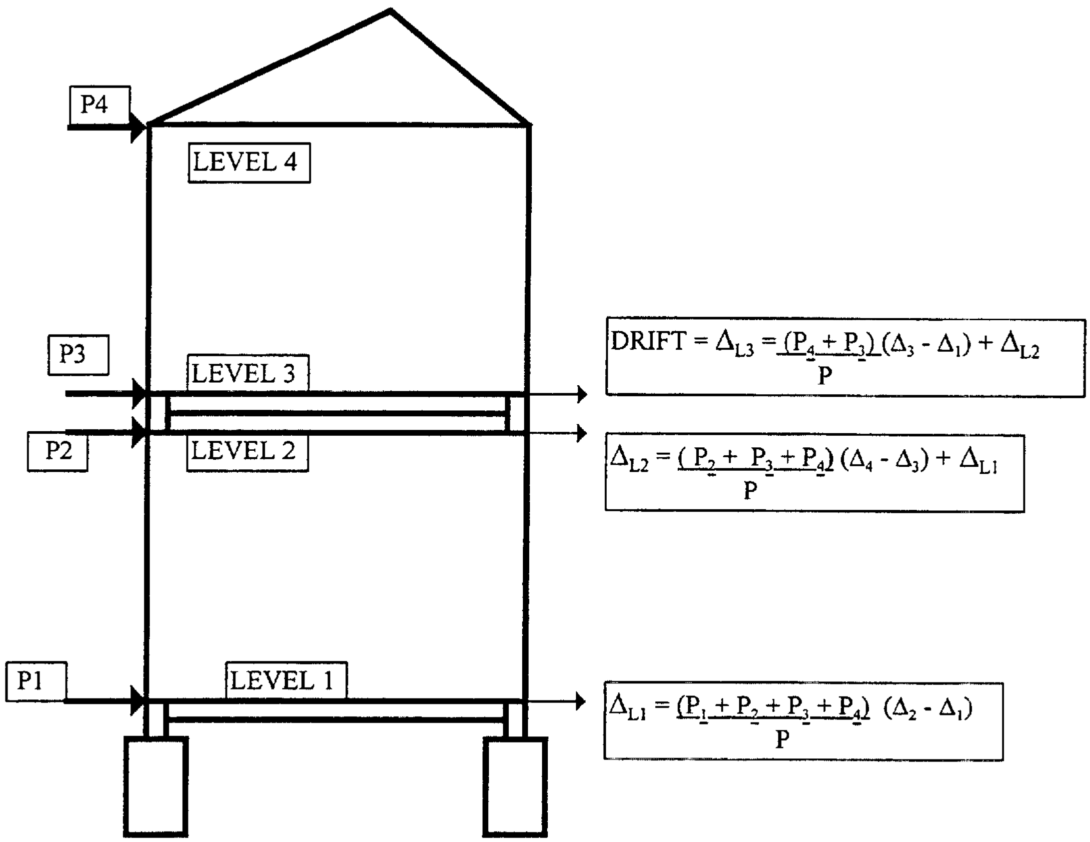


Figure 3.3 Analytical Model for Drift Computation (after Yancey and Somes, 1973).

Based on the results of the one-thousand cycle racking test, it was concluded that the module responded elastically to the test loading after the slack was removed. Because no standard test methods existed for full-scale house testing, it was recommended that new or improved test methods be developed.

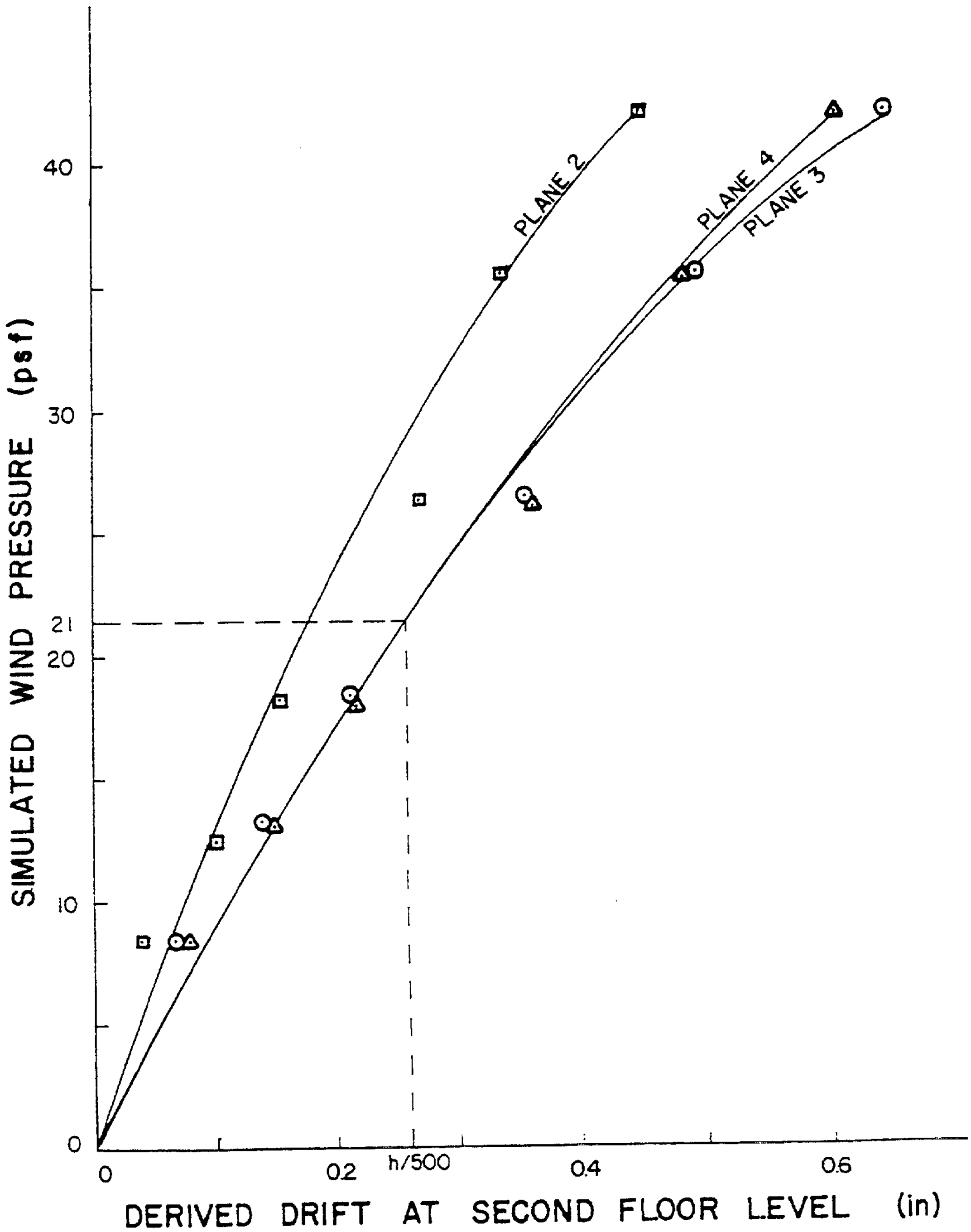


Figure 3.4 Derived Drift versus Simulated Wind Pressure (after Yancey and Somes, 1973).

3.2.3 Testing of a Full-Scale House Under Simulated Snowloads and Windloads (Tuomi and McCutcheon, 1974)

The motivation for this study was the need to determine the interaction between the structural components in low-rise buildings when the buildings are subjected to combined wind and snow loads. Although prescriptive standards and guidelines for house construction had resulted in structures that performed adequately, it was not possible to determine whether these structures are over-designed because of the lack of engineering design principles in the construction of conventional houses. Moreover, there were no standard procedures for determining baseline criteria for the performance of wood frame structures.

The primary objective of the study was to determine the structural response of a conventional wood-frame house to simulated snow and wind loads. A full-scale house, constructed in accordance with the minimum requirements of the Federal Housing Administration (1966), was subjected to a series of six tests under horizontal forces to assess the racking properties of the house. One additional test was also performed to determine the strength of the roof system. Concentrated loads were applied in the first five racking tests to determine the lateral stiffness during progressive stages of construction. The five stages of testing included: 1) racking test of each end wall covered only with plywood sheathing, 2) racking test of each end wall after the installation of two windows and a door, 3) racking test of the end walls after the installation of interior gypsum wallboard and exterior wood siding, 4) racking test of the structure consisting of two end walls and two side walls, and 5) racking test of the complete house after the addition of the roof system. The sixth test employed uniformly distributed lateral loads applied to one wall to determine both stiffness and strength.

The test house was 7.3 m (24 ft) wide in the roof truss direction and 4.9 m (16 ft) long. The two 7.3 m (24 ft) end walls (parallel to the direction of horizontal load) each contained two windows and one door. The walls were identical in their construction except that the door openings were positioned on opposite ends with respect to the loaded end. The walls were built with 51 mm x 102 mm (2 in x 4 in) studs spaced at 406 mm (16 in) on center. Double end studs, double top plates, and single bottom (sole) plates were used. Exterior sheathing consisted of 1.2 m x 2.4 m (4 ft x 8 ft) sheets of plywood, 10 mm (3/8 in) thick. The interior wall covering was 13 mm (1/2 in) thick gypsum wallboard. Both the plywood and gypsum wallboard sheets were attached with their longitudinal axes oriented vertically. Western red cedar siding was used for the exterior cladding. The 13 mm x 152mm (1/2 in x 6 in) siding boards were attached horizontally.

The roof system consisted of seven W-type trussed rafters spaced at 0.6 m (2 ft) on center and two gable end sections. The roof trusses spanned 7.3 m (24 ft). Roof sheathing was 10 mm (3/8 in) thick plywood sheets attached normal to the trusses. The ceiling consisted of 13 mm (1/2 in) thick gypsum wallboard installed perpendicular to the bottom chord of the trusses.

The floor was constructed using 51 mm x 152 mm (2 in x 6 in) joists spaced 406 mm (16 in) on center and 13 mm (1/2 in) thick plywood subflooring. The plywood sheets were attached with face grain perpendicular to the joists.

The house was supported on a "pseudo foundation" consisting of four steel beams. The base of the house was bolted to the steel beams with 13 mm (1/2 in) diameter bolts located at the four corners and at 2.4 m (8 ft) intervals along each side.

A structural loading frame, consisting of steel beams and columns, was erected and secured to the laboratory test floor. Hydraulic jacks were attached to the loading frame to apply horizontal loads during the first five racking tests. For the final racking test, an air bag system was used to apply a uniform load to one of the 4.9 m (16 ft) long walls. Figure 3.5 shows the position of the loading jacks and location of the displacement gages for Stage 5 testing (racking test of the complete house).

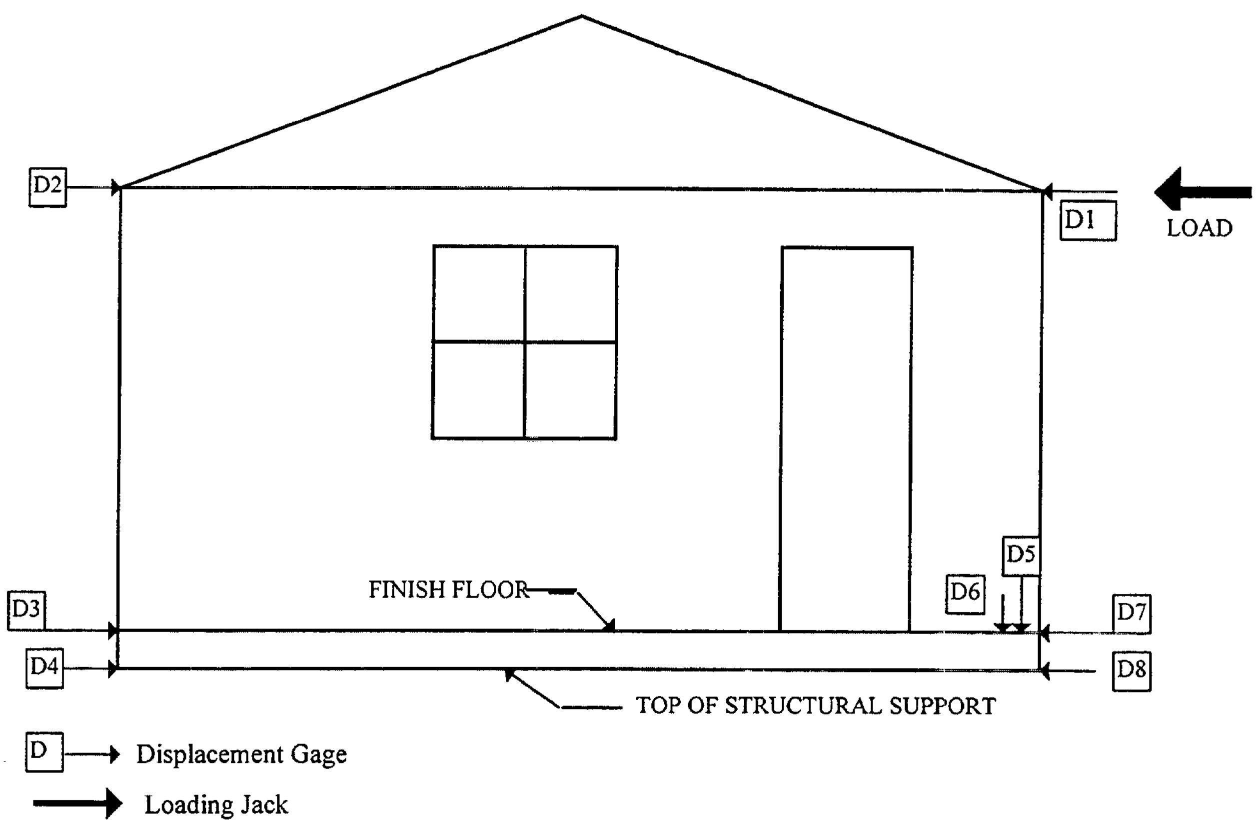


Figure 3.5 Location of Loading Jacks and Displacement Gages
(after Tuomi and McCutcheon, 1974).

For the first five stages of racking tests, it was desirable to restrict maximum horizontal displacement to prevent yielding in the structure. It had been previously determined from laboratory wall racking tests that limiting the displacement to a maximum of 4 mm (0.15 in) should ensure that objective. The load was applied using displacement increments of 0.6 mm (0.025 in) and was released after each new displacement level was reached and the residual displacement (set) was recorded after a relaxation period. Progressive increments continued until net horizontal displacement was in the range of 3 mm to 4 mm (0.125 in to 0.150 in), at which time a cycle was complete. The load was cycled four additional times using the same incremental displacement procedure.

Stage 6 loading was applied in 250 Pa (5 lb/ft^2) increments to a pressure of 1900 Pa (40 lb/ft^2). The pressure was released following each load increment. The pressure was cycled three additional times to 1900 Pa (40 lb/ft^2) and on the third cycle continued until failure occurred.

During Stage 6 loading, the first failure occurred at the sole plate of the loaded wall at a pressure of 3000 Pa (63 lb/ft^2). The sole plate split in line with the nails which secured the loaded wall to the floor system. As there was no significant racking distortion at that point, the loaded wall was reinforced and loading continued. At a pressure of 5900 Pa (123 lb/ft^2), the house slid off the sill plate and testing was terminated. The failure pressure was equivalent to that caused by a wind velocity of 98 m/s (220 miles/hour). Regression curves were derived for the load-deflection relationship for each end wall for Stages 5 and 6. The curves are in the form:

$$P = a\,\Delta^b$$

where, P represents the racking load, Δ is net horizontal deflection at load P, and a, b are coefficients determined by a least squares analysis.

A load-displacement curve for one of the end walls during Stage 5 is shown in Figure 3.6. As a means of evaluating the serviceability performance of the house, windows and doors were checked for operability throughout the uniform load test. The windows remained operable throughout most of the test, but the door near the loaded end started to bind at wall deflections above 2.5 mm (0.10 in) or 0.10% drift. The door, however, could be opened and closed with slight force. The original top clearance was about 0.8 mm (1/32 in). The weak links observed in this study were the connection systems between the sole plate and the floor, and the sill plate connections. Based on the deflected shape of the loaded wall during the uniform load test (Stage 6), it was concluded that the stiffness of the wall sheathing and siding was sufficient to cause the loaded wall to act as a plate resulting in approximately three-eighths of the total windward wall force being resisted by each of the end shear walls. It was concluded that this fraction of lateral load resistance would likely change for a longer house with interior partitions which would act as intermediate shear walls.

The authors noted the difficulty of comparing their test results with those from the limited number of other full-scale house studies due to a wide variation in the types of construction, test procedures and loading conditions. Therefore, it was recommended that standard guidelines be developed for future testing of full-scale houses. The guidelines should define the type of construction of the conventional house, specify standard test procedures and describe load conditions that reasonably simulate the actual forces imposed on the house in service.

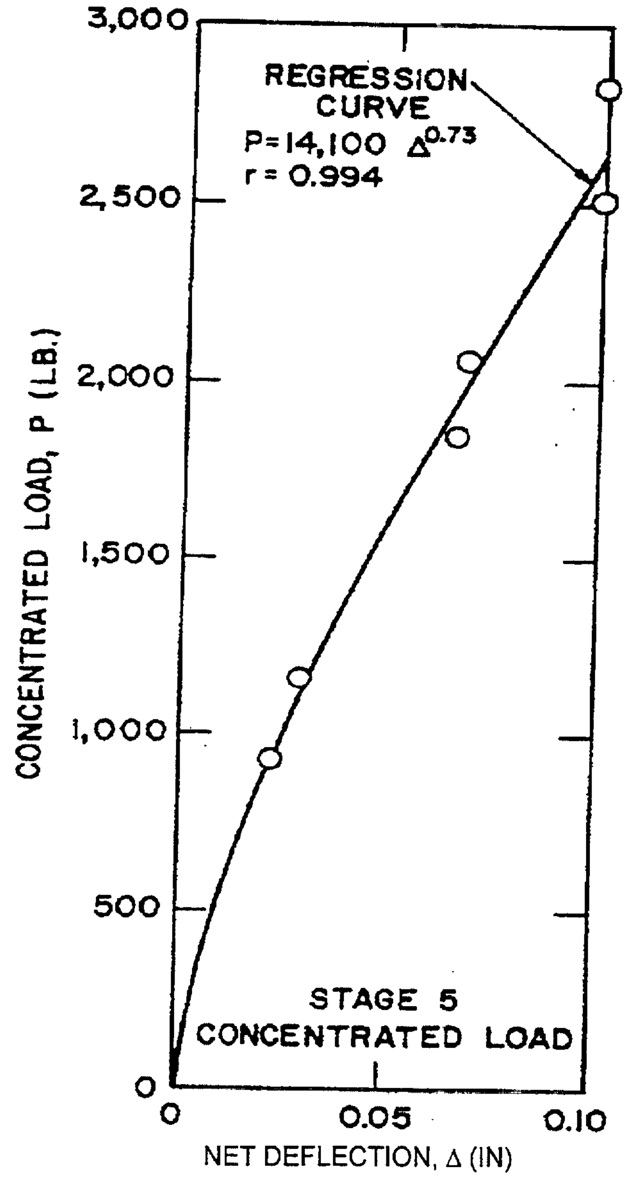


Figure 3.6 Regression Curve for an End Wall
(after Tuomi and McCutcheon, 1974).

3.2.4 Simulated Wind Tests on a House (Boughton and Reardon, 1982)

In the early 1980s, the Cyclone Structural Testing Station at James Cook University, Australia, initiated a research program to test full-scale houses under simulated high wind loads (Boughton and Reardon, 1982). The study involved the field testing of a house that had been condemned by the local housing authority for non-structural reasons. The test house was built in the early 1940s in accordance with U.S. Air Force building specifications. The house was not originally used for lodging, but was later converted into two dwelling units. The single-story house, approximately 162 m^2 (1750 sq. ft) in plan, was erected on timber "stumps."

A series of racking tests were conducted with the primary objective of determining the distribution of forces through the house when subjected to high wind loads. A second objective was to identify the weak links in the chain of structural elements that transmit the racking forces into the foundation, and a third was to relate the performance of structural elements assembled in the house to the performance of similar elements in laboratory tests.

The construction of the test house was not typical of single-family houses built in the United States nor typical of modern Australian single-family housing. The roof system consisted of bolted W-trusses with 51 mm x 125 mm (2 in x 6 in) wood purlins which supported corrugated steel roofing. The trusses spanned approximately 6.7 m (22 ft) and were spaced 3 m (10 ft) on center. Wood studs, 76 mm x 76 mm (3 in x 3 in), supported the ends of the trusses. A window was placed between the wood studs. Diagonal let-in braces were nailed to the studs.

Interior wall sheathing consisted primarily of plywood, the exception being that asbestos cement panels were used in the bath and laundry rooms. Exterior siding consisted of 30 mm x 250 mm (1 1/4 in x 10 in) horizontal boards nailed directly to the studs.

Four steel frames with bolted connections were erected on one side of the house to apply horizontal loads at one of three levels: top plate level, mid-height of the studs, and floor joist level. Uplift loads were also applied to one side of the sloped roof. A schematic of the test setup for horizontal load applied at the top plate level is shown in Figure 3.7.

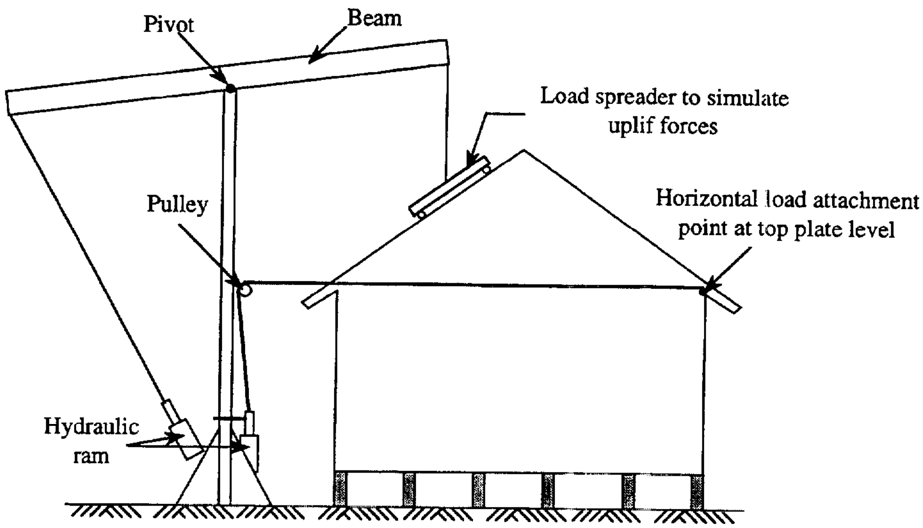


Figure 3.7 Schematic of Test Setup for Horizontal Load at the Top Plate and Vertical Load (after Boughton and Reardon, 1982).

Six stiffness tests were conducted with the number of load increments ranging from 10 to 20. The house was tested to determine its lateral stiffness for intact condition and after successive removal of five structural elements:

1. Weatherboard siding from the loaded exterior wall,
2. All ceiling sheathing,
3. Roof sheathing from one side of the roof,
4. Roof sheathing from the other side of the roof, and
5. All ceiling battens.

For the stiffness tests mentioned above, a single point load was applied at the top plate, in close proximity to an internal shear wall. The results from the stiffness test conducted with all elements intact permitted the analytical determination of the lateral load distribution capacity of the roof assembly. Also, lateral strength tests were run with the horizontal load applied at the top plate, mid-height of the studs, and at the floor joist level, respectively. Additional strength tests were conducted on the wall studs after the exterior siding was removed. A roof uplift test was conducted with two of the structural loading frames simultaneously applying load to one slope of the roof (see Figure 3.7).

Increments of load were applied using manually actuated hydraulic jacks. Strength tests continued until failure occurred in a structural element. For stiffness tests, load increments increased until a pre-determined deflection near the upper end of the elastic range was reached. The load was then released slowly and the house was allowed to return to its original position.

One internal shear wall was removed from the house and subjected to a racking strength test in the laboratory. The wall was loaded monotonically to the approximate load levels sustained by the walls during various in-situ stiffness tests. The wall was then unloaded and the permanent deformation measured. Subsequently, the wall was loaded again until failure occurred as indicated by local failure of fasteners and buckling of the interior plywood sheathing.

Full-scale test loads for both roof and racking tests were determined by first testing a 1:50 scale model of the house in a wind tunnel. The external pressure was based on the provisions of the Australian Wind Loading Code (1981) for a 50-year return period in Townsville, the site of the test house. The Australian Code's lateral and uplift pressures were compared to those calculated for Cyclone Althea, one of three cyclones that the test house withstood. The code-derived pressures were about 40% higher than those attributed to Cyclone Althea.

Although the test house had successfully withstood three tropical cyclones during its 40-year life, some structural deficiencies were observed during testing that may result in unacceptable performance according to the 1981 Australian Code design wind loads. The purlin-to-top chord of the truss connection was an area of deficiency. Reliance on the building weight to prevent the building from sliding on the stumps (piles) also resulted in an unacceptable safety factor (less than 2.0) according to the then current code. The bending strength of the studs appeared to significantly exceed the strength needed to resist current (1981) code design wind loads. In addition, the roof assembly had adequate in-plane strength to distribute the lateral forces to the shear walls. Based on the analyses of stiffness test results, it was concluded that approximately 60% of the lateral load was transferred to the shear walls through the roof sheathing and ceiling systems. The remainder was transmitted directly to the internal shear walls or was resisted in flexure by the windward wall. Based on the lateral load strength tests, safety factors of 3.37 and 4.57 were derived with respect to the Australian Standards Code and Cyclone Althea peak wind speeds, respectively.

3.2.5 Load Sharing Characteristics of Three-Dimensional Wood Diaphragms (Phillips, 1990)

Most of the preceding research on low-rise buildings had focused on testing and analysis of building components, and only a few studies had examined the response of a full-scale, three-dimensional building. The study by Phillips (1990) was intended to collect information that could be used in a reliability-based design methodology.

One of the primary objectives of the study by Phillips was to determine the load sharing characteristics of wood diaphragm systems. Other objectives included the determination of the hysteretic response of low-rise wood-frame structures under cyclic lateral loading, the determination of the load-slip behavior of typical joints within the structure, and the evaluation of the stiffness of two-dimensional wall diaphragms with different sheathing materials.

The test structure was a 9.8 m (32 ft) long by 4.9 m (16 ft) wide house, comprised of two 4.9 m (16 ft) end walls, two full-width internal shear walls, two 9.8 m (32 ft) side walls, a joist floor assembly, and a trussed roof assembly (see Figures 3.8 and 3.9). The walls were 2.4 m (8 ft) high. Framing for the walls and the ceiling diaphragm consisted of 51 mm x 102 mm (2 in x 4 in) timber. One of the interior walls (wall three) was sheathed with 13 mm (1/2 in) gypsum board on both faces and the other (wall two) was sheathed with 13 mm (1/2 in) plywood on both faces. All of the perimeter walls were sheathed with 13 mm (1/2 in) gypsum board on the interior face and T1- 11 plywood siding on the exterior face. The floor diaphragm consisted of nominal 51 mm x 254 mm (2 in x 10 in) floor joists spaced at 406 mm (16 in) on center. Floor sheathing consisted of 16 mm (5/8 in) plywood. The roof consisted of seven W-shaped trusses, two gable trusses, 13 mm (1/2 in) plywood roof sheathing, and 13 mm (1/2 in) gypsum board on the ceiling. The top chords of the trusses were formed with 51 mm x 152 mm (2 in x 6 in) lumber; all other truss members were fabricated from 51 mm x 102 mm (2 in x 4 in) lumber.

The structure was loaded laterally at the end of each of four stages of construction. Prior to the four-phase test program, the load/slip behavior was measured for seventy single-nail joints. Testing was conducted on seven different types of joints [e.g. plywood sheathing and 51 mm x 102 mm (2 in x 4 in) framing lumber].

During Phase 1 of the lateral load test, the two 4.9 m (16 ft) end walls (walls one and four in Figure 3.9) and two internal shear walls (walls two and three in Figure 3.9) were tested individually after the attachment of one layer of sheathing. Each wall was supported laterally to prevent out-of-plane displacements. The quasi-static cyclic load tests took place after the walls were attached to the floor diaphragm. Three cycles of load were applied laterally up to a maximum of ± 3.6 kN (800 lb).

In Phase 2, each of the four shear walls (walls one to four in Figure 3.9) was tested again after the application of sheathing on the other face of the wall. As in Phase 1, three cycles of lateral load were applied up to a maximum of ± 3.6 kN (800 lb).

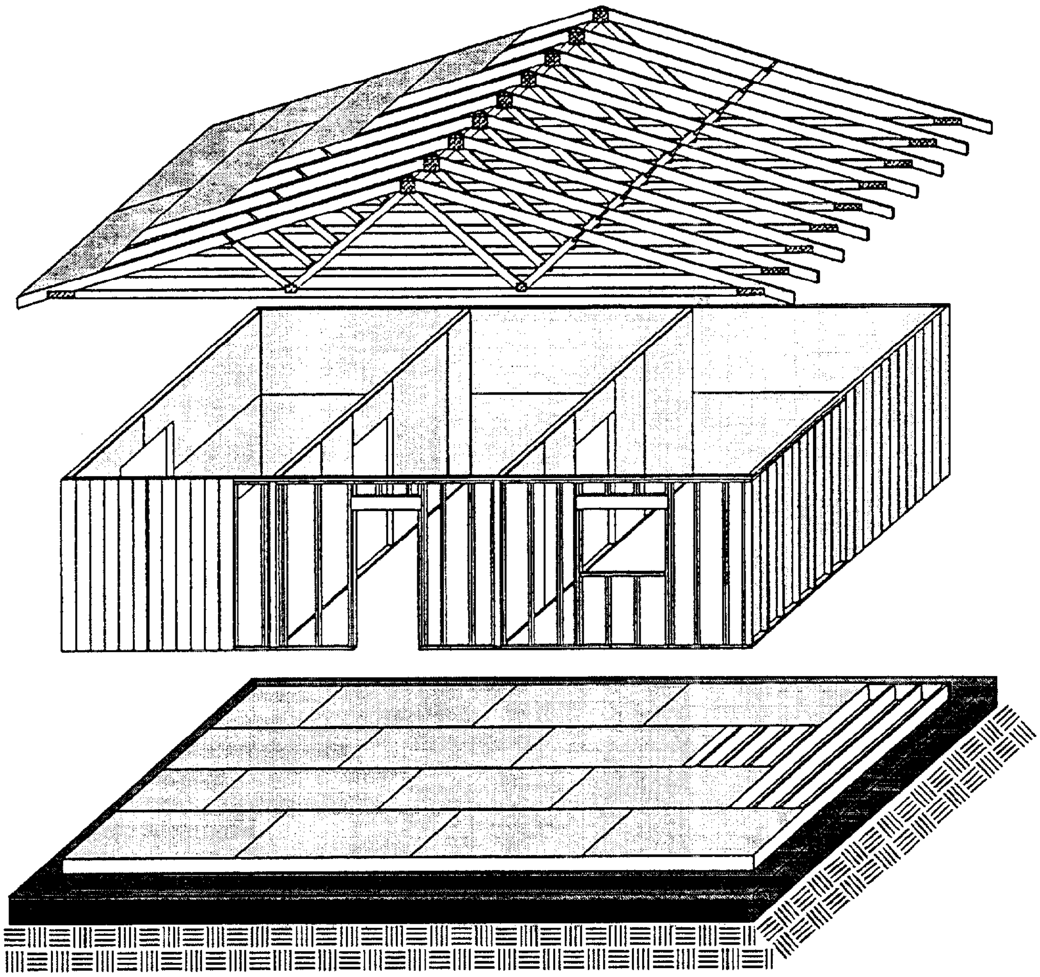


Figure 3.8 Exploded View of Three-Dimensional Test House
(after Phillips, 1990).

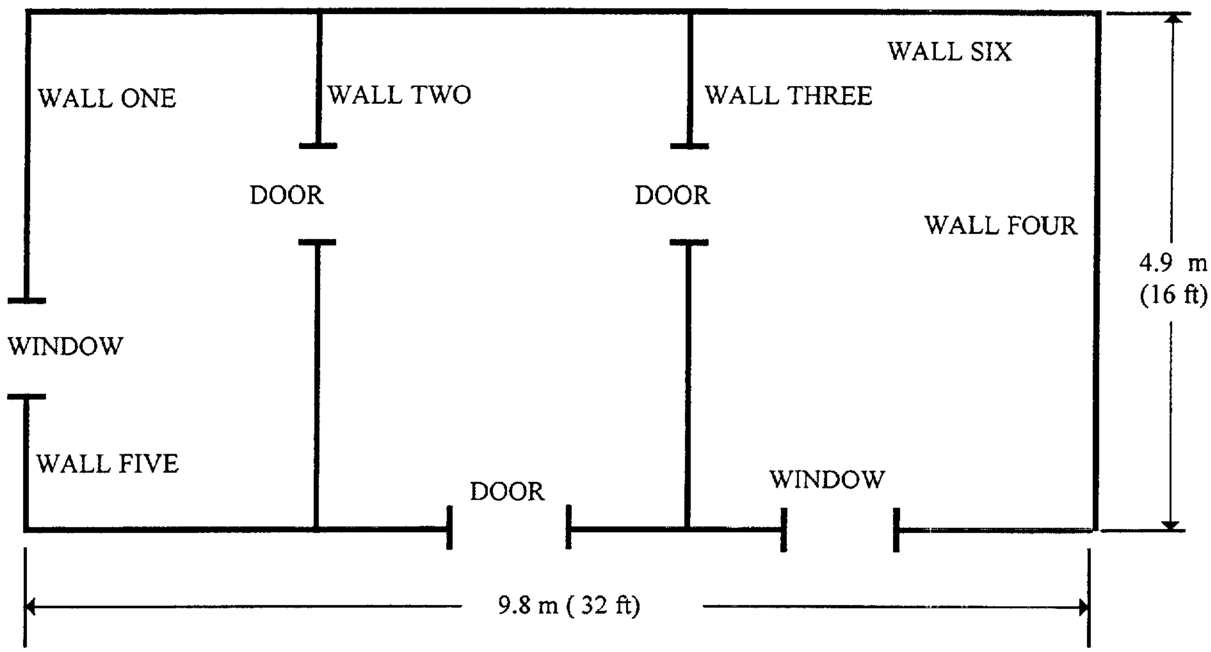


Figure 3.9 Floor Plan of Test House
(after Phillips, 1990).

The Phase 3 testing was conducted after the two 9.8-m (32-ft) side walls (walls five and six in Figure 3.9) were attached to the floor diaphragm, thus forming a box structure without a top. During Phase 3, three load tests were conducted: 1) three cycles of lateral load applied to one end wall and one internal shear wall (walls one and two in Figure 3.9), 2) three cycles of lateral load applied to the set consisting of walls three and four (see Figure 3.9), and 3) three cycles of lateral load applied simultaneously to the four shear walls. In each test, the maximum lateral load applied to each wall was ± 3.6 kN (800 lb). By limiting the maximum load in Phases 1- 3, linear behavior of the walls was observed and no damage occurred.

In Phase 4, the roof assembly was connected to the walls, thus forming a completely closed house. As in Phase 3, a series of three tests was conducted in Phase 4. The peak load per shear wall was ± 5.4 kN (1200 lb) and ± 8.1 kN (1800 lb) for Tests 1 and 2 respectively. In Test 3 of Phase 4, the load was cycled once at ± 9.8 kN (2200 lb) per shear wall and then increased in increments of 3.6 kN (800 lb) up to a maximum load of ± 31.2 kN (7000 lb) per shear wall.

In addition to recording the horizontal load, horizontal displacement at the top of the shear walls, slip along the bases of the walls, and uplift at the lower corners of the loaded edges were measured. Eight load cells - four horizontal and four vertical - were installed between the base of each of the four shear walls and the floor diaphragm to measure the forces in the walls.

From the results of Phases 1 and 2 testing, regression analyses were performed to determine the percent of the applied load measured by the internal load cells attached to shear walls 1 through 4. The linear equation used for the regression analyses was in form:

$$Y = m X,$$

where, X is the load applied to a single shear wall, and Y is the internal load for that shear wall. The fraction of the applied load measured by the internal load cells, coefficient m, was determined from the regression analyses which showed that the internal load cells generally matched the applied load within ± 5%.

Table 3.1 shows the variation in linear stiffness, base slip/gross displacement ratio, and base uplift/gross displacement ratio for the four shear walls, during Phases 1 and 2. The table also shows the increase in wall stiffness after a second layer of sheathing was added (compare Phase 1 results for a particular wall with Phase 2 results for the same wall).

Table 3.1 Linear Stiffness, Percent Slip and Percent Uplift for Phases 1 and 2 Testing (after Phillips, 1990).

Wall Number	**Approximate Linear Stiffness**		**Slip % of Gross Displacement**	**Uplift % of Gross Displacement**
	kN/m	lb/in		
Phase 1				
1	1400	8100	12	32
2	400	2125	7	17
3	500	3000	11	29
4	2100	12100	27	32
Phase 2				
1	3300	19000	24	38
2	750	4300	16	25
3	1100	6350	19	36
4	2900	16300	31	32

As in Phases 1 and 2, regression analyses were performed to calculate the percentage of lateral load carried by the four shear walls during Phase 3. Table 3.2 shows the values of the coefficient, m, for the three tests. All of the shear walls carried less than 100% of the load attributed to them due to the contribution of the transverse walls. Table 3.3 illustrates the variation in linear stiffness, slip/gross displacement ratio and uplift/gross displacement ratio for the four shear walls as obtained from Tests 1, 2, and 3.

Table 3.2 Coefficient m for Shear Walls, Phase 3 Tests
(after Phillips, 1990).

Wall Number	Test 1		Test 2		Test 3	
	m‡	R^2	m	R^2	m	R^2
1	0.885	0.998	Not loaded		0.888	0.994
2	0.691	0.994	Not loaded		0.700	0.990
3	Not loaded		0.691	0.986	0.700	0.990
4	Not loaded		0.787	0.990	0.782	0.999

‡ Linear coefficient

Table 3.3 Linear Stiffness, Percent Slip and Percent Uplift for Phase 3 Testing
(after Phillips, 1990).

Wall Number	Approximate Linear Stiffness		Slip % of Gross Displacement	Uplift % of Gross Displacement
	kN/m	lb/in		
1	3800	21500	34	6
2	800	4600	16	0
3	1500	8300	24	0
4	7000	40000	60	4

Table 3.4 shows the percent of applied load carried by the four walls during Test 3 of Phase 4 for the first and last cycles. To determine the actual amount of load carried by one of the walls, divide the percentage shown in Table 3.4 by 400% and multiply the quotient by the total applied load [e.g. (123/400 x 8800lb = 2706 lb) carried by Wall 1 in Cycle 1]. Note that for a given cycle, the percentages do not add up to 400% (i.e. four concentrated loads, each representing 100%) due to the plate action of the transverse wall.

Table 3.4 Lateral Load Distribution for Four Shear Walls, Phase 4 Tests (after Phillips, 1990).

Cycle Number	**Peak Load (= 400%)**		**% of Applied Load Carried by Each Wall**				
	kN	lb	Wall One	Wall Two	Wall Three	Wall Four	Summation (%)
1	+ 39.2	+8800	123	57	80	90	350
	-39.2	-8800	118	59	74	107	358
7	+124.6	+28000	120	55	93	98	366
	-124.6	-28000	120	59	90	104	373

Several conclusions reached by Phillips from comparing the load distributions for Phase 4 testing are:

1. The outside shear walls (one and four) received a larger percentage of the applied load.
2. Wall three (plywood on both sides) gained stiffness with increasing load.
3. As the load increased, a smaller percentage of the applied load (refer to the last column in Table 3.4) was carried by the transverse walls (five and six).
4. Plywood-sheathed walls were about 50% stiffer than gypsum-sheathed walls.

Phillips (1990) also concluded that the behavior of roof and ceiling assemblies was much closer to that of a purely rigid diaphragm than that of a flexible diaphragm. Thus, the lateral load was distributed to the shear walls in proportion to their relative stiffnesses.

The results from the load-slip tests on the seven types of joint coupons were analyzed to derive a best fit equation. The equation is in the form:

$$P = a\,\Delta^{b},$$

where, P = load
Δ = slip displacement
a, b = coefficients determined by statistical analysis.

Figure 3.10 shows the curve fit to the test data for plywood sheathing coupon. The derived load-slip equations may be used in future analytical experiments.

Phillips recommended that existing test procedures for characterizing diaphragm behavior be improved. Once improved test procedures are adopted, several individual vertical (shear walls) and horizontal diaphragms need to be tested under dynamic loads, up to their ultimate strengths. Also, horizontal-to-vertical diaphragm assemblies should be tested under dynamic loads.

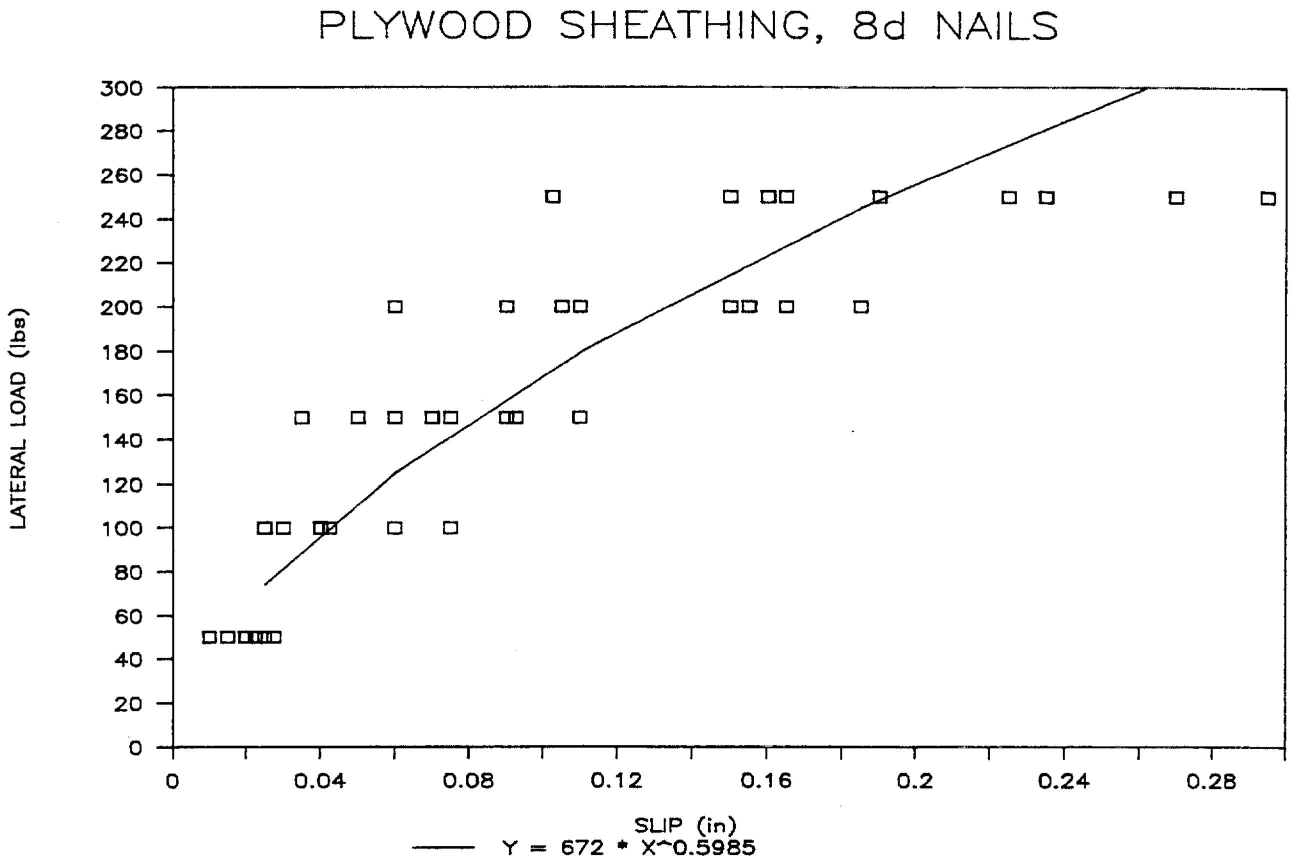


Figure 3.10 Power Curve Fit to Plywood Sheathing Data
(after Phillips, 1990).

3.3 SHEAR WALL (VERTICAL DIAPHRAGMS) TESTING

Timber shear walls, acting as vertical diaphragms, comprise one of the primary lateral-load-resisting components in both non-engineered and engineered wood-frame construction. There have been numerous studies of timber shear wall behavior, the overwhelming majority of which involved the application of monotonic loading and the use of ASTM Standard Method E72 ("Standard Methods of Conducting Strength Tests of Panels for Building Construction"). Standard E72 was adopted in the early 1940s to compare the contribution to lateral load resistance of plywood sheathing with that of traditional "let-in" corner bracing or diagonal lumber sheathing. Thus, ASTM E72 was never intended to facilitate the investigation of overall shear wall panel behavior. ASTM E564-76 ("Standard Practice of Load Test for Shear Resistance of Framed Walls for Buildings") was adopted in 1976 to provide a means of loading and anchoring light-frame shear walls. The following presents a summary of experimental studies conducted to investigate the behavior of shear walls.

3.3.1 Formulas for Wood Shear Walls (Easley, Foomani, and Dodds, 1982)

In their analytical study, Easley et al. (1982) developed formulas to predict the linear stiffness and the nonlinear lateral load-shear strain behavior of shear walls. In support of the analytical study, shear wall tests were conducted to determine the accuracy of these formulas. To provide additional comparative data, linear and nonlinear finite element analyses (refer to Chapter 4 in this report) were conducted on the shear walls which were tested.

Eight shear walls, consisting of duplicates of four different configurations, were tested under monotonic static loading to failure. All test walls were 2.4 m (8 ft) high and 3.7 m (12 ft) long and constructed with 51 mm x 102 mm (2 in x 4 in) framing lumber. The frames were built with double top and bottom plates. Plywood sheathing, 10 mm (3/8 in) thick, was nailed to one face of the frames. Test variables included: 1) stud spacing, 2) nail spacing along the perimeter of the wall, and 3) nail spacing along interior studs. Figure 3.11 shows a typical test setup.

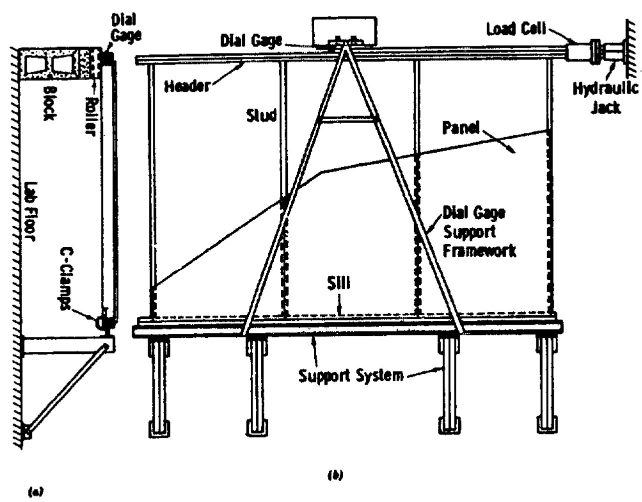


Figure 3.11 Shear Wall Test Setup: a) Side View; b) Plan View
(after Easley, Foomani, and Dodds, 1982)

Shear load versus shear strain plots obtained from the four sets of tests were used as a basis of comparison for the load-strain plots obtained from the new formulas and the finite element analysis. Two such comparative plots are presented in Figures 4.1 and 4.3. The comparisons indicated that the formulas provide relatively accurate predictions of load-strain behavior for the range of shear walls tested.

3.3.2 Contribution of Gypsum Wallboard to Racking Resistance of Light-Frame Walls (Wolfe, 1983)

Due to the relatively brittle nature and the low stiffness and strength of the gypsum core, gypsum wallboard is generally not considered to contribute to the racking resistance of light-frame buildings. This study was conducted to determine whether the contribution of gypsum wallboard-sheathed walls is significant enough to be included in a building's racking resistance. Another objective was to determine the influence of the orientation of wallboard (horizontal or vertical) on strength and stiffness.

Thirty wall specimens were subjected to monotonic load racking tests in accordance with ASTM Standard E564-76. Wall construction variables included: length, panel orientation, and type and orientation of diagonal bracing.

All walls were 2.4 m (8 ft) tall and constructed using 51 mm x 102 mm (2 in x 4 in) studs spaced 605 mm (24 in) on center, single top and bottom plates, and single end studs. Twenty-two of the 30 wall specimens were constructed with 13 mm (1/2 in) gypsum wallboard sheathing attached to one side of the framing. The other eight walls were constructed with a bare frame and diagonal bracing. Diagonal bracing was achieved by one of three methods: 1) 25 mm x 102 mm (1 in x 4 in) let-in wood braces stressed in tension, 2) 25 mm x 102 mm let-in wood braces stressed in compression, or 3) steel strap tension braces. Three wall lengths [2.4 m, 4.9 m, and 7.3 m (8 ft, 16 ft, and 24 ft)] were selected as the minimum necessary to observe the nonlinear relationships between length and attributes such as strength and stiffness. There was a total of 13 different combinations of the test variables.

A horizontal load was applied to an upper corner of each wall through the use of a single hydraulic actuator attached to the test frame. The lower corner of the loaded edge was held down by a steel fiber-core cable attached to the test frame base.

Loading was applied in two phases. For the first phase, load was applied until the top plate of the wall displaced 6 mm (0.25 in) horizontally. The load was then released and after 5-minutes, the second phase load was applied. For the second phase, the load was increased monotonically until the wall's resistance no longer increased with increasing displacement.

A parallel spring mathematical model was formulated to characterize the stiffness contributions of the gypsum sheathing and bracing elements. The model is in the form:

$$R_i = \Delta_i \,(K_{1,i} + K_{2,i} + \ldots\ldots K_{n,i})$$

where,

R_i = composite racking resistance at displacement Δ_i,

$K_{n,I}$ = secant modulus from the load-displacement curve of bracing element n at deformation Δ_i.

To test the accuracy of the parallel spring model, the results from braced frames without gypsum wallboard (Figure 3.12) were added to the results from unbraced walls with gypsum wallboard (Figure 3.13) and the sum was compared to the results from braced walls with gypsum wallboard sheathing. There was good agreement between the parallel spring model predictions and the test results from the composite walls. This limited comparative study suggested that the model can provide acceptable estimates of composite gypsum-sheathed wall performance, if the load-distribution curves are available for the independent element contributions.

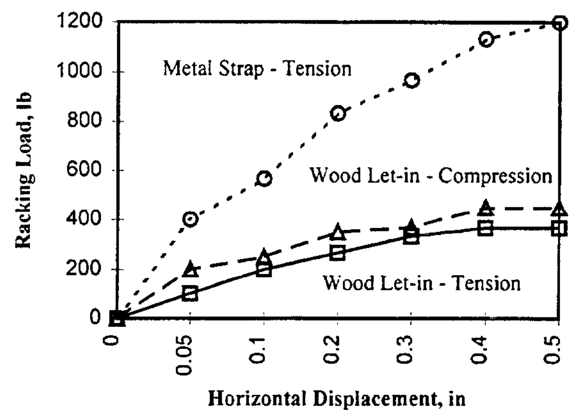


Figure 3.12 Average Load-Displacement Curves for Braced Frames Without Sheathing (after Wolfe,1983).

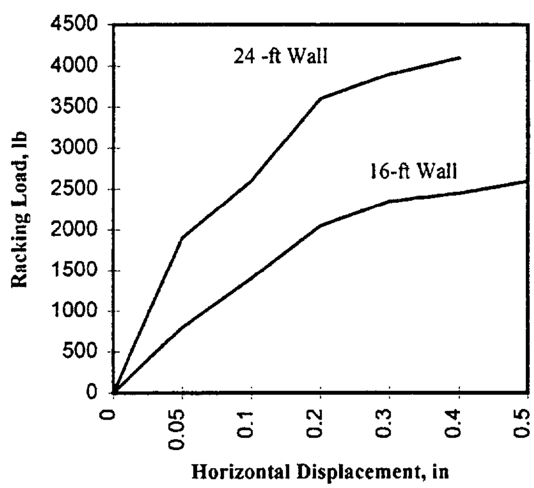


Figure 3.13 Load-Displacement Curves for Unbraced Gypsum-Sheathed Walls (after Wolfe,1983).

Figure 3.12 shows comparative load-displacement curves for braced frames without gypsum wallboard. The plots indicate that the frames braced with steel tension straps were significantly stronger and stiffer than those built with wood let-in braces.

It was concluded that gypsum wallboard can provide a significant contribution to wall racking resistance. Walls tested with the panels oriented horizontally were approximately 40% stronger and stiffer than those with panels oriented vertically. Based on the experimental relationships observed between resistance (strength and stiffness) and wall length, the following mathematical relationship was recommended:

$$\text{Resistance} = B\ (\text{Length})^{A}$$

in which values for the parameters A and B were obtained from least squares regression analysis. The value of A was inversely proportional to displacement, while the value of B increased with increased displacement.

In addition, it was concluded that the racking resistance (strength or stiffness) of test walls constructed with gypsum wallboard and a diagonal brace appeared to equal the sum of the resistances obtained for these two elements tested independently. Because of insufficient data, it was difficult to conclude what effect, if any, wall length had on the contribution to lateral bracing.

It was acknowledged that there were too few replications in this study to form a basis for design or code recommendations. Studies incorporating more replications are needed to confirm the relationships observed during this study and to relate individual wall contribution to complete-house performance.

3.3.3 Racking Performance of Light-Frame Walls Sheathed on Two Sides (Patton-Mallory, Gutkowski and Soltis, 1984)

The primary objective of this study was to determine whether one can predict the strength and stiffness of wood-frame walls sheathed on two sides from the test results of walls with only one side sheathed. In addition, the relative strengths of exterior walls (e.g. walls sheathed with plywood and gypsum wallboard) and interior walls (e.g. walls with gypsum wallboard on both sides) were investigated.

Small-size (i.e. one-fourth height) wall specimens were loaded to failure in shear in a test frame. Five wall configurations and four different lengths were included in the study. Ten replicas were conducted for each of the 20 combinations of wall type and length.

The framing material for the 200 wall specimens was the same, consisting of 51 mm x 102 mm (2 in x 4 in) Douglas-fir studs and top and bottom plates. The studs were spaced

305 mm (12 in) on center. The height of each specimen was 559 mm (22 in). The four wall lengths were 0.6 m, 1.2 m, 1.8 m and 2.4 m (2 ft, 4 ft, 6 ft, and 8 ft). The wall configurations consisted of five sheathing combinations: 1) plywood on one side of the frame, 2) plywood on both sides, 3) gypsum wallboard on one side, 4) gypsum on both sides, and 5) plywood on one side and gypsum on the other side.

Walls were loaded to failure (i.e. maximum load) or 51 mm (2 in) diagonal displacement, whichever occurred first. The test frame forced the wall specimens to deform in a parallelogram. A schematic drawing of a typical wall positioned in the test frame is shown in Figure 3.14.

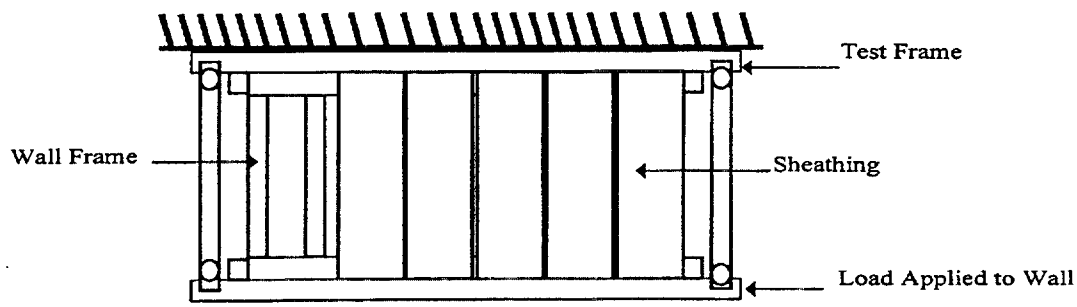


Figure 3.14 Test Setup for Wall Racking Tests
(after Patton-Mallory, Gutkowski, and Soltis, 1984).

The test results from two-sided walls were compared with those from one-sided walls to determine whether the strength and stiffness of two-sided walls can be predicted from one-sided wall test results. The strength comparisons are shown in Table 3.5. Generally, the values predicted using the one-sided wall test results were within 10% of the two-sided wall test results. The relative strengths and stiffnesses of interior and exterior walls were also compared. Interior walls are represented by test specimens constructed with gypsum wallboard attached on one side or on both sides. Based on comparative analyses, it was observed that under monotonic loading interior wall panels with gypsum on both sides provide between 57% and 67% of the racking resistance of double-sided plywood-gypsum exterior wall panels. Single gypsum wallboard sheathing provided at least 38% of the racking resistance of single plywood sheathing in small-scale tests using monotonic loads.

Table 3.5 Ultimate Strength for Five Wall Types and Four Lengths (after Patton-Mallory, Gutkowski, and Soltis, 1984).

	Wall Length							
	0.6 m		1.2 m		1.8 m		2.4 m	
Wall Configuration	Ultimate Strength (kN)	Standard Deviation (kN)	Ultimate Strength (kN)	Standard Deviation (kN)	Ultimate Strength (kN)	Standard Deviation (kN)	Ultimate Strength (kN)	Standard Deviation (kN)
Plywood one side	7.0	0.3	12.9	0.7	18.7	0.8	24.5	1.2
Plywood two sides	13.4	0.7	24.9	1.1	37.4	1.1	47.2	1.3
Gypsum one side	2.7	0.1	5.9	0.3	8.8	0.4	10.0	0.5
Gypsum two sides	5.1	0.3	12.0	0.4	17.4	0.4	20.9	0.9
Plywood/Gypsum	9.0	0.4	18.0	0.8	25.8	0.7	33.8	1.6

Several conclusions were drawn on the basis of this study: 1) the strength and stiffness of double-sided shear walls can be predicted by computing the sum of individual single-sided wall racking test results which agrees with the conclusion reached by Wolfe (1983); 2) racking resistance of plywood-sheathed walls appeared to be directly proportional to wall length; and 3) racking resistance of gypsum-sheathed walls was not directly proportional to wall length, but could be estimated by a linear relationship.

3.3.4 Light-Frame Shear Wall Length and Opening Effects (Patton-Mallory, Wolfe, and Soltis, 1985)

The main objective of this study was to show how shear wall performance varies with length. Secondary objectives included the determination of how individual sheathing layers and panel sections contribute to a wall's resistance to racking or in-plane shear forces. This study was conducted in conjunction with another study by Patton-Mallory, Gutkowski, and Soltis, (1984), which was discussed in Section 3.3.3. The same loading frame was used in both studies for testing small-size wall panels.

Prescriptive design guidelines for shear walls sheathed with plywood and/or gypsum were previously derived from racking tests on 2.4 m x 2.4 m (8 ft x 8 ft) walls with sheathing attached to one face of a wood frame. Sparse data were available from which to extrapolate racking resistance of walls with aspect ratios (length:width) greater than 1.

Eleven full-scale wall specimens with aspect ratios between 1 and 3 were tested under racking loads. The walls were sheathed with gypsum wallboard. The full-scale wall specimens were tested in accordance with ASTM E564-76. Two hundred smaller walls, with aspect ratios ranging from 1 to 4 were tested. There is no standard method for testing smaller wall specimens, but the E564 recommendations for load application and racking deformation measurement were followed. All of the small size specimens were 559 mm (22 in) high. Test variables included length [0.6 m, 1.2 m, 1.8 m and 2.4 m (2 ft, 4 ft, 6 ft, and 8 ft)] and sheathing configuration (single- and double-sheathed plywood walls, single- and double-sheathed gypsum walls and walls with gypsum wallboard on one face and plywood on the other). Ten walls were tested of each combination of length and sheathing configuration.

Full-scale test walls were 2.4 m (8 ft) high and sheathed with 13 mm (1/2 in) gypsum wallboard. Three wall lengths were used: 2.4 m, 4.9 m and 7.3 m (8 ft, 16 ft, and 24 ft). Six of the test walls were constructed with the gypsum wallboard sheathing oriented vertically. The other five walls had the wallboard oriented horizontally. In addition to solid walls, some specimens were constructed with windows and pre-hung doors.

The small-sized walls were tested to failure in a steel test frame under a "pure shear" load (refer to Figure 3.14 for a schematic of the test setup). The walls were forced to deform as parallelograms. The full-scale wall specimens were tested in a vertical orientation, using the test setup recommended in ASTM E564. The racking load was applied at the top plate in a two-phase procedure. First, static monotonic load was applied until the top plate moved horizontally 6 mm (0.25 in). The load was then completely released and no load was applied for a period of 5 minutes. During the second phase, a static monotonic load was applied until the wall failed. Diagonal elongations were recorded to obtain the horizontal displacement along the top edge of the wall.

Figure 3.15 shows that for both full-scale and small-size wall panels, the ultimate strength of walls sheathed on one side with gypsum wallboard was linearly proportional to wall length for aspect ratios between 1 and 3. In addition, Figure 3.16 shows that a linear relationship between strength and aspect ratio was obtained for small-size walls sheathed on: both sides with plywood, both sides with gypsum wallboard, one side with plywood and the other with gypsum and a single side only with plywood.

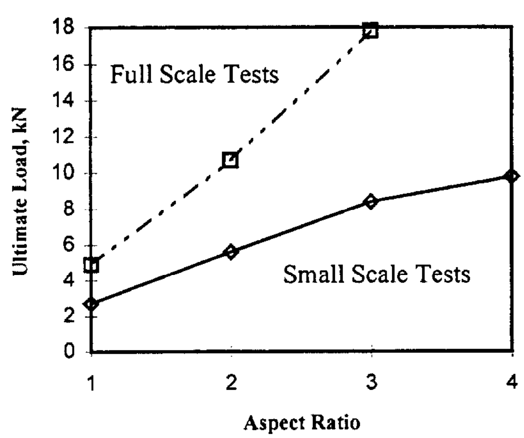


Figure 3.15 Ultimate Load versus Aspect Ratio for Gypsum Sheathing on One Side (after Patton-Mallory, Wolfe and Soltis, 1985).

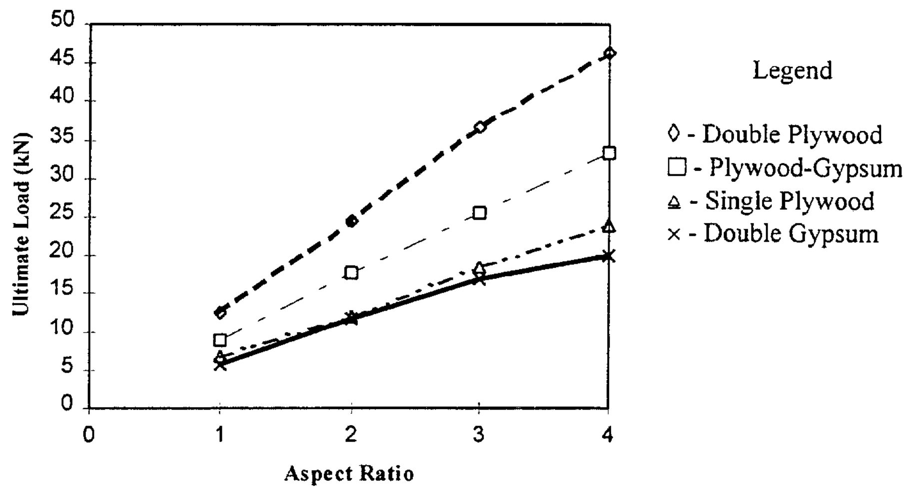


Figure 3.16 Ultimate Strength versus Aspect Ratio for Small Size Walls (after Patton-Mallory, Wolfe and Soltis, 1985).

Wall stiffness was compared with aspect ratio for full-scale gypsum-sheathed walls. The results of the comparison did not indicate that the stiffness of gypsum-sheathed walls was proportional to length. No such comparison was reported for full-scale plywood-sheathed walls.

To determine the effect of door and window openings, the load-displacement curve for a 7.3 m (24 ft) long wall with openings was compared to that for a similar wall without openings. The effective length of the wall with openings was about 72% of the total length. The ultimate strength of the wall with openings was about 70% of that of the continuous wall. Also, the actual load-displacement curve for the wall with openings was compared to a predictive curve obtained by using an "effective length ratio." The ratio, obtained by dividing the sum of uninterrupted lengths by the total wall length, was multiplied by the strength coordinate on the curve of a full-length wall without openings to obtain the predictive curve. The predicted load-displacement curve overestimated the stiffness up to about 80% of ultimate load; thereafter there was very good agreement up to failure.

Failure modes were compared qualitatively for full-scale and small-size wall tests. Full-scale walls with an aspect ratio of 1 and small-size walls with aspect ratios of 1 and 2 exhibited a symmetrical fastener deformation pattern. The wall specimens with larger aspect ratios exhibited shear failure in the fasteners primarily along the top and bottom edges of the wall panels.

Several conclusions from this study were:

1. The ultimate racking strength of gypsum-sheathed wall panels (single- and double-sided) was proportional to wall length for aspect ratios between 1 and 3. The racking stiffness

of gypsum-sheathed walls, however, was not linearly proportional to wall length. Racking strength of plywood-sheathed (single- and double-sided) walls was proportional to wall length for aspect ratios between 1 and 4.

2. The effective length of walls with window and door openings can be used to predict their racking strengths. The effective length approach, however, tended to over predict the stiffness of walls with door and window openings.

3. The racking stiffness and strength of a double-sheathed wall was equal to the sum of the resistances measured in tests of its components at a given displacement and at ultimate load when the components tested are single sheathed walls. Thus, $P_{doubled\text{-}sheathed} = P_1 + P_2$, where P_1 and P_2 were the strengths or stiffnesses of wall panels sheathed on one side only with either sheathing material 1 or 2.

4. Small size and full-scale walls sheathed on one side with plywood were slightly stronger than corresponding walls sheathed on both sides with gypsum.

3.3.5 Structural Behavior of Wood Shear Wall Assemblies (Nelson, Wheat, and Fowler, 1985)

This study focused on shear wall assemblies common to manufactured housing. The assemblies consisted of the shear wall and contiguous roof, floor, and side wall components (see Figure 3.17). The structural behavior under static racking loads was investigated to provide information to update current design practices.

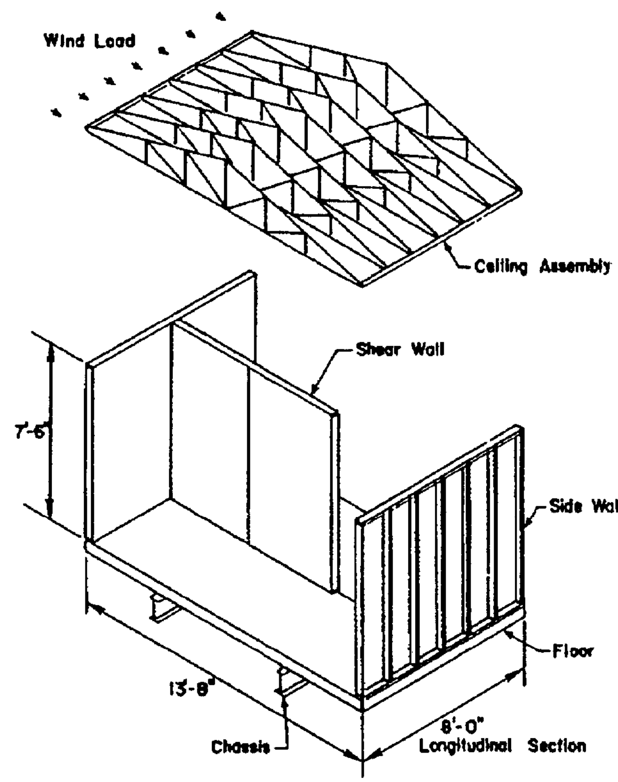


Figure 3.17 Pictorial View of Test Shear Wall Assembly
(after Nelson, Wheat and Fowler, 1985).

Seven shear wall assemblies were tested under monotonic racking loads until they reached their ultimate capacity. Horizontal load was applied approximately at the top of the wall. Test variables included: the length and location of the shear wall within the assembly, the number of glued sides of hardwood panels, and the number (single or double) of floor joists beneath the shear wall.

The test specimens consisted of full-scale, 2.4 m (8 ft) longitudinal segments of a typical single-wide mobile home. The width of the segment was 4.2 m (13 ft 8 in). In addition to a single shear wall, the 2.4 m (8 ft) segment was comprised of two steel chassis beams; 51 mm x 152 mm (2 in x 6 in) floor joists with 16 mm (5/8 in) particleboard floor decking; side walls constructed with 51 mm x 102 mm (2 in x 4 in) studs and sheathed with 4 mm (5/32 in) luaan hardwood paneling; and a ceiling assembly consisting of seven Howe trusses and 10 mm (3/8 in) plywood ceiling diaphragm. The shear walls were constructed with 51 mm x 76 mm (2 in x 3 in) studs, spaced at 406 mm (16 in) and sheathed with hardwood panel sheets. The shear walls measured either 2.4 m (8 ft) or 3.2 m (10 ft 6 in) in length. A typical set of load-displacement curves is shown in Figure 3.18.

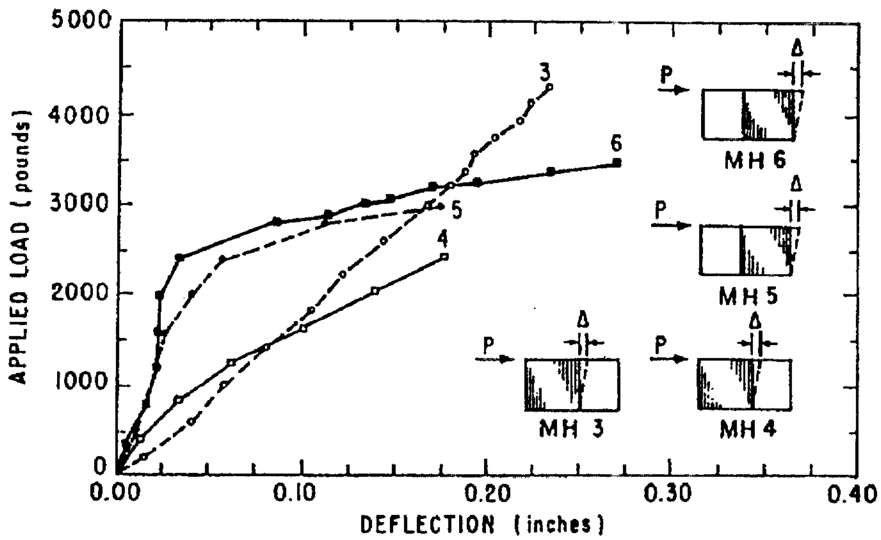


Figure 3.18 Load-Displacement Curves for Four Shear Wall Configurations (after Nelson, Wheat and Fowler, 1985).

The results of the test program led to following conclusions:

1. Shear walls located on the windward side of the assembly exhibited a higher ultimate capacity than those located on the leeward side because of the resistance to uplift provided by the transverse side wall,
2. The connection of the shear wall to the floor on the windward side of the wall was the most common location of failure, and
3. Slip measurements indicated that a significant amount of the load may be transferred into the side wall.
4. There was no significant difference in strength and stiffness of an assembly when hardwood paneling was glued to both sides of the shear wall versus one side.

3.3.6 Experimental Study on Behavior of Wooden Frames with Bearing Walls Subjected to Horizontal Load (Ohashi and Sakamoto, 1989)

This study consisted of testing two wall segments, each two stories high. The wall specimens were typical of Japanese wood-frame construction for residential dwellings. The objective of the study was to determine the behavior of the wood-frame walls when subjected to horizontal racking loads.

The test walls were subjected to reversed cyclic horizontal loading and a constant vertical load representing gravity load. The walls were tested at successive stages of construction: 1) first with the bare frame and let-in brace; 2) then with sheathing on one face; and 3) finally with sheathing on the other face.

The wall segments were 4.5 m (14 ft 9 in) long and 5.0 m (16 ft 5 in) high. They were described as wooden frames consisting of nominal 102 mm x 102 mm (4 in x 4 in) columns, beams of unspecified dimensions, and 25 mm x 102 mm (1 in x 4 in) let-in braces located at each end and each level of the walls. The columns were spaced approximately 0.9 m (3 ft) on center. The structural members were joined with nails or metal plates. One door and one window opening were present on each level. The two specimens differed in several construction details: the orientation of the lower level let-in braces, the type of sheathing materials, and connection details between the end columns and beams. Exterior sheathing consisted of either cemented chip hard board or a mortar finish applied over a wood lath. The interior sheathing consisted of either gypsum wallboard or gypsum lath board.

A horizontal load was applied to the test unit at the top and at the second floor through a hydraulic jack and a load divider beam. The cyclic load was applied according to a loading schedule. In the first cycle, the load was increased to the level causing a story drift of height/500. In subsequent cycles, the target displacement was $\sqrt{2}$ times the displacement of the previous cycle until the drift reached the limit of height/120, at which point the test was stopped.

Hysteresis curves of horizontal load versus interstory drift are presented for each stage of construction. Figure 3.19 shows the curves for the fully-constructed wall segments. The frame sheathed with gypsum wallboard on the interior and mortar on the exterior was more than ten times stronger and about three times stiffer than the bare frame. The frame sheathed with gypsum lath boards on the interior and cemented chip hard boards on the exterior was three times stronger and stiffer than the bare frame. The frame sheathed with cemented chip hard boards was more ductile than the frame with the mortar finish on the exterior.

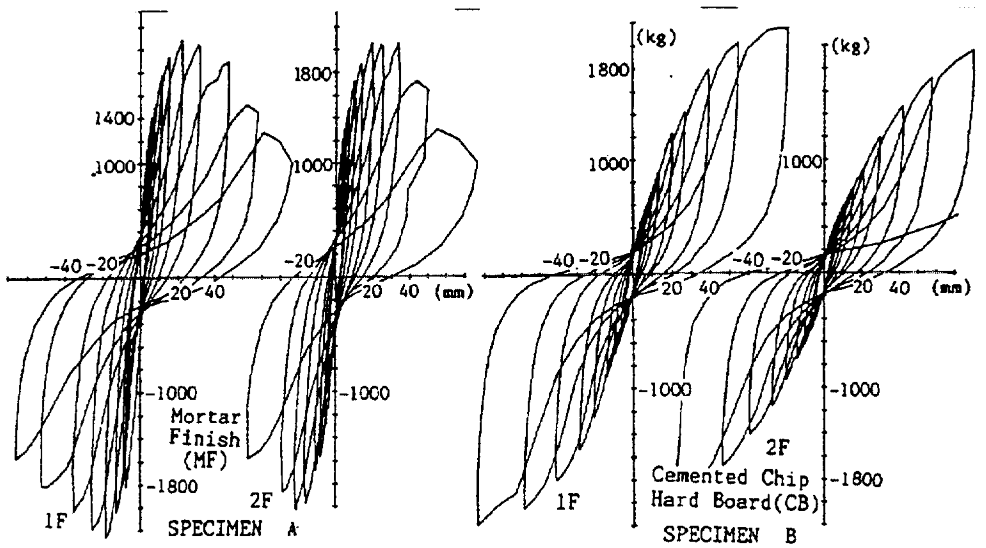


Figure 3.19 Hysteretic Response for Two-Story Wood Frames
(after Ohashi and Sakamoto,1989).

3.3.7 The Dynamic Response of Timber Shear Walls (Dolan, 1989)

The primary objective of this study was to develop a general finite element model to predict the response of timber shear walls to dynamic earthquake loading. The finite element model was intended as an analytical research and not a design tool. Other objectives of the study were: 1) to develop a numerical model to accurately predict the static, one-directional racking behavior of wood-frame shear walls, 2) to develop a mathematical model to predict the steady state response of wood-frame shear walls, 3) to develop a dynamic test method for wood-frame shear walls that realistically represents the expected loading during an earthquake, and 4) to investigate the critical connections in shear walls. To accomplish the objectives, connection tests and a racking test program on full-scale timber shear walls were conducted. The results from the full-scale tests were used to verify the numerical models. The analytical studies are discussed in Chapter 4 and the experimental program will be summarized in the following paragraphs.

Prior to performing the full-scale wall racking tests, two types of connections were tested. Three types of tests were conducted on the connection between framing and sheathing materials: 1) static one-directional tension, 2) static cyclic, and 3) dynamic cyclic. A triangular cyclic wave displacement pattern was used for the static cyclic test. The connection was tested by repeating the displacement pattern for four cycles, at three different displacement amplitudes. The elapsed time to failure was about 60 minutes. The dynamic cyclic test followed the same displacement pattern, but with a loading rate 30 times faster than that used in the static cyclic test. These tests were repeated for each of the possible grain orientations and material combinations used in the full-scale wall tests. Between three and five specimens were tested for each combination. Figure 3.20 shows the various test specimen configurations. P indicates the direction of load application. A typical load-deflection curve from the static one-directional tests is shown in Figure 3.21.

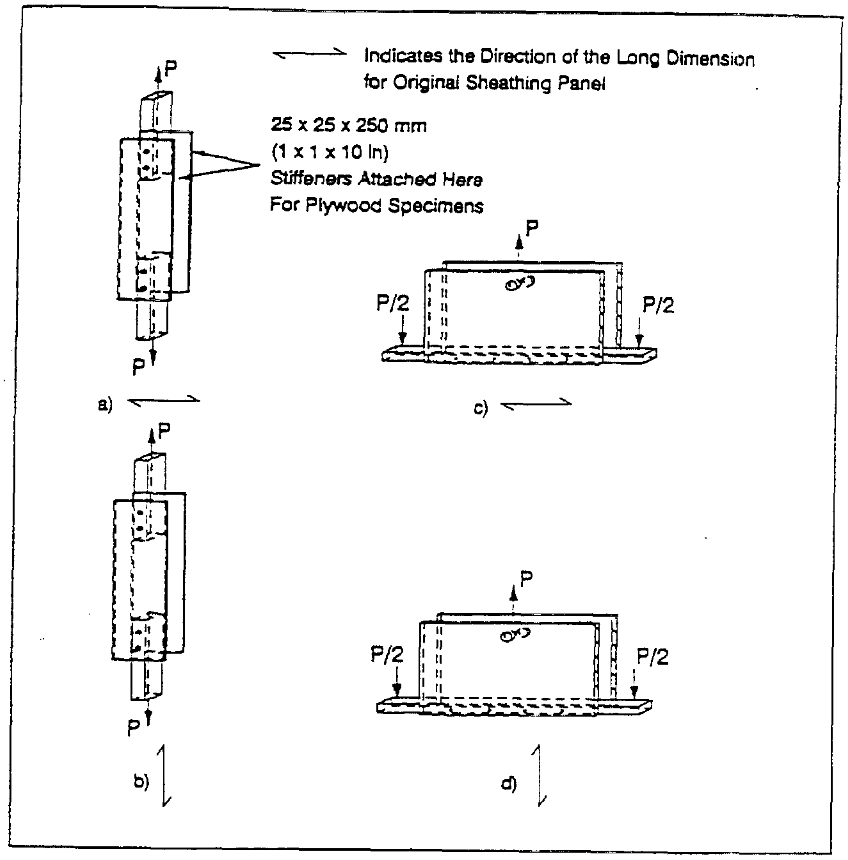


Figure 3.20 Sheathing Connection Test Specimen Configuration (after Dolan, 1989).

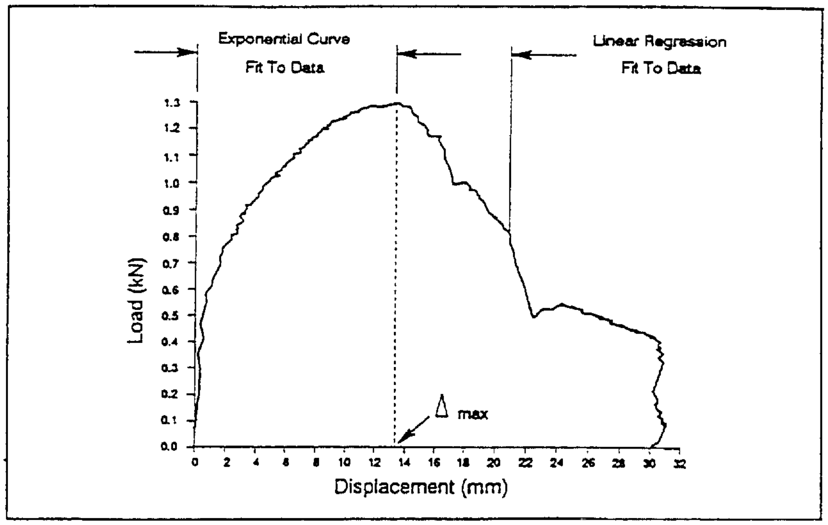


Figure 3.21 Typical Load-Displacement Curve for Static One-Directional Test (after Dolan, 1989).

The family of one-directional test curves was analyzed using a least squares regression method to fit the equation:

$$F_{con} = (P_o + K_2\Delta) \ \{1 - \exp(-K_o\Delta / P_0)\}$$

to the data, up to the peak load. In the equation, F_{con} represents the connection force and Δ is the slip displacement between the sheathing and the framing. K_o, K_2, and P_o are parameters shown in Figure 3.22a. Linear regression was applied to the post-peak data to obtain the descending slope of the load-deflection curve (refer to Figure 3.21).

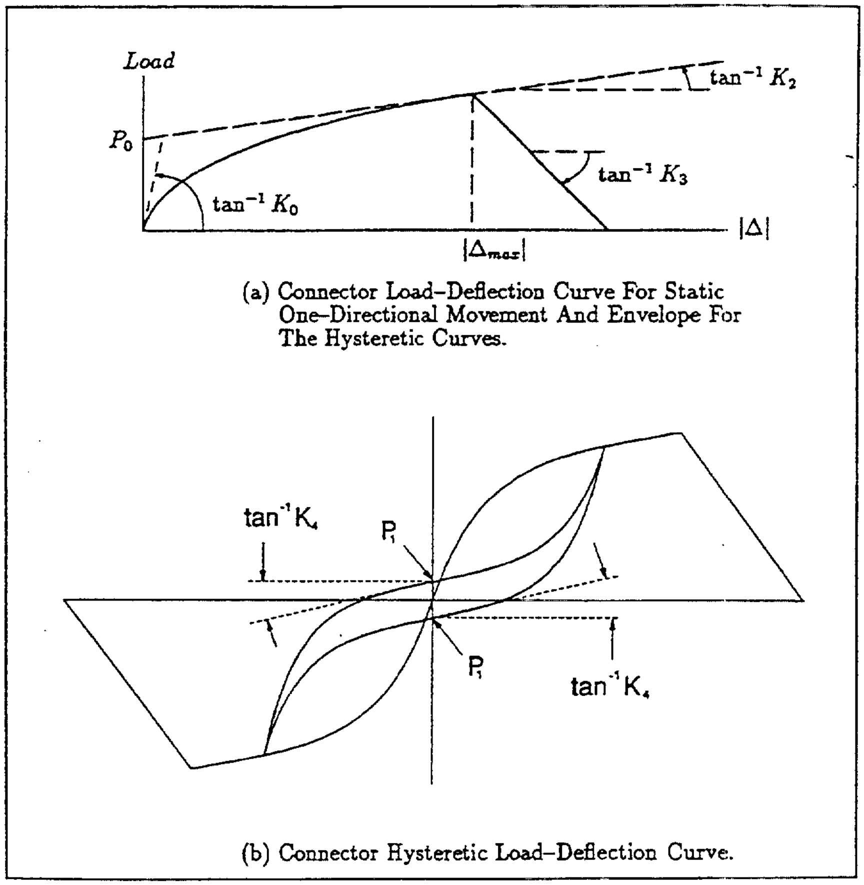


Figure 3.22 Sheathing Connector Load-Displacement Curve Parameters (after Dolan, 1989).

One of the conclusions drawn from the static one-directional test results was that the parameters, K_2, and P_o,, were insensitive to the direction of face grain orientation for both plywood and waferboard sheathings. The initial stiffness, K_o was a slightly dependent on the grain orientation of plywood, but was insensitive to grain orientation for waferboard. It was observed that average values of K_o and K_2 were higher for plywood than for waferboard, but the intercept value, P_o, for waferboard was higher than for plywood.

Figure 3.23 presents a typical load-displacement curve for a static cyclic test. The parameters K_o, K_2, and P_o were obtained from the first cycle curve at each displacement level and a least squares regression analysis was performed to fit the equation above to the data up to the peak load. K_o, K_2, and P_o were insensitive to the direction of face grain orientation for both plywood and waferboard sheathings. The parameters P_1 and K_4 in Figure 3.22b were found using linear regression on the data for the second through the fourth cycles at each displacement level between ± 2.5 mm (± 0.1 in). Dynamic cyclic test data were reduced in a similar procedure to that applied to the static cyclic test data.

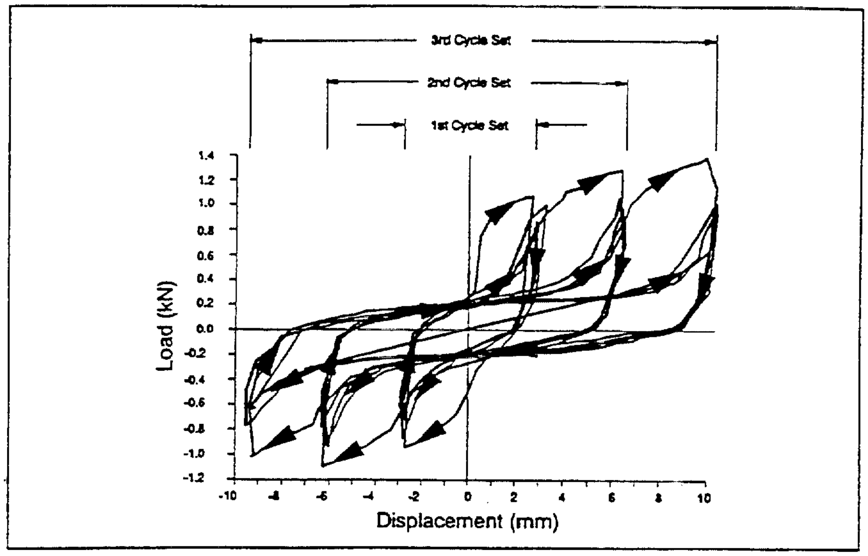


Figure 3.23 Typical Hysteresis Curves for Static Cyclic Sheathing Connection Test (after Dolan, 1989).

The other connection tests were performed on connections between studs and sole (bottom) plate and between studs and sill (top) plate. The primary purpose of the series of corner connection tests was to determine an average stiffness value for the corner connection in two of the finite element models. A typical corner connection is shown in Figure 3.24. Five connection specimens were loaded monotonically, in tension, to failure. A typical load-displacement curve is shown in Figure 3.25. Given the relatively linear behavior of the connections up to the ultimate load, linear regression analysis was performed to obtained the stiffness in tension. The average value of connection stiffness was 1.24 kN/mm (7.1 kip/in) and the coefficient of variation was 0.22.

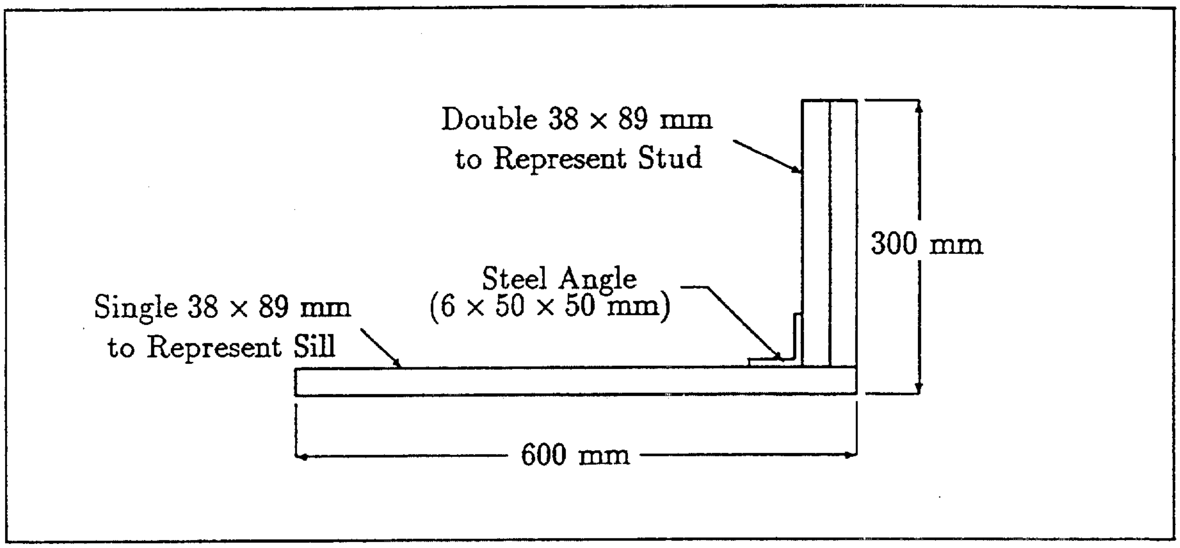


Figure 3.24 Corner Connection Test Specimen Configuration
(after Dolan, 1989).

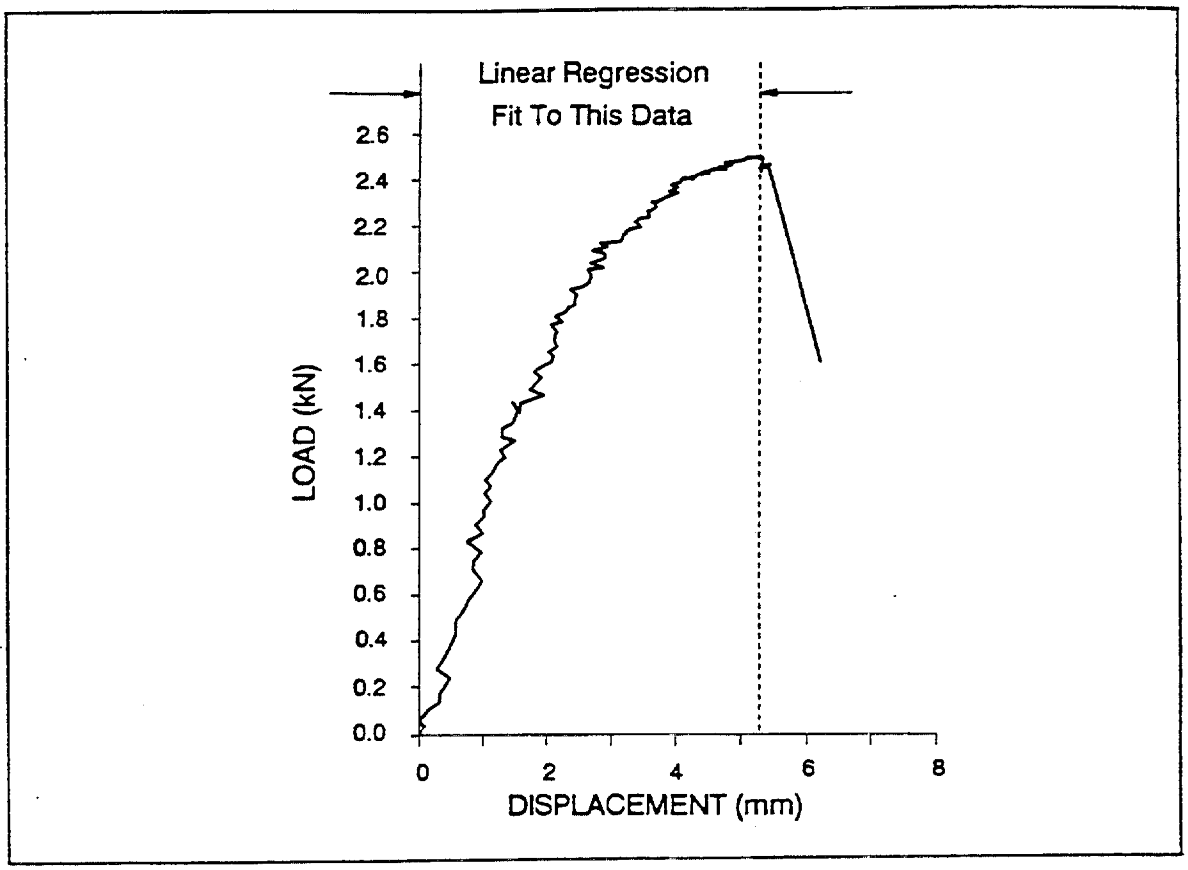


Figure 3.25 Typical Load-Displacement Curve from Corner Connection Test
(after Dolan, 1989).

A total of 42 timber-framed walls, measuring 2.4 m x 2.4 m (8 ft x 8 ft), were tested. The framing configuration was standardized while the type and orientation of the sheathing and the nail spacing were varied. The framing was sheathed on one face only with either 10 mm (3/8 in) plywood or 9.5 mm (3/8 in) waferboard. The sheathing was primarily oriented vertically, but two specimens were tested with plywood sheets oriented horizontally. The five test protocols were described as: 1) static one-directional, 2) static cyclic, 3) free vibration, 4) sine wave frequency sweep, and 5) earthquake. In one of the static one-directional tests, a panel with waferboard sheathing was tested with a vertical load distributed along its top edge to investigate whether dead load affected the racking performance of the wall. Vertical load was applied to five of the 27 walls tested during the earthquake loading series.

In all but the free vibration tests, the wall panels were tested on a shake table. The table was required to move in either a one-directional or cyclic mode while the top of the wall panel was restrained against horizontal movement. An inertial mass was used to represent the upper two stories of a three-story apartment building. Thus, the test walls were intended to represent shear walls on the ground floor of an apartment building experiencing an earthquake. All of the test walls were anchored at the lower corners to resist overturning. Base shear resistance was provided by intermediate bolts which were attached to the shake table.

Twenty-seven earthquake tests were conducted to verify the accuracy of the general finite element model. The base of the test wall was displaced according to the acceleration records of two earthquakes: 1952 Kern County, CA and February 9, 1971 San Fernando, CA. The loads experienced by the walls were directly proportional to the accelerations experienced by the inertial mass. Table 3.6 summarizes the full-scale shear wall test program.

Table 3.6 Summary of Full-Scale Shear Wall Tests (after Dolan, 1989).

Sheathing Type	Sheathing Orientation	Nail Spacing Perimeter/Field mm/mm	in/in	Dead Load Applied kN	(lb)	Earthquake Record Used	Number of Specimens
Static One-Directional Test							
Waferboard	Vertical	100/150	(4/6)	0	0	N/A	3
Waferboard	Vertical	100/150	(4/6)	44.5	10,000	N/A	1
Plywood	Vertical	100/150	(4/6)	0	0	N/A	3
Static Cyclic Test							
Waferboard	Vertical	100/150	(4/6)	0	0	N/A	2
Plywood	Vertical	100/150	(4/6)	0	0	NA	2
Sinewave Test							
Waferboard	Vertical	100/150	(4/6)	0	0	N/A	2
Plywood	Vertical	100/150	(4/6)	0	0	N/A	2
Earthquake							
Waferboard	Vertical	100/150	(4/6)	0	0	Kern Cnty.	4
		100/150	(4/6)	44.5	10,000	Kern Cnty.	3
		50/150	(2/6)	0	0	Kern Cnty.	2
		150/150	(6/6)	0	0	Kern Cnty.	2
		300/300	(12/12)	0	0	Kern Cnty.	1
Plywood	Vertical	100/150	(4/6)	0	0	Kern Cnty.	4
		100/150	(4/6)	44.5	10,000	Kern Cnty.	2
		50/150	(2/6)	0	0	Kern Cnty.	2
		150/150	(6/6)	0	0	Kern Cnty.	2
		300/300	(12/12)	0	0	Kern Cnty.	1
	Horizontal	100/150	(4/6)	0	0	Kern Cnty.	2
	Vertical	100/150	(4/6)	0	0	San Fernando	2

Figure 3.26 shows the load-displacement curves for the seven static one-directional tests which were conducted in accordance with ASTM Standard E564. The curves indicate that the average initial stiffness of the waferboard-sheathed walls was slightly higher than that of the plywood-sheathed walls. Also, the average ultimate load for plywood-sheathed walls was slightly higher as compared to the waferboard-sheathed walls. However, when the relative coefficients of variation were considered, it was concluded that there was no statistically significant difference between the ultimate load capacity of walls sheathed with plywood and that of waferboard-sheathed walls. Due to the similarity in shape of the load-displacement curves obtained from the static one-directional tests of sheathed connections and full-scale walls, Dolan applied a similar exponential curve to the full-scale wall data for both types of sheathing material. The form of the exponential equation is:

$$F = (P_o + K_2 \Delta)(1 - \exp(-K_o \Delta / P_o))$$

The parameters P_o, K_o, and K_2 are defined and determined in the same manner as those for the sheathed connection tests.

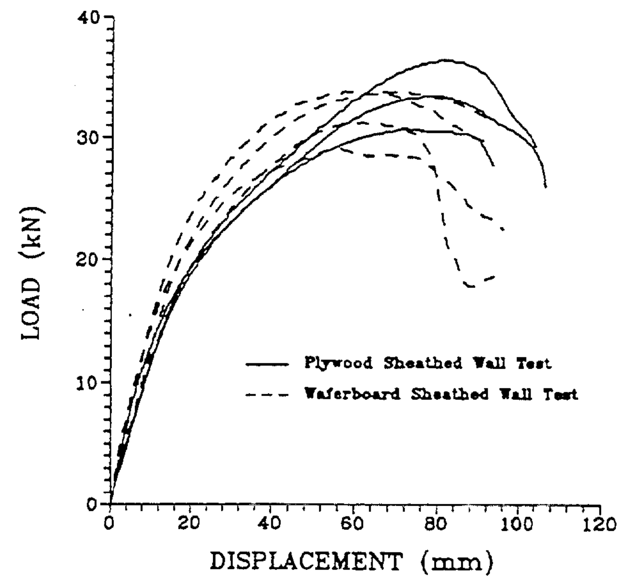
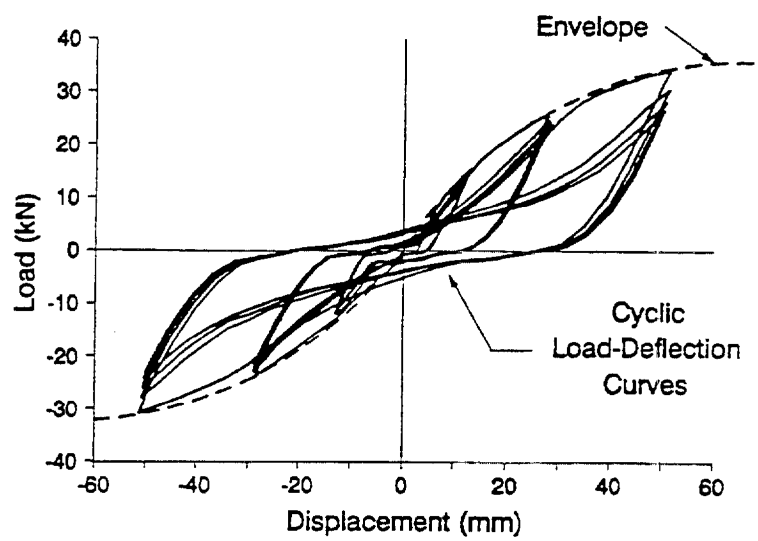


Figure 3.26 Load-Displacement Curves for Static One-Directional Tests of Walls (after Dolan, 1989).

The four static cyclic tests were performed to observe the hysteretic behavior of the plywood-sheathed walls. Four cycles of loading were applied at three displacement levels: 13 mm (0.5 in), 25 mm (1 in) and 51 mm (2 in). A typical set of load-displacement curves is shown in Figure 3.27. It was observed that in general the hysteretic curves could be enveloped by a curve similar in shape to the static one-directional curves. Data points were extracted from the first curve in each series to obtain parameters for the exponential equation mentioned above. The static cyclic parameters agreed very well with those obtained for the static one-directional tests.



Figurė 3.27 Typical Hysteresis Curves for Static Cyclic Tests of Walls (after Dolan, 1989).

Based on the earthquake test results, it was concluded that the holddown anchors and corner connections were sufficiently stiff and strong to promote racking deformations as opposed to rigid body rotation. Racking deformation is more desirable because it dissipates more energy. It was observed that neither the sheathing orientation nor the presence of vertical dead loading had any effect on lateral displacement, uplift, base slip or out-of-plane displacement. Moreover, the average peak horizontal displacements for walls sheathed with either plywood or waferboard were essentially the same. Some of the wall specimens with 300 mm (12 in) nail spacing, while exhibiting more ductility, failed during the 0.18 g Kern County earthquake. None of the walls with perimeter nail spacing of 150 mm (6 in) or greater survived the 0.3 g (actual San Fernando and scaled up Kern County) earthquakes. The peak accelerations of the load at the top of the wall and horizontal displacements at the top of the wall were essentially the same for Kern County and San Fernando earthquake wall tests.

Dolan's recommendation for additional research included: 1) a reliability study of timber shear walls to promote the design code use of probability-based design provisions for timber construction; 2) an experimental or analytical investigation of shear wall response to a wide range of earthquake excitations; and 3) experimental and analytical studies of the dynamic response of timber shear walls built with different methods of attaching the sheathing to the framing.

3.3.8 Racking Behavior of Wood-Framed Gypsum Panels Under Dynamic Load (Oliva, 1990)

This experimental program was conducted to address a gap in knowledge regarding the deformation characteristics of wood-frame wall assemblies sheathed with gypsum wallboard, particularly when they undergo cyclic lateral deformations. Although gypsum wallboard has been the most common sheathing material for interior walls and partitions in residential construction, its contribution to lateral load resistance had not been sufficiently quantified by laboratory tests.

The objectives of this study were to:

1. Define deformations which can be tolerated in gypsum sheathed walls without forming visual damage or causing the need for expensive repairs;
2. Provide a preliminary definition of the available initial stiffness in gypsum sheathed walls with cyclic lateral load;
3. Define the lateral load resisting capacity of gypsum sheathed walls under cyclic loading; and
4. Identify the failure process or damage expected of walls.

Fifteen walls were tested to failure. The wall panels were tested in a vertical position with bottom edges constrained to simulate the as-built conditions in a building. A uniformly distributed gravity load was applied at the top of each test specimen and lateral loads were applied at the top edge. All tests were conducted under displacement control. The experimental program included static monotonic tests (2 unglued and 2 glued walls), pseudo-dynamic cyclic tests (2 unglued and 2 glued walls), and dynamic tests (5 unglued and 2 glued walls). The primary difference between the pseudo-dynamic and dynamic tests was the frequency of the input displacement function. For the pseudo-dynamic tests, three reversed cycles of lateral top displacement were applied for amplitudes of 2.5 mm, 5.1 mm, 10.2 mm, 12.7 mm and 19.1 mm (0.1 in, 0.2 in, 0.4 in, 0.5 in, and 0.75 in). For the dynamic tests, a cyclical displacement history was applied at a rate of 5 cycles per second (refer to Figure 3.28 to see the complete pattern).

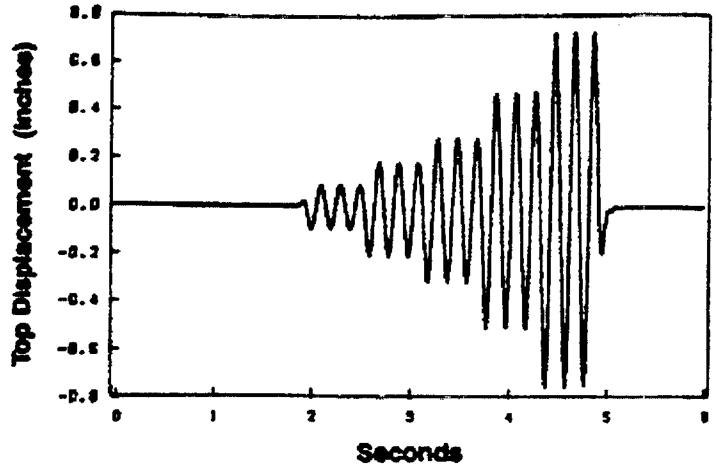


Figure 3.28 Dynamic Loading Pattern
(after Oliva,1990).

The wall panels were fabricated using 51 mm x 102 mm (2 in x 4 in) framing and 13 mm (1/2 in) gypsum wallboard sheathing. All of the wall specimens measured 2.4 m x 2.4 m (8 ft x 8 ft) and were sheathed on one side. Two methods were used to attach the gypsum wallboard to the frame members: 1) in the first group the gypsum wallboard sheets were attached horizontally with nails only, and 2) in the second group a bead of construction adhesive was applied along each nail line between the wallboard and the adjoining framing members. The frames had double top plates, single sole plates, single boundary studs and single intermediate studs, spaced 0.6 m (24 in) on center.

The load-displacement curves for the two unglued walls under monotonic loading were characterized by constantly varying stiffnesses, with no distinctly linear portion. Oliva, however, observed that the response of these panels can be reasonably idealized by a tri-linear load-displacement plot. Figure 3.29 shows the tri-linear load-displacement model and the stiffnesses of the three legs.

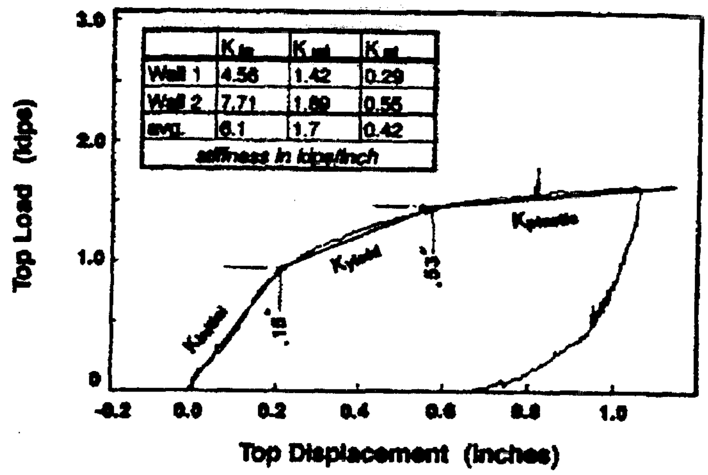


Figure 3.29 Tri-linear Load-Displacement Model of Unglued Walls
(after Oliva, 1990).

Hysteresis curves were plotted for each of the two identical unglued wall panels tested under cyclic loading. Figure 3.30 shows the hysteresis curves for one of the unglued walls. Both sets of curves were characterized by "a continuous gradual decline in wall lateral stiffness with increased displacement and with repeated cycling at any constant displacement." The nail damage initiated at the corners of the walls and propagated first along the bottom plate and then along the top plate. The overall damage pattern was generally symmetrical about two axes as the maximum applied displacements were reached. The maximum lateral load capacity, per unit of wall length, developed in the cyclic tests of unglued walls was 2.48 kN/m (170 lb/ft) compared to a maximum capacity of 3.58 kN/m (245 lb/ft) in the monotonic static tests.

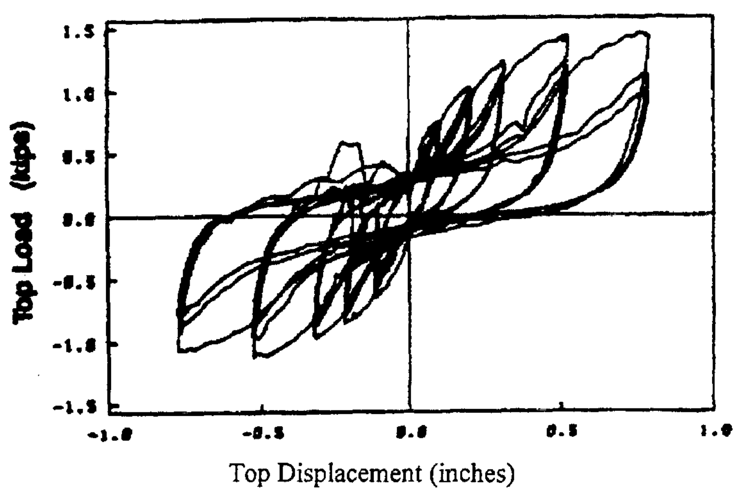


Figure 3.30 Hysteresis Curves for an Unglued Wall
(after Oliva, 1990).

Figure 3.31 shows the load-displacement curves obtained from the monotonic load tests of the two glued walls. The average ultimate load for the walls was 4.82 kN/m (330 lb/ft). Oliva suggested a tri-linear load displacement model with average values of linear stiffness assigned to each leg:

1. An initial linear region - effective stiffness of 2.5 kN/mm (14.2 kip/in),
2. A second region of changing stiffness - average stiffness of 0.3 kN/mm (1.7 kip/in),
3. A third "yielding" region - stiffness of 23 N/mm (0.13 kip/in).

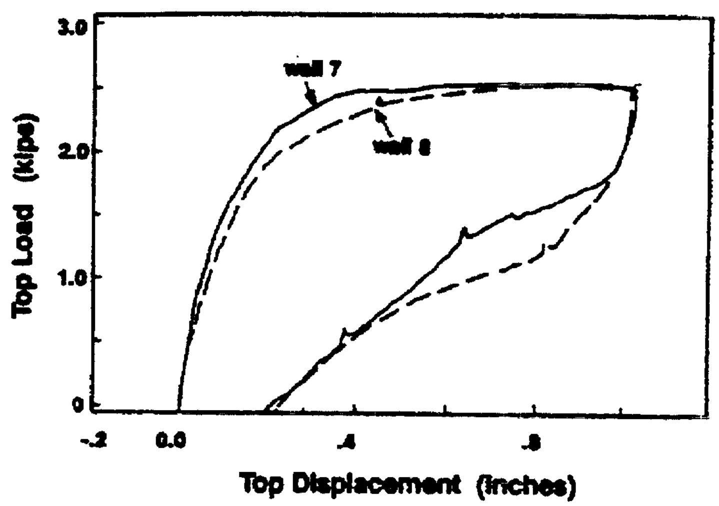


Figure 3.31 Load-Displacement Curves for Two Glued Walls
(after Oliva, 1990).

A comparison of the load-displacement curves for the unglued and glued walls is shown in Figure 3.32. Figure 3.33 shows the hysteresis curves from one of the glued walls subjected to cyclic loads. The two glued walls reached an average maximum load of 4.14 kN/m (284 lb/ft) at a displacement of 13 mm (0.5 in). The failure pattern was characterized by joint compound spalling over the nails in the bottom plate. The gypsum wallboard sheathing first developed horizontal cracks at the bottom plate and finally broke away from the framing over a distance of 460 mm (18 in) from the base of the wall. Unlike the unglued wall panels, the glued walls showed no visible nail damage along the top plate.

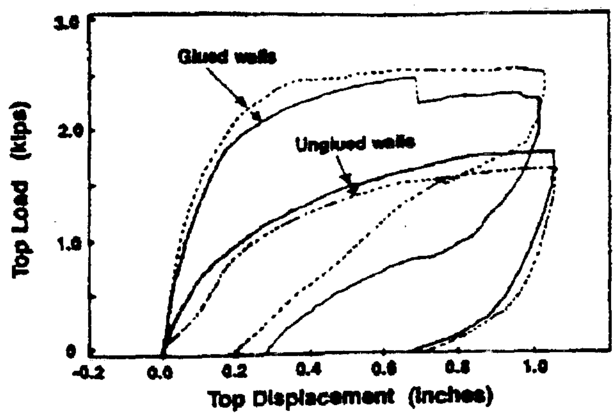


Figure 3.32 Comparison of Load-Displacement Curves for Unglued and Glued Walls (after Oliva, 1990).

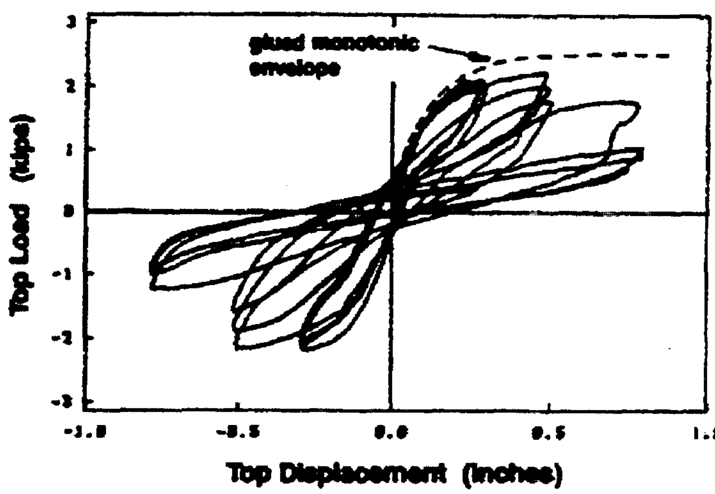


Figure 3.33 Hysteresis Curves for Glued Walls Subject to Cyclic Loads (after Oliva, 1990).

All five of the unglued walls subjected to dynamic testing exhibited failure patterns that were concentrated at the bottom plate-to-sheathing connection. Horizontal slip of the sheathing along the bottom plate increased with each new displacement amplitude. At the maximum top displacement of 19 mm (0.75 in), horizontal slip between the sheathing and the bottom plate peaked at 7 mm (0.27 in) or 36% of the top displacement.

During dynamic testing of glued walls, damage in the first wall occurred along the top plate and in the second wall damage was concentrated at the bottom plate. Due to the presence of the glue line between the sheathing and the frame, horizontal slip did not reach measurable levels until the top displacement reached 13 mm (0.5 in) or 0.5% interstory drift. At this point the bottom glue joint failed and horizontal slip was initiated.

As a result of the monotonic and cyclic load tests, it was concluded that gypsum wallboard-sheathed walls should be limited to a lateral displacement of about 0.2% of story height (h/500) if visible damage is to be prevented. The application of glue along nail lines can increase the interstory drift limit about 0.3% before damage is observed. When, as some building codes permit, the maximum allowable interstory drift is set at 0.5% of story height, severe damage can be expected in gypsum wallboard-sheathed walls.

Nailed gypsum wallboard-sheathed walls resisted lateral loads up to 1.46 kN/m (100 lb/ft) before non-linearity occurred. Adding a single bead of glue to the nail pattern increased the linear elastic capacity to 3.65 kN/m (250 lb/ft).

The use of glue in combination with nailing to attach the gypsum wallboard to the frame assembly increased the ultimate strength of gypsum wallboard-sheathed walls by up to 42% and initial stiffness by up to 130%. The primary disadvantage in using glue was that it created a more brittle type of behavior in the wall panel, which would not be desirable in regions with potentially strong earthquake motion.

3.3.9 Monotonic and Cyclic Tests on Timber Shear Walls (Dolan and Madsen, 1992a)

This study was a subset of an extensive investigation (Dolan, 1989) whose purpose was to define the dynamic response of timber shear walls subjected to earthquakes and to develop a general numerical model capable of predicting the dynamic behavior of shear walls. The primary objective of the tests was to obtain monotonic and cyclic racking behavior data for shear walls sheathed with both plywood and waferboard panels. The secondary objective was to compare the displacement and strength characteristics of plywood and waferboard sheathed shear walls.

Eleven full-size wall panels, 2.4 m x 2.4 m (8 ft x 8 ft), were tested. In-plane monotonic loading was applied to seven panels - four sheathed with 10 mm (3/8 in) waferboard and three sheathed with 10 mm (3/8 in) plywood. One of the panels with waferboard sheathing was tested with a vertical load distributed along its top edge to investigate whether the dead load affected the racking performance of the wall. Four panels were subjected to static, reversed cyclic horizontal load. Vertical load was not applied during any of the four cyclic loading tests.

In both the monotonic and cyclic loading series, the wall panels were anchored in a test frame that held the top of the wall rigidly against horizontal movement, but allowed vertical movement. The base of each wall panel was forced to move according to a predetermined displacement sequence. The test setup for the monotonic tests is as prescribed in ASTM E564.

In the monotonic load tests, the displacement rate was approximately 0.4 mm/s (0.9 in/min) and the test duration was 5 minutes. The displacement rate for the static cyclic shear wall tests was the same as that used in the monotonic tests, resulting in a test duration of about

45 min. A constant displacement rate of approximately 0.4 mm/s (0.9 in/min) was maintained for the cyclic wall tests.

Monotonic load-displacement curves for plywood and waferboard sheathing were compared for strength and stiffness characteristics. Also, the peak loads and corresponding displacements were tabulated for the two sheathing materials (see Table 3.7).

Table 3.7 Peak Load and Corresponding Displacement for Monotonic Shear Wall Tests (after Dolan and Madsen, 1992a).

Specimen	**Peak Load** kN (kips)	**Corresponding Displacement** mm (in)	**Dead Load Applied** kN (kips)
Waferboard			
WS-11	28.6 (6.4)	62 (2.4)	0
WS-12	33.8 (7.6)	77 (3.0)	44.8 (10.1)
WS-13	31.2 (7.0)	74 (2.9)	0
WS-14	33.8 (7.6)	81 (3.2)	0
Average	31.8 (7.1)	74 (2.9)	
Plywood			
PS-1	30.4 (6.8)	78 (3.1)	0
PS-2	33.5 (7.5)	86 (3.4)	0
PS-3	36.5 (8.2)	89 (3.5)	0
Average	33.5 (7.5)	84 (3.3)	

Figure 3.34 shows the typical deformation pattern of a shear wall. The deformation pattern develops in several stages: 1) initially as the wall deforms, the nails located at the corners of the sheathing panels deform more than the other nails, 2) with increasing deformation, the corner nails exhibit inelastic behavior, while the other nails deform elastically, and 3) as the racking deformations continue to increase, more nails along the edges deform inelastically. Dolan and Madsen observed close similarity between the hysteresis curves obtained from the cyclic shear wall tests and those obtained from cyclic connection tests performed in another of their studies (Dolan, 1989).

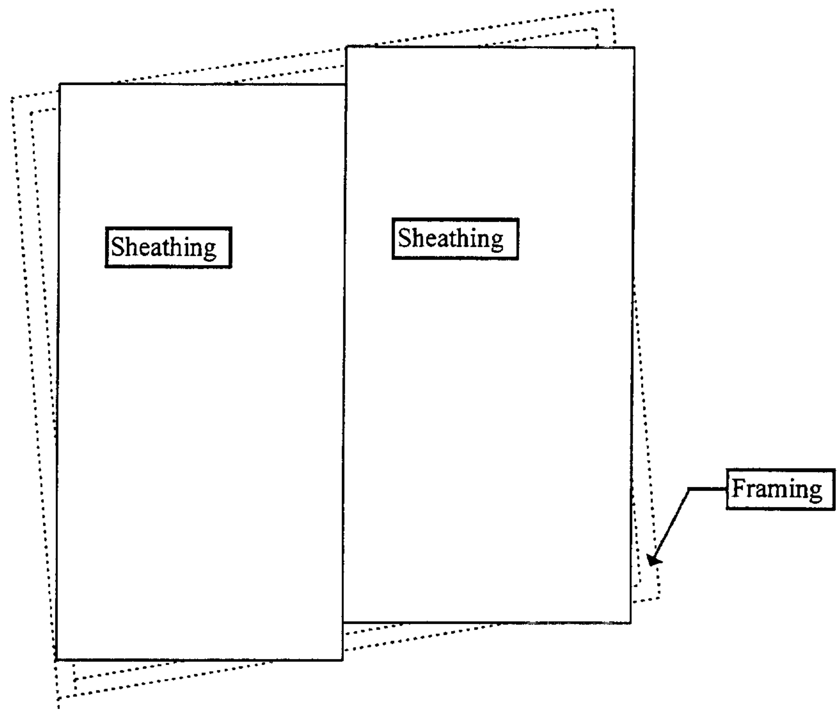


Figure 3.34 Deformation Pattern for Racking Displacement of Typical Shear Wall (after Dolan and Madsen, 1992a).

Following are some of the conclusions drawn from the results of this study:

1. There was no significant difference in the shear load capacities of the plywood and waferboard sheathed wall panels.

2. When the load was greater than 50% of the failure load, the plywood-sheathed shear walls were more ductile than waferboard-sheathed walls for a given load.

3. The out-of-plane displacements of the waferboard panels were significantly less than those for the plywood panels. However, in neither case did the out-of-plane displacement have a significant effect on the in-plane behavior of the shear wall panels.

4. The hysteresis curves for cyclic loading of the shear walls were contained in the envelope formed by the monotonic racking load-displacement curve.

5. The overturning anchor connections and corner connections at the end of the shear walls were necessary to force the wall panels to undergo racking deformations and to prevent the walls from deflecting as a rigid body.

3.3.10 Cyclic Testing of Narrow Plywood Shear Walls (Applied Technology Council, 1995)

The objective of the study was to quantify the strength and displacement characteristics of narrow plywood-sheathed shear wall panels under static and dynamic loading.

Door and window openings often disrupt the continuity of exterior walls, resulting in short segments of wall being required to effectively resist in-plane horizontal forces. The 1991 Uniform Building Code specified a maximum height-to-width ratio of 3.5 to 1 for a shear wall to be considered effective in resisting lateral forces. Five wall panels, each built with 3.5 to 1 height-to-width ratio, were tested to collect quantitative data about the strength and displacement characteristics of these minimally allowable shear walls.

The wall panels were constructed in accordance with specifications of the Applied Technology Council's (ATC's) advisory Project Engineering Panel. Two specimens were subjected to monotonic loading while the other three panels were tested dynamically, under a sinusoidally varying, deflection-controlled, cyclic excitation of 2.0 Hz. In the dynamic testing, a series of sixteen different displacement amplitudes were applied, each amplitude being a part of a three period sine wave. In both the monotonic and dynamic tests, progressively greater, in-plane deflection was imposed at the top of the wall panels, while the bottom was restrained, until the panel could no longer provide resistance to the maximum applied force (i.e. until the panels failed).

Each of the five panels measured 0.9 m (27.5 in) wide and 2.4 m (96 in) high. A single sheet of 10 mm (3/8 in) thick plywood was nailed to one side of the timber frame. The frame consisted of double 51 mm x 102 mm (2 in x 4 in) end studs, a single 51 mm x 102 mm (2 in x 4 in) stud at the middle, a double 51 mm x 102 mm top (sill) plate and a single 51 mm x 102 mm bottom (sole) plate. Bolted hold-down devices were attached to the boundary studs. The results of the two static and three dynamic tests are summarized in Tables 3.8 and 3.9.

Table 3.8 Static Test Results (after ATC, 1995).

	Test 1		**Test 2**	
Max. Holddown Load	34.4 kN	7725 lb	33 kN	7421 lb
Displ. At Top of Wall at Panel Failure	222.2 mm	8.75 in	203.2 mm	8.00 in
Max Slip Relative to Concrete Base	1.3 mm	0.05 in	1 mm	0.04 in
Uplift Displacement at Panel Failure	55.1 mm	2.17 in	50.8 mm	2.00 in
Max. Diagonal Displ. Of the Plywood Panel	2.8 mm	0.11 in	3.3 mm	2249 lb
Applied Force at Which Sill Plate Failed	10 kN	2243 lb	10 kN	2249 lb
Ultimate Load	10 kN	2243 lb	10 kN	2249 lb

Table 3.9 Dynamic Test Results (after ATC, 1995).

	Test 1		Test 2		Test 3	
Max. Displacement At Top of Panel	154.9 mm	6.10 in	143.2 mm	5.64 in	148.6 mm	5.85 in
Maximum Slip Relative to Concrete Base	2 mm	0.08 in	0.8 mm	0.03 in	1 mm	0.04 in
Maximum Uplift Force	31.5 kN	7080 lb	30.2 kN	6795 lb	31.8 kN	7151 lb
Max. Uplift Displacement	33.3 mm	1.31 in	29.5 mm	1.16 in	28.2 mm	1.11 in
Max. Diagonal Displ. of Plywood Panel	2.3 mm	0.09 in	1.8 mm	0.07 in	2.3 mm	0.09 in
Cycle at Which Loss of Holddown Tension Occurred	4	4	5	5	4	4
Ultimate Load	8.6 kN	1925 lb	8.4 kN	1890 lb	8.2 kN	1855 lb

Based on the results of both the static and dynamic tests, it was concluded that the narrow plywood-sheathed wall segments can undergo substantial inelastic deformation. As a result of this inherent flexibility, there is likely to be a loss of initial load in the anchor bolts of the holddown device in the case of dynamic loading, and in large horizontal movement of the top edge of the panel for both static and dynamic loading. The total horizontal movement, 41 mm (1.6 in), at the design load was over three times the maximum of 12 mm (0.48 in) specified in the Uniform Building Code.

The average ultimate load achieved during the tests was approximately 2.5 times the rated horizontal load capacity of the test panels. It was recommended that several alternative engineering approaches to improving the in-plane shear performance of narrow plywood-sheathed wall panels be compared using a standardized evaluation protocol. To facilitate uniform interpretations of comparative test results, the authors acknowledged that it is first necessary to develop a standardized test procedure for wood-frame shear walls.

3.3.11 Lateral Resistance of Wood Shear Walls with Large Sheathing Panels (Lam, Prion, and He, 1997)

The primary objective of this study was to investigate and quantify the stiffness, strength, and energy dissipation capacity of wood-frame shear wall systems built with nonstandard sizes of oriented strand board (OSB) sheathing panels. The results of this study will be used to develop analytical and design procedures for the seismic performance of shear walls built with nonstandard large dimension panels.

The weak link in wood-frame shear wall construction with board product sheathing is the joint along the panel edges in spite of the decreased nail spacing along the edges. Thus, improved shear wall performance was sought by using continuous panels for sheathing. It was reported that in Canada some OSB mills can produce panels up to 3.3 m x 7.3 m (11 ft x 24 ft) in size. Nonstandard large dimension panels are more likely to be used in commercial construction than in residential construction.

This study constituted the first phase of a research program on the lateral resistance of wood-frame shear walls, with and without openings, built with regular and nonstandard large dimension OSB panels under monotonic and cyclic loading conditions. The results of the second phase of the study, involving shear walls with openings, is to be reported later by the investigators.

Eleven shear walls were tested under combined vertical and lateral loads. Monotonic and cyclic loading tests were performed on walls incorporating: regular panels with seams and continuous panels, two types of nails, three nail spacings, and a range of the number of nails. There was no replication of testing for a given combination of these variables.

Wall specimens were built to finished dimensions of 2.4 m (8 ft) high by 7.3 m (24 ft) long. The wall framing was typical of many other shear wall tests: 51 mm x 102 mm (2 in x 4 in) studs spaced 406 mm (16 in) on center, double top plates, double end studs, and a single bottom plate. The OSB panels were attached to one side of the frame with the long axis of the panels parallel to the length of the wall. Steel I sections were used for load distribution and as base beams. Horizontal load was applied along the top plate. Three pairs of hydraulic jacks were mounted to the test floor on either side of the wall and attached via steel rods to the top distribution beam to deliver static uniformly distributed vertical load to the wall.

Loading was applied as a series of sinusoidal cyclic groups, with each group comprised of three identical cycles. The amplitude of each cycle group was set as a percentage of the nominal yield displacement. The yield displacement was obtained by locating the point on monotonic load-displacement curve at which the load equaled one-half of the maximum load. The displacement amplitude schedule of the various cycle groups is shown in Table 3.10 for two of the test walls.

Table 3.10 Displacement Amplitude Schedule as a Percentage of Yield Displacement (after Lam, Prion and He, 1997).

Cycle Group	Wall Number 3	Wall Number 4
1	25%	25%
2	50	50
3	25	25
4	100	100
5	50	50
6	150	150
7	100	100
8	200	200
9	150	150
10	250	250
11	200	200
12	300	300
13	250	250
14	350	350
15	300	300
16	400	400
17	350	350
18	450	450
19	400	500
20	500	600
21	450	700
22	600	800
23	750	--

For the monotonic load tests, load-displacement curves were superimposed to illustrate the effects of: 1) continuous versus regular panel configuration, 2) spiral nails versus common nails, and 3) 76 mm (3 in) versus 152 mm (6 in) nail spacing. Figure 3.35 shows a typical comparative plot, in which the results from three oversized panels with different nailing schedules are compared. Table 3.11 summarizes some of the results obtained from the eleven shear wall tests.

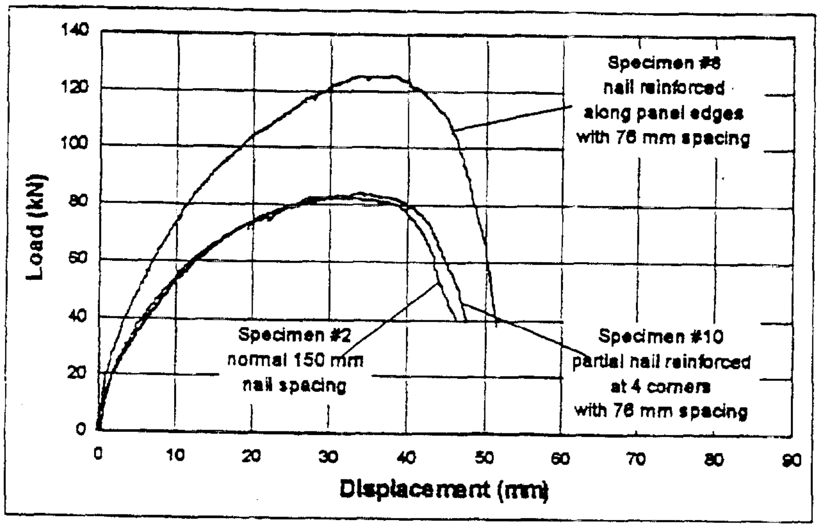


Figure 3.35 Load-Displacement Curves for Three Shear Walls with Oversize Panels and Spiral Nails (after Lam, Prion, and He,1997).

Table 3.11 Partial Summary of Shear Wall Test Results (after Lam, Prion, and He, 1997).

Wall No - Size	P_{max} kN	δ_{max} mm	Ultimate Shear Strength, S_u[a] N/m	Shear Stiffness, G[a] N/m	Ductility Factor, D[a]	Total Number of Nail Connectors for the Wall
1 - R	62.77	82	8.581	1.07	5.5	424
2 - OS	82.21	46	11.239	2.33	5.7	281
3 - OS	+64.07, -59.40	43	-	-	-	281
4 - R	+59.29, -51.69	79	-	-	-	424
5 - OS	71.13	41	9.724	3.35	6.2	281
6 - OS	+61.25, -56.69	36	-	-	-	281
7 - R	54.73	84	7.482	0.88	4.9	375
8 - OS	125.21	51	17.117	2.81	5.1	409
9 - OS	+101.54, -99.80	60	-	-	-	409
10 - OS	84.27	48	11.520	2.16	5.2	325
11 - OS	+71.35, -66.24	46	-	-	-	325

R = Regular size panel; OS = Oversized panel
[a]Values for S_u, G, and D were calculated per ASTM 564 for monotonic tests only.

The monotonic test results showed that for the same nail spacing, walls with larger sheathing panels had greater strength, higher stiffness, higher ductility, and smaller deformations compared to walls sheathed with regular sized [1.2 m (4 ft) x 2.4 m (8 ft)] panels. Based on two comparable test results, it was concluded that using common nails as fasteners will result in the wall being stiffer but weaker than if spiral nail fasteners were used.

The cyclic test results showed that walls with oversized panels were about 5% stronger than comparable walls with regular sized panels. Ductility ratios (displacement at maximum load/displacement at half the maximum load) did not vary significantly for the two panel dimensions. Walls built with regular sized panels dissipated more energy than those built with oversized panels.

The failure mode for walls tested under monotonic load was triggered by nail withdrawal from blocking and framing. In contrast, the predominant failure mode for walls tested under cyclic loading was caused by nail fatigue fracture. Although the failure modes differed, the location of the failures was the same for walls of similar construction.

Given the significantly higher initial stiffness of the walls with oversized panels, the investigators suggested that if such construction was to be used in seismic risk areas, there is the potential for reduced nonstructural damage to brittle elements such as brick veneers.

Further investigation of the cyclic load sequence used in this study was recommended to characterize the failure modes in wood frame shear walls.

3.3.12 Cyclic Tests of Long Shear Walls with Openings (Dolan and Johnson, 1997a)

This study had two objectives: 1) to determine the effects of openings on full-sized wood-frame shear walls under cyclic loads, and 2) to determine whether the perforated shear wall method conservatively predicts shear capacity.

The perforated shear wall design method is based on an empirical equation developed by Sugiyama (1993). The equation, which relates the strength of shear wall segments with openings to the strength of walls without openings, forms the basis for adjustment factors included in the SBCCI Standard Building Code (1996) and the High Wind Edition of the Wood Frame Construction Manual for One- and Two- Family Dwellings (1995). The tabulated adjustment factors are used to reduce the strength of traditional fully-sheathed shear wall segments due to the presence of openings.

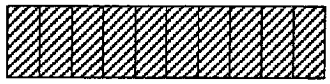
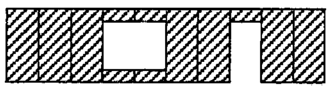
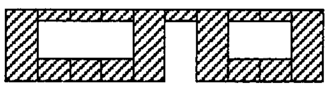
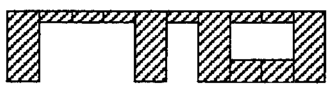
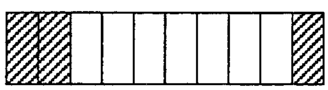
The ratio of the shear strength of a wall segment with openings to the strength of a fully sheathed wall segment without openings is called the shear capacity ratio, F. The ratio is determined from the following equation:

$$F = r / (3 - 2 \cdot r)$$

where, 'r' is the sheathing area ratio (≤ 1.0), see Table 3.12.

All five shear walls tested used the same type of framing, sheathing, nails, and nail patterns. Each wall had a different configuration of wall and window openings. The size and placement of openings were selected to cover the range of sheathing area ratios applicable to light wood-frame construction. The walls were 12.2 m (40 ft) long and 2.4 m (8 ft) tall and the framing consisted of 51 mm x 102 mm (2 in x 4 in) studs spaced 406 mm (16 in) on center, double 51 mm x 102 mm (2 in x 4 in) top plates, single bottom plates, double end studs, and either double or triple studs adjacent to openings. Exterior sheathing included 1.2 m x 2.4 m (4 ft x 8 ft) sheets of 12 mm (15/32 in) plywood oriented vertically. Interior sheathing consisted of 13 mm (1/2 in) gypsum wallboard oriented vertically. For each test wall, two tie-down anchors were used, one at each double end stud. Table 3.12 shows the five wall configurations and their respective sheathing area ratios (r).

Table 3.12 Wall Configurations and Sheathing Area Ratios
(after Dolan and Johnson, 1997a).

Wall Configuration	Wall Type	Sheathing Area Ratio, (r)
	A	1.0
	B	0.76
	C	0.55
	D	0.48
	E	0.30

The cyclic load pattern used was a modification of the Sequential Phased Displacement (SPD) procedure used by the Joint Technical Coordinating Committee on Masonry Research for the United States-Japan Coordinated Earthquake Research Program. The displacement was a triangular, ramp function with a frequency of 0.5 Hz (refer to Figure 3.36).

The cyclic loading consisted of two displacement patterns. The first displacement pattern was characterized by three reversed cycles each at 25, 50 and 75% of the anticipated yield displacement (referred to as the First Major Event or FME). Dolan and Johnson determined an artificial elastic-plastic curve for each wall. The curve was defined such that the area under the curve was equal to the area under the load-displacement curve from 0 mm to displacement at failure. The displacement at yield was defined as the intersection of the artificial elastic and plastic lines. In the final cycle of the first phase, the wall was displaced to approximately the FME. At this point the walls responded inelastically (the onset of nonlinear behavior) and the second displacement pattern began. A typical sequence for the second displacement pattern was: 1) one reversed cycle at the designated peak displacement, 2) three decay cycles with each 25% less than the previous one (i.e. the first decay cycle was 75% of the peak, second was 50%, and third was 25%), and 3) three cycles at the designated peak displacement. Figure 3.36 shows the entire displacement pattern used in SPD. Figure 3.37 shows a single phase of the SPD pattern after reaching FME.

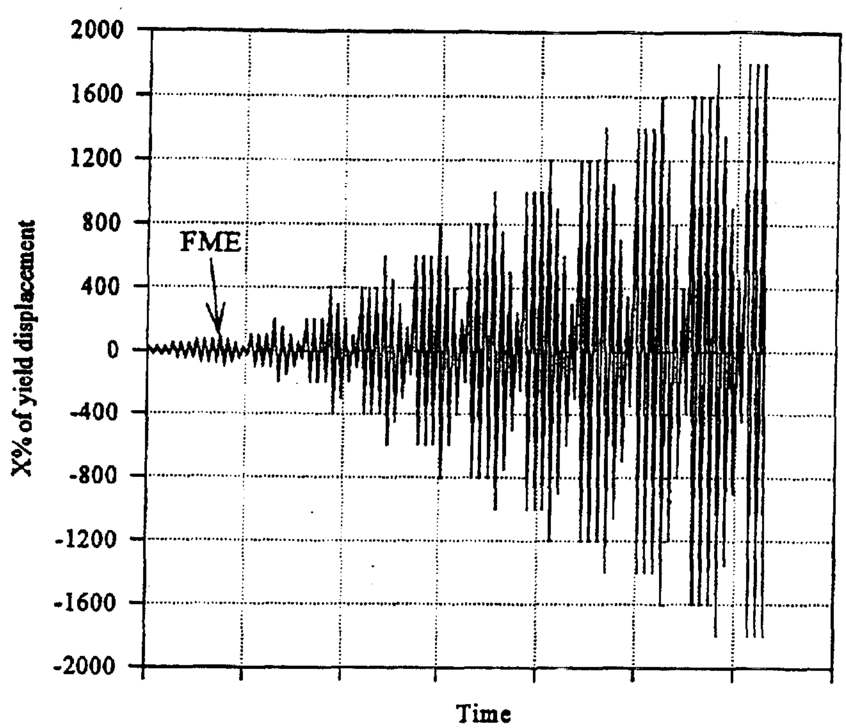


Figure 3.36 Displacement Pattern Used in Sequential Phased Displacement (after Dolan and Johnson, 1997a).

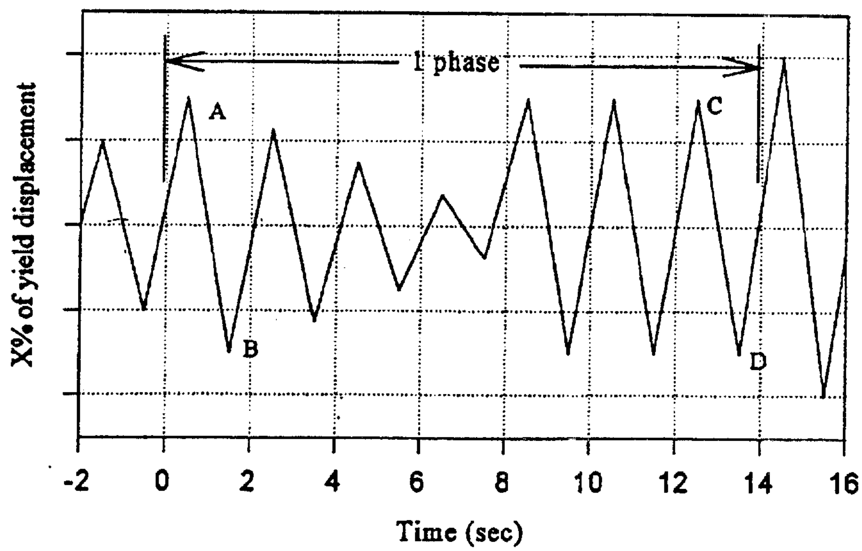


Figure 3.37 Single Phase of Sequential Phased Displacement Pattern (after Dolan and Johnson, 1997a).

The results are presented in the form of several load-deflection curves: 1) hysteresis loops for the entire loading history experienced by each wall during the second phase of loading (i.e., post yielding), 2) hysteresis loops from the initial cycle for each phase of the SPD loading until failure was reached, 3) hysteresis loops from the stabilized cycle for each phase of the SPD loading until failure, and 4) initial and stabilized load envelope curves derived from the hysteresis loops described in 2) and 3).

As a final analysis, the accuracy of the shear capacity ratio equation was checked against actual test data. Table 3.13 shows the comparisons of the predicted and actual shear capacity ratio, F, for both the initial and stabilized cycles of the SPD.

Table 3.13 Application of Perforated Shear Wall Method to Cyclic Tests (after Dolan an Johnson, 1997).

	Wall Configuration				
	A	B	C	D	E
Predicted Shear Capacity Ratio (F)	1.0	0.51	0.29	0.24	0.13
Initial Cycle					
Capacity (kN)	142.4	90.3	60.5	51.2	33.4
Actual Shear Capacity Ratio (F)	1.0	0.63	0.43	0.36	0.23
(F) Actual / (F) Predicted	1.0	1.24	1.48	1.50	1.77
Stabilized Cycle					
Capacity (kN)	122.4	77.4	52.5	44.1	29.4
Actual Shear Capacity Ratio (F)	1.0	0.63	0.43	0.36	0.24
(F) Actual / (F) Predicted	1.0	1.24	1.48	1.50	1.85

Based on the comparison of the predicted shear strength ratios to the actual ratios, it was concluded that the perforated shear wall method for wood shear walls provides conservative estimates of the cyclic capacity when the cyclic capacity of fully sheathed walls is used as the basis. The degree of conservatism increased approximately linearly as the amount of openings increased.

3.3.13 Monotonic Tests of Long Shear Walls with Openings (Dolan and Johnson, 1997b)

This study was the companion to the study by Dolan and Johnson (1997a) in which cyclic loads were applied to five long shear walls with different configurations (refer to Section 3.3.12). Each wall in this study was the duplicate of one of the walls in the cyclic test study.

The two objectives of this study were: 1) to determine the effects of openings on full-sized wood-frame shear walls under monotonic loads, and 2) to determine whether the perforated shear wall method conservatively predicts shear capacity.

The five walls were constructed identically to those described in Section 3.3.12. Table 3.12 shows the five wall configurations and their respective sheathing area ratios (r). As shown in Section 3.3.12, r is used in the equation to predict the shear capacity ratio, F.

The walls were loaded monotonically to failure. No dead load was applied in any of the tests. Load-displacement curves for the five walls are shown in Figure 3.38. Maximum shear strength (shear capacity) values ranged from 36.5 kN (8.2 kips), Wall E, to 172.7 kN (38.8 kips), Wall A. In Figure 3.39 the curve for predicted shear capacity is compared to the actual capacities for the five test walls. The curve was derived by first computing the shear capacity ratio, F, for the range of sheathing area ratios from 0 to 1.0. Then, the values of F were multiplied by the known capacity of a fully-sheathed wall (i.e. Wall A) to obtain the predicted capacities. For example, using an r of 0.3 (corresponding to Wall E), the resulting value of F is 0.125. When F is multiplied by the known capacity of Wall A, [172.7 kN (38.8 kips)], the predicted capacity of Wall E is 21.6 kN (4.85 kips). As shown in Figure 3.39, the shear capacity ratio equation conservatively estimates monotonic load capacity for the four perforated wall configurations included in this study.

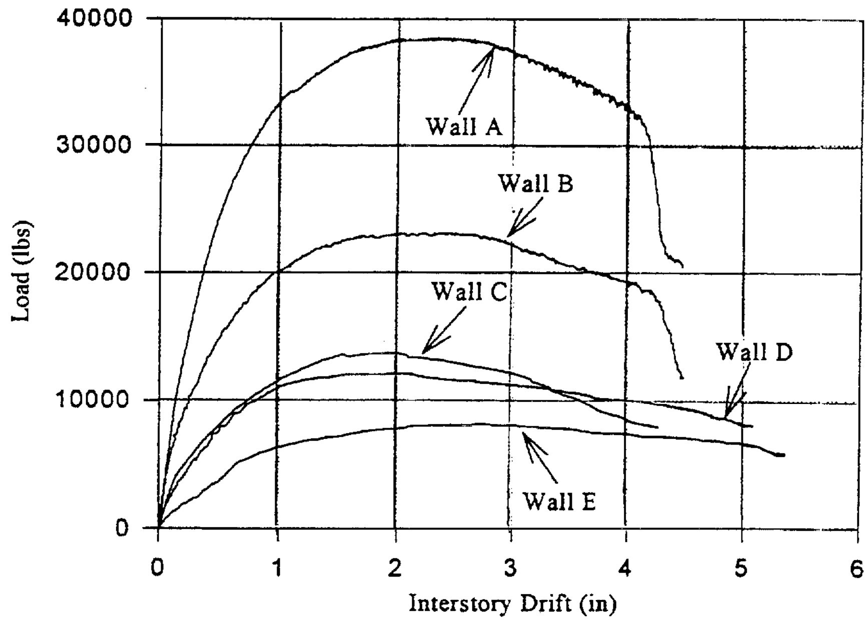


Figure 3.38 Load-Displacement Curves for Five Walls
(after Dolan and Johnson, 1997b).

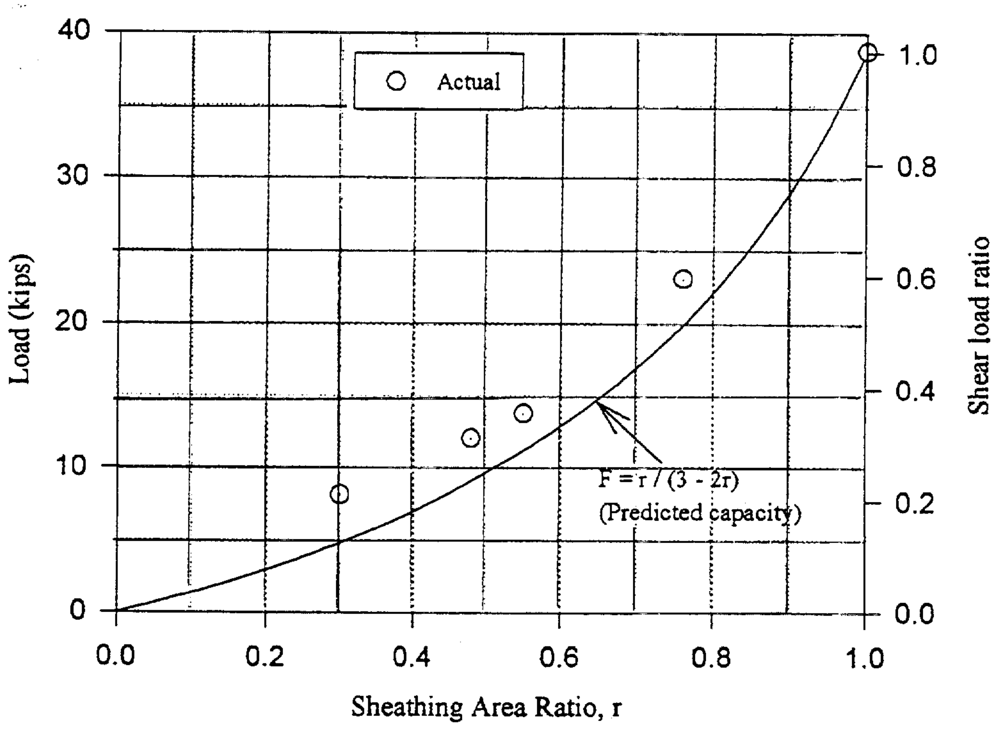


Figure 3.39 Actual versus Predicted Shear Capacity
(after Dolan and Johnson, 1997b).

Dolan and Johnson reported that the failure modes for walls with and without openings were quite similar. "Typical failures at large displacements (significantly larger displacements than at capacity) for all walls often included buckling of the plywood panel adjacent to the load cell, and nail head pull-through at the bottom edge of the plywood panel, and nail tear through or head pull through of the gypsum wallboard panel edges. Walls with and without openings also had similar performance characteristics including relatively elastic performance until an interstory drift of approximately 0.5% [13 mm (0.5 in)] and the ability to support relatively high loads at displacements well beyond maximum capacity. Reduced strength and stiffness performance of shear walls with openings based on an equivalent length of full-height sheathing can be attributed to lack of uplift restraint provided adjacent to openings."

3.4 INTERCOMPONENT CONNECTIONS

3.4.1 Background

Low-rise, wood-frame construction consists of subassemblies - walls, floors, roof, and foundation - which are mechanically connected to create a three-dimensional structure. The subassemblies are comprised of wood (studs, joists, rafters) or man-made (e.g. gypsum wallboard) elements that are joined by fasteners such as nails, screws, staples, bolts or glue. As documented in Chapter 2, the performance of the structure as a whole under earthquake and wind loads, is primarily governed by the behavior of the connections between subassemblies (intercomponent connections). Moreover, the structural integrity of the subassemblies, when subjected to earthquake and wind-gust loads, is governed by the hysteretic behavior of the fasteners connecting the structural elements (intracomponent connections). The generally good performance of wood-framed houses in earthquakes and hurricanes (refer to Sections 2.3 and 2.4) can largely be attributed to the ductility and good energy dissipation characteristics of both intercomponent and intracomponent connections. The development of three-dimensional finite element models and the establishment of methodologies for predicting the performance of wood-frame houses require performance data on both types of connections.

Dating back to the 1960s mechanical fasteners in wood have been tested primarily using ASTM Standard E1761 (Standard Methods of Testing Mechanical Fasteners in Wood). This standard was adopted in 1960 to assist in quantifying the tensile and shear resistance of fasteners under static monotonic loads. There have been many studies conducted to quantify the static strength and stiffness of mechanical fastener joints that comprise intracomponent connections in houses. To date there has been no standard test methodology adopted for evaluating the behavior of fastener joints under static cyclic or dynamic loads. Nonetheless, over the past twenty years several studies have been conducted involving cyclic load testing of nailed joints in wood. The following is a partial listing of references addressing the subject of intracomponent connection behavior: Mack (1966); Foschi (1974); McLain (1975); Debonis and Bodig (1975); Wilkinson (1976); Jenkins, Polensek, and Bastendorff (1979); Foschi (1982); Polensek and Schimel (1986); Polensek and Bastendorff (1987); Chou and Polensek (1987); Dolan and Madsen (1992b); and Gutshall (1994). Details of most of these studies are summarized by Foliente (1994).

Tests of connections between subassemblies, like tests on full-scale houses, are complex and expensive to conduct. This probably explains why there have been so few experimental studies conducted on these types of connections. Following are summaries of three studies which were partially devoted to evaluating the behavior of intercomponent connections.

3.4.2 Seismic Behavior of Bending Components and Intercomponent Connections of Light Frame Wood Buildings (Polensek and Laursen, 1984)

Tests were conducted on a typical connection between stud walls, joist floors and concrete strip foundations. The primary purpose of this phase of the test program was the verification of a two-dimensional linkage element used in a finite element model. Four specimens were tested, but Polensek and Laursen (1984) reported that meaningful data were obtained from only two of the tests due to problems with the data acquisition. Details of the connection specimens are described in Figure 3.40. To simulate the effect of gravity load transferred by the walls to the foundation, concrete blocks were secured on top of the wall sole plates.

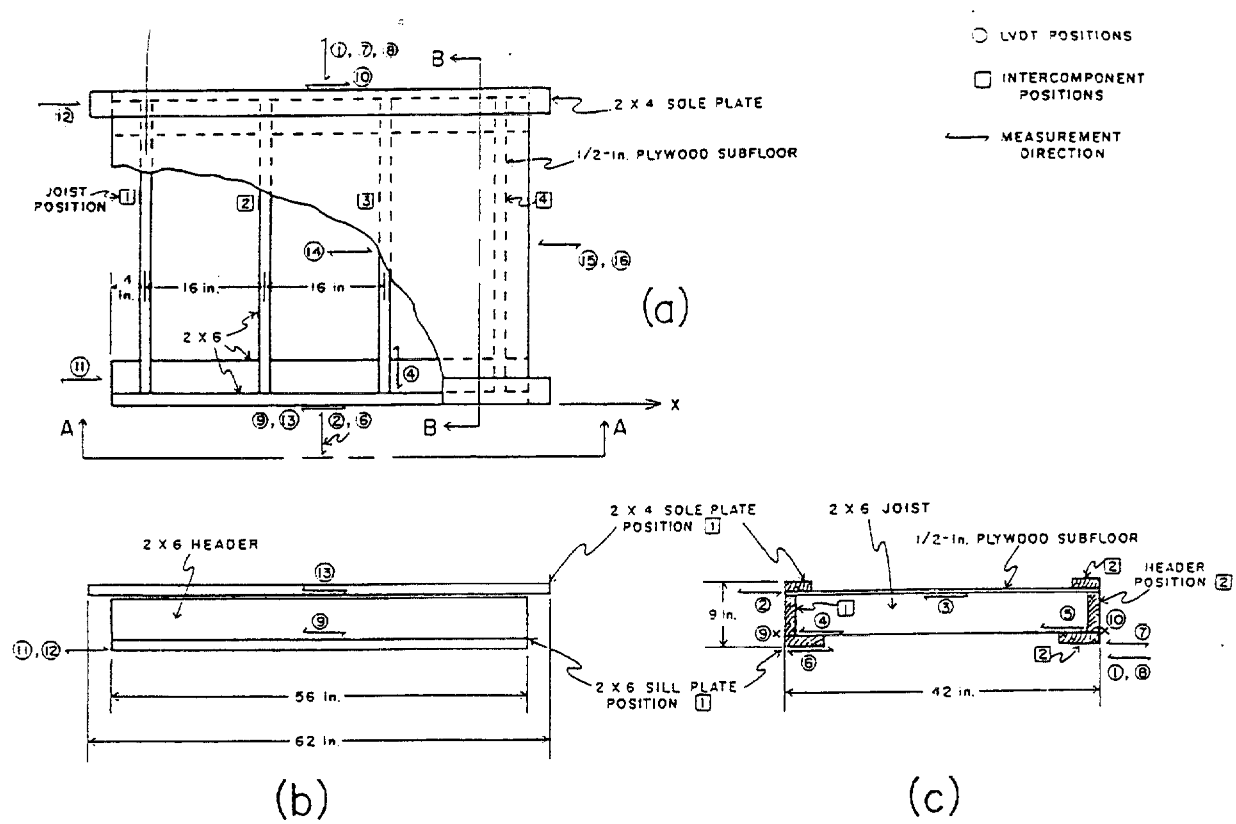


Figure 3.40 Typical Intercomponent Connection Specimen (after Polensek and Laursen, 1984).

The specimens were tested on a movable frame that simulated two parallel wall foundations. The test frame/load actuator assembly was capable of producing linear displacement either parallel or perpendicular to the shear wall sole plate. Figure 3.41 illustrates one of the two orientations of the test frame/load actuator assembly. Three types of loading function were applied to each connection specimen for each direction of load application: 1) free vibration, 2) forced sinusoidal vibration, and 3) a seismic displacement spectrum obtained from the El Centro Imperial Valley earthquake of 1940. The objectives of the free and sinusoidal vibration tests were to evaluate the damping ratio and the natural frequency. The sinusoidal vibration tests were conducted at 1 Hz increments for frequencies ranging from 6 to 12 Hz.

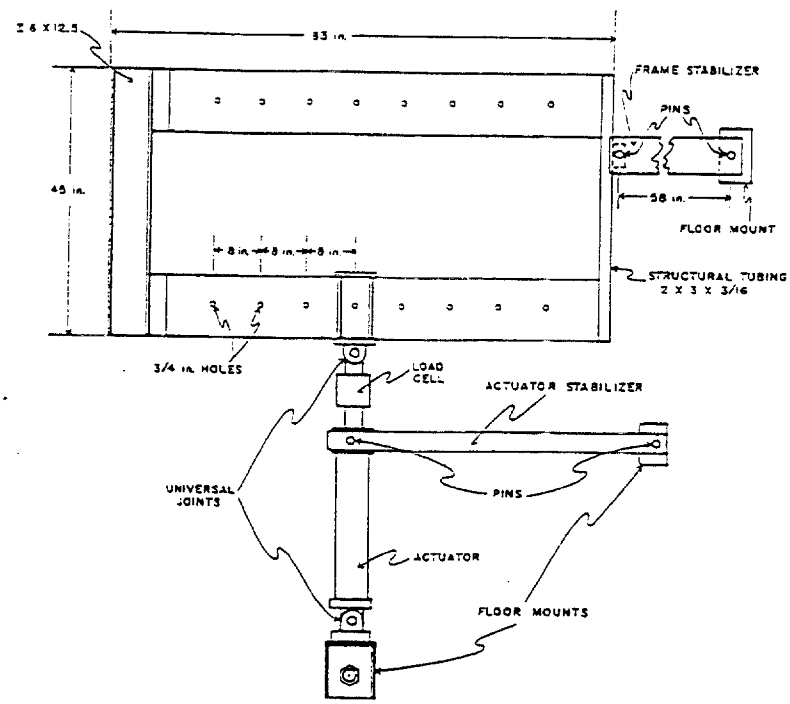


Figure 3.41 Test Frame/Actuator Assembly for Displacement Perpendicular to Shear Wall (after Polensek and Laursen, 1984).

The test results were presented in the form of a series of graphs of frequency versus slip response that was measured at five locations on the connection specimen. A set of graphs for one of the specimens is shown in Figure 3.42. The graphs were used to obtain the natural frequency and damping ratios. The natural frequency was found to be about 7 Hz. The following equation was used to calculate the damping ratio, λ:

$$\lambda = \{[1/(r_2^2 - r_1^2)^2 + \tfrac{1}{2}]^{1/2} - 1/(r_2^2 - r_1^2)\}/2$$

where, $r_1 = F_1/F_n$ and $r_2 = F_2/F_n$, with F_1, F_2, and F_n being identified in Figure 3.42. Applying the damping equation to the graphs in Figure 3.42 resulted in damping ratios between 9% and 11% for the five LVDT locations shown. No results were presented from the El Centro Imperial Valley earthquake tests.

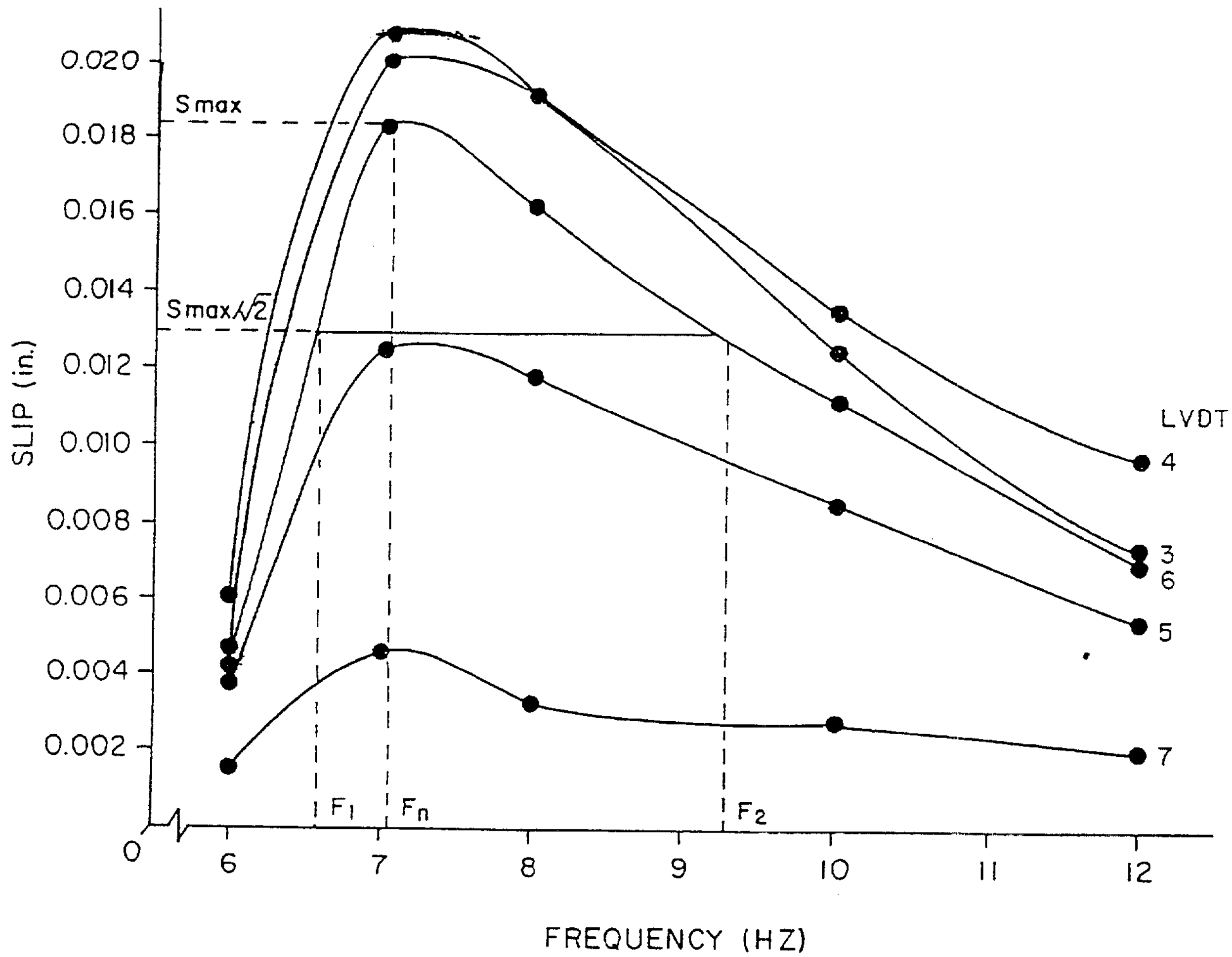
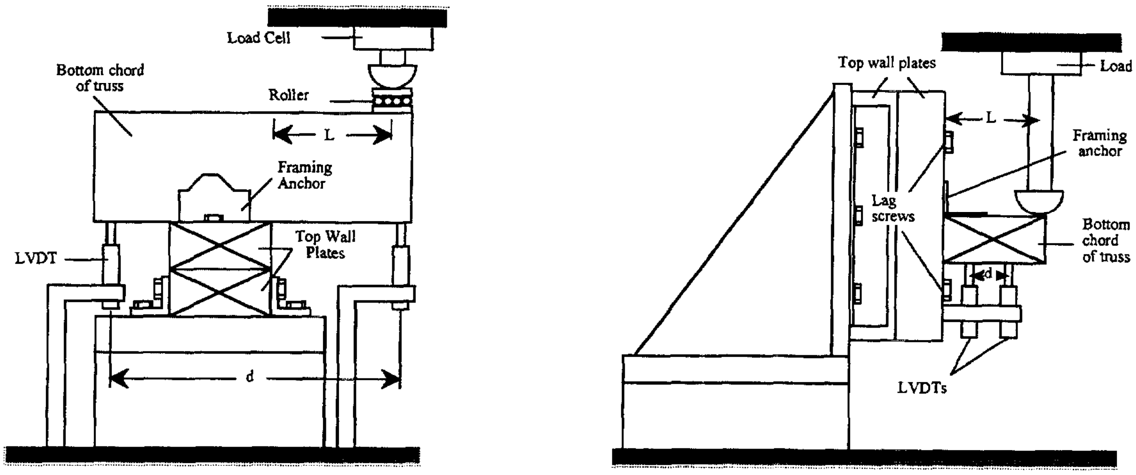


Figure 3.42 Response of a Connection Specimen to Sinusoidal Loading (after Polensek and Laursen, 1984).

3.4.3 Finite-Element Model of a Nonlinear Intercomponent Connection in Light-Frame Wood Structures (Groom and Leichti, 1991)

As a part of their analytical study of the connection between an interior wall and a roof truss, Groom and Leichti (1991) conducted experimental tests on a typical connection. The connection details are described in Figure 4.12a. There were two test setups, one to cause rotation about the longitudinal axis of the top plates of the wall, and one to cause rotation about the bottom cord of the truss. Fifteen connection samples were tested for each rotational mode. The experimental setups are described in Figure 3.43.



(a) rotation about long axis of wall top plate

(b) rotation about long axis of bottom chord of truss

Figure 3.43 Experimental Setup for Connection between Roof and Wall (after Groom and Leichti, 1991).

The purpose of the experimental study was to verify the accuracy of the finite element model that is discussed in Section 4.4 and illustrated in Figure 4.12b. Joint rotation was measured and moment-rotation relationships were plotted for each test. Regression analyses were performed on each set of 15 curves and the resulting moment-rotation curves formed a basis of comparison for the corresponding analytical curves. The comparative plots are presented in Figure 4.12c.

3.4.4 Transforming a Corner of a Light-Frame Wood Structure to a Set of Nonlinear Springs (Groom and Leichti, 1994)

The primary objective of this analytical study was to examine a mathematical technique for reducing the number of degrees of freedom from a detailed finite element model of an intercomponent connection to a set of energetically equivalent nonlinear springs. This study was a subset of a larger study by Kasal (1992), the purpose of which was to develop a three-dimensional finite element model of the single-story wood-frame house tested by Phillips (1990). The details of the analytical part of this study and of the study by Kasal (1992) are presented in Chapter 4.

An exterior wall-to-exterior wall connection representative of the corner connections for the four exterior walls of the Phillips' house (see Figure 3.9 for a plan view of the house) was first tested and subsequently modeled The actual connection consisted of three Douglas-fir studs, T1-11 plywood exterior sheathing, and gypsum board interior sheathing. Figure 3.44 shows the connection and the next adjacent stud. The studs and sheathing panels were 610 mm (24 in) long. The 13 mm (1/2 in) thick gypsum wallboard and the 13 mm (1/2 in) thick plywood sheets were nailed to the studs with their long edges running parallel to the length of the studs.

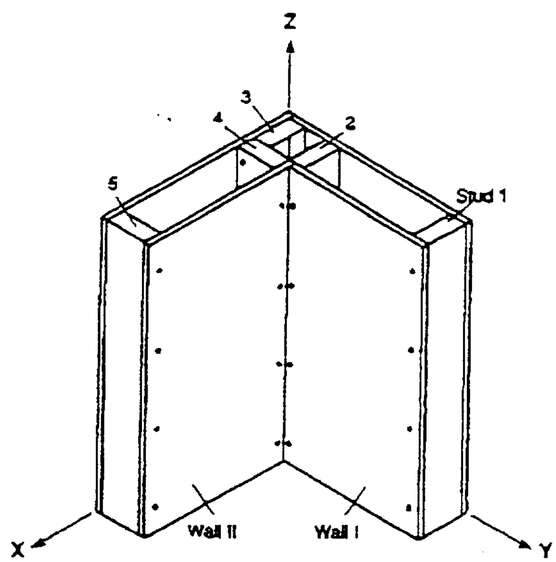


Figure 3.44 Detail of the Wall-to-Wall Connection
(after Groom and Leichti, 1994).

Six corner connection specimens were tested under static cyclic loading to cause the two walls to open and close relative to one another. The experimental setup for this series of tests is shown in Figure 3.45. The hysteresis loading pattern consisted of one cycle in which the maximum displacement in either direction was 13 mm (0.5 in). The sequence for the first corner specimen was close-open-close (back to the original unloaded position). The second specimen was loaded in the opposite sequence, open-close-open. This alternating pattern was followed for the remaining four tests. Typical results of the connection tests are shown as the hysteresis curves in Figure 3.46. The average hysteresis curves for the six connection specimens were used to verify the finite element model.

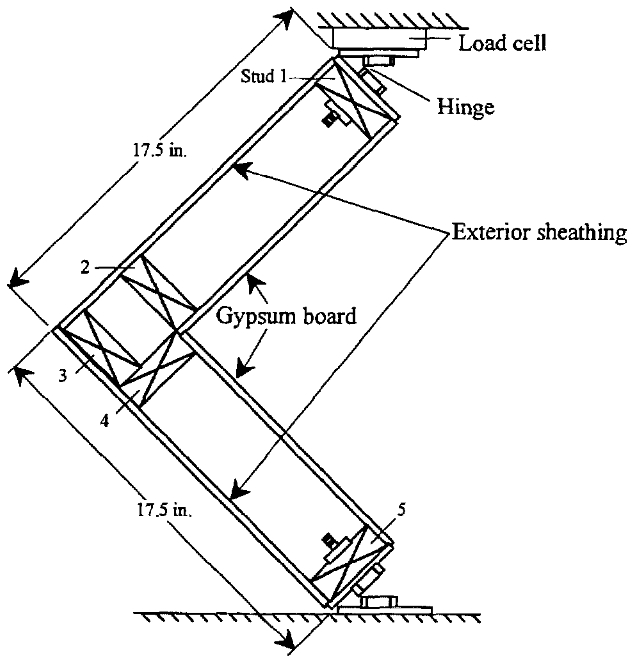


Figure 3.45 Experimental Setup for Wall-to-Wall Connection Test
(after Groom and Leichti, 1994).

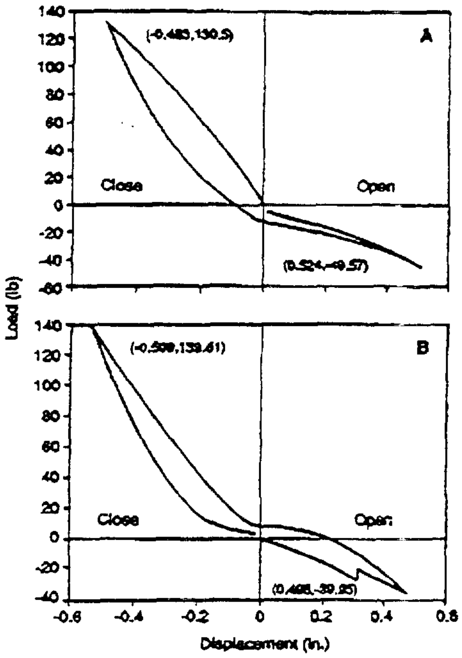


Figure 3.46 Hysteresis Curves for Wall-to-Wall Connection Tests
(after Groom and Leichti, 1994)

3.5 SUMMARY

3.5.1 Full-Scale House Testing

A relatively small number of full-scale houses have been tested during the past 25 years. These tests have been performed on one- and two-story houses under field and laboratory conditions. The range of objectives included:

1. Determination of the behavior of the house under simulated wind loads,
2. Determination of the applicability of drift criteria for medium- and high-rise buildings to low-rise, single-family houses,
3. Evaluation of the racking behavior of a 40+ year old house against current wind load criteria,
4. Determination of the lateral load distribution to interior and exterior shear walls and the lateral load resistance provided by horizontal diaphragms, and
5. Determination of the behavior of a factory-built housing module under static and reversed cyclic lateral loading.

Table 3.14 summarizes the full-scale house experimental studies reviewed for this report.

Table 3.14 Summary of Full-Scale House Test Reports.

REF.	LAB. OR FIELD TESTS	SHAPE OF HOUSE	PLAN DIMENSIONS in meters (feet)	TYPES OF LOADING		LOAD TO FAILURE?
				Monotonic	Cyclic	
Yokel et al. (1973)	FIELD	RECTANG. 2-story	14.3 x 7.9 (47 x 26)	X		NO
Yancey et al. (1973)	LAB	RECTANG. 1-story	18 x 3.7 (59 x 12)	X	X	NO
Tuomi et al. (1974)	LAB	RECTANG. 1-story	7.3 x 4.9 (24 x 16)	X	X	NO
Boughton et al. (1982)	FIELD	L-SHAPE 1-story	18.3 x 13.1 x 6.6 (60 x 43 x 22)	X		YES
Phillips (1990)	LAB	RECTANG. 1-story	9.8 x 4.9 (32 x 16)		X	YES

3.5.2 Shear Walls

Shear wall behavior under lateral load has been the focus of most studies on wood-frame house subassemblies. In most of the early studies, 2.4 m x 2.4 m (8 ft x 8 ft) standard wood frames, sheathed with plywood, were tested under static monotonic loading, usually to failure. The tests were conducted according to ASTM E72 because it was the only standard method available for many years. ASTM E72 was intended as a means of comparing the lateral resistance of sheathing materials such as plywood with the resistance provided by let-in, diagonal corner bracing. Thus, E72 did not readily support experiments with other objectives such as: 1) to determine the lateral resistance of narrow shear walls, 2) to determine the effects of openings on shear wall behavior, 3) to compare the behavior of walls as a function of nail type and pattern, and 4) to study shear wall behavior under cyclic and dynamic loads.

During the past twenty years, many of the experimental studies have used ASTM Standard E564-76 because it accommodates a much wider range of test variables (e.g. size of specimens, type and location of anchorage, and presence of door and window openings). During the past ten years, many of the shear wall experiments have incorporated cyclic loading and a few have used dynamic loading protocols. However, there are still no standardized procedures for testing shear walls under cyclic or dynamic loading. As noted in Section 3.3, many of the recent shear wall tests have been conducted to verify finite element models and support analytical parametric studies. Table 3.15 summarizes the shear wall studies covered by this literature review.

Other recent shear wall studies not covered by this review include: Dinehart and Shenton (1998a); Dinehart and Shenton (1998b); Ge, Gopalaratnum, and Liu (1991); Shenton, Dinehart, and Elliott (1998); Skaggs and Rose (1996); and Stewart, Dean, and Carr (1988).

3.5.3 Intercomponent Connections

The structural integrity of the connections between subassemblies under earthquake and wind loading is key to single-family wood-frame houses performing as three-dimensional structures. In addition, finite element model analysis requires load-displacement and rotational stiffness data to model elements that account for the response of intercomponent connections. However, there have been very few experimental studies conducted with the explicit objective of defining the strength, stiffness, ductility, energy dissipation capacity, and damping ratio of these connections.

Table 3.15 Summary of Shear Wall Test Reports.

Reference	Type of Loading	Size of Specimens	Number of Specimens	Type of Sheathing	Remarks
Easley et al. (1982)	Static monotonic	2.4 m x 3.7 m	8	Single Plywood	
Wolfe (1983)	Static monotonic	H - 2.4 m W - 2.4, 4.9, 7.3 m	30	Single Gypsum Board Bare Frame	Used ASTM E564; 3 different bracing methods
Patton-Mallory et al. (1984)	Static monotonic	H - 0.6 m W- 0.6, 1.2 , 1.8, 2.4 m	200	Plywood & Gypsum Wallboard	5 sheathing combinations; all specimens loaded to failure
Patton-Mallory et al. (1985)	Static monotonic	H - 2.4 m W- 2.4, 4.9, 7.3 m	11	Single Gypsum Wallboard	Used ASTM E564; gypsum panels oriented vertically and horizontally
Nelson et al. (1985)	Static monotonic	2.4 m x 2.4 m or 3.2 m x 2.4 m	7	Double Hardwood Panels	Specimens were longitudinal segments of a single-wide mobile home
Ohashi & Sakamoto (1989)	Cyclic	4.5 m x 5.0 m	2	Ext. - Cement chip board or mortar Int. - Gypsum	Two-story high frames; constant vertical load; specimens had door & window openings
Dolan (1989)	Static monotonic, cyclic & dynamic	2.4 m x 2.4 m	42	Single Plywood Single Waferboard	Vertical load applied to 6 walls
Oliva (1990)	Static monotonic, cyclic, & dynamic	2.4 m x 2.4 m	15	Single Gypsum Wallboard	Constant vertical load, Glued & unglued wall panel; ASTM E564 for static tests
Dolan & Madsen (1992a)	Static monotonic & cyclic	2.4 m x 2.4 m	11	Single Plywood Single Waferboard	Monotonic tests conducted according to ASTM E564
ATC (1995)	Static monotonic & dynamic	0.9 m x 2.4 m	5	Single Plywood	3-cycle sine wave displacement for dynamic tests
Dolan & Johnson (1997)	Cyclic	12.2 m x 2.4 m	5	Exterior - Plywood Interior - Gypsum Board	Sequential Phased Displacement used for loading pattern
Dolan & Johnson (1997)	Monotonic	12.2 m x 2.4 m	5	Exterior - Plywood Interior - Gypsum Board	Completed cyclic load tests run on identical wall specimens
Lam, Prion & He (1997)	Static monotonic & cyclic	7.3 m x 2.4 m	11	Single Oriented Strand board	Sinusoidal displacement function; vertical load applied in all tests

3.6 RESEARCH NEEDS

Based on the literature review presented in this chapter, several areas of needed research are identified:

1. *Develop standard test protocols for testing full-scale houses under lateral load:* The loading procedures and types of measurement should be dependent upon the test objective (e.g. verification of analytical models, defining load paths, or determining the contribution of subassemblies).

2. *Conduct additional full-scale house tests to study overall structural behavior:* Conventional one- and two-story houses need to be tested under simulated wind and earthquake loads to add to the existing sparse database. Full-scale test results are needed to better define the contributions of the various building subassemblies to lateral load resistance. The effects of such variables as building shape, fastener type and spacing, sheathing material type, and type of connections between subassemblies need to be quantified.

3. *Develop standard methods for conducting cyclic and dynamic tests on shear walls and intercomponent connections:* The methods should incorporate standard loading sequences and displacement measurements for hysteretic behavior.

4. *Conduct more tests on intercomponent connections:* Cyclic and dynamic tests should be conducted on different types of connection details for the junctions between different subassemblies (e.g. wall-to-wall, floor-to-wall, and wall-to-foundation) to expand the sparse database.

5. *Conduct more shear wall tests:* Monotonic and cyclic loads up to failure of one- and two-story walls with different aspect ratios and various configurations of openings.

6. *Conduct tests on nail and screw connections under combined loading conditions:* Sheathing-to-stud and sheathing-to-plate connections should be tested under combined tension and shear and compression and shear loads. Comparative studies of the behavior of nail and screw fasteners should be included in the experimental program.

7. *Conduct shear wall tests on walls with metal studs:* Montonic and cyclic loads should be applied to walls framed with metal studs and traditional sheathing materials.

Due to wide variability in construction practices, material properties, etc., it is also important to conduct probability studies to develop reliable upper and lower bounds for expected responses.

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4.0 ANALYTICAL STUDIES ON THE PERFORMANCE OF WOOD-FRAME HOUSES

4.1 INTRODUCTION

Wood-frame buildings consist of several components such as walls, floors, and roof joined by intercomponent connections such as nails, anchor bolts, metal plates, and other propriety connectors (e.g. hold-down brackets). The performance of wood-frame buildings is influenced by the behavior of the individual components and connections. Therefore, an understanding of the behavior of the different structural components and their connections is essential to accurately predict the performance of a housing unit under different types of loading.

Wood-frame buildings perform well under gravity loads. Considerable damage, however, has been observed to such structures under severe and moderate earthquake motions and severe wind storms (see Chapter 2). It is, therefore, important to understand the behavior of wood-frame buildings subjected to lateral loads generated by earthquakes and hurricanes so that the risk to life and property can be reduced. By using accurate analytical tools to model the different structural components as well as the complete structure, performance parameters such as strength, deformation, ductility, damage states, and failure modes can be reliably predicted. Multiple simulations to study the influence of different parameters on the structural performance may, therefore, be conducted more cost-effectively than testing. Experiments, however, are still required to verify and refine the analytical tools. The end result would be improved design standards and procedures which enable buildings to resist the expected earthquake and wind loads without significant damage or collapse.

Analyzing the structural components of a wood-frame building subjected to lateral loads is a difficult task due to several sources of nonlinearity (e.g. material nonlinearities, nonlinear joints and connections, and discontinuities between adjacent elements), the complex nature of the connections and fasteners, and the wide variability in material properties and construction techniques. Since shear walls are the most important elements in resisting lateral loads, they have been extensively studied by several researchers since the early 1970s. Simplified methods and finite element analyses were performed to predict the behavior of the walls under static lateral loading. A few researchers also attempted to predict the dynamic behavior of the walls subjected to earthquake ground motion.

The development of tools for analyzing complete wood-frame houses started in the mid-eighties. This step included analyzing the different structural components as well as the intercomponent connections in an assembled model. Comparisons with experimental studies were performed to assess the validity of the proposed models and analyses. The following sections present a brief description of the analytical studies performed on different structural components and connections as well as the complete building.

4.2 WOOD STUD SHEAR WALLS

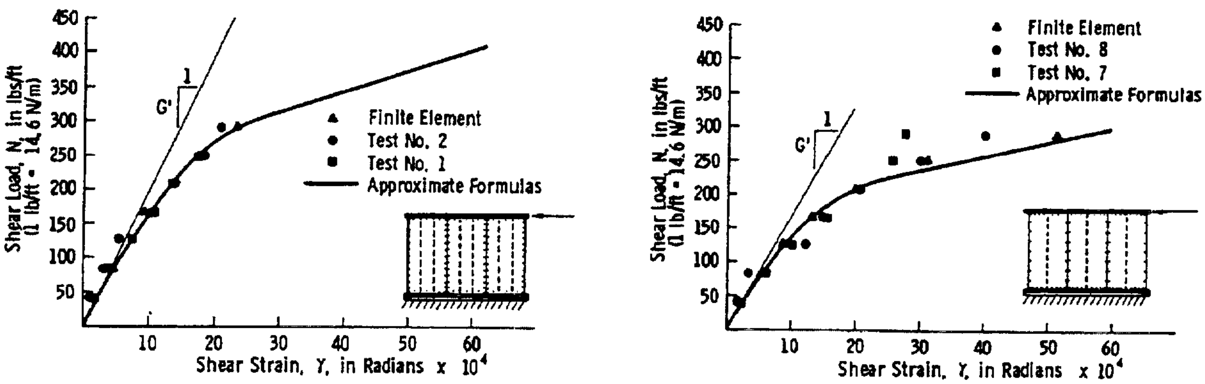
Wood stud shear walls are commonly used in residential and low-rise buildings to provide lateral stiffness and transmit in-plane and out-of-plane forces to the foundation. A typical wall consists of vertical studs at constant spacing, horizontal upper and lower plates (known as top and sole plates, respectively), and sheathing panels which are fastened to the framing members to create a plate-like structure. The behavior of wood shear walls is complicated because of the nonlinear characteristics of the connections, the discontinuity in the sheathing material where the transfer of compressive forces between sheathing panels is nonlinear and the transfer of tension is not possible, and the non-isotropic behavior of wood and wood composites. It should be noted that the in-plane diaphragm behavior of the floors and roofs is similar to that of shear walls. Therefore, it is important to understand the load-deformation behavior of shear walls, or diaphragms in general. Several investigators have analytically studied the behavior of wood stud shear walls, and attempted to predict their behavior under different loading patterns and correlate their results with those from experimental studies. Most of those analytical studies can be divided into the following two categories: 1) closed form and simplified methods and 2) finite element analyses. The following presents a brief discussion of the studies in each category.

4.2.1 Closed Form and Simplified Methods

A closed form solution to calculate the racking (in-plane shear) strength of wood shear walls was developed by Tuomi and McCutcheon (1978). In their derivation, they assumed that the stud frame distorts as a parallelogram while the sheathing remains rectangular and that the load-slip relationship for the nail connectors is linear. The derivation was based on equating the internal energy absorbed by the nails to the external work exerted by the racking force. The racking strength of the wall was estimated in terms of the panel geometry, the number and spacing of nails, and the lateral resistance of a single nail. The theoretical results were in close agreement with the experimental data for two panel sizes; small-scale [0.6 m by 0.6 m (2 ft by 2 ft)] and full-scale panel [2.4 m by 2.4 m (8 ft by 8 ft)]. Robertson (1980), in his discussion of the Tuomi-McCutcheon model, argued that contrary to the experimental results, the model yielded a constant shear strength per unit length regardless of any change in wall length. He also pointed out that the model did not include the influence of vertical loads nor the variation of nail spacing.

Itani et al. (1982) improved the Tuomi-McCutcheon model by including the shear deformation of the sheathing and representing its stiffness with two diagonal springs. The model allows the determination of the racking resistance of continuous wall panels with or without openings. In a later study, McCutcheon (1985) expanded the Tuomi-McCutcheon model by considering the nonlinear nail load-slip behavior and the shear resistance of the sheathing material. Using power curves to define the nonlinear nail-slip behavior ($P = a\Delta^b$ where P is the nail force, Δ is the nail slip, and a and b are constants), it was possible to predict the racking performance of the wood shear walls where their load-deformation curve was also found to be a power function. The study emphasized the influence of the load-slip behavior of the fasteners on the overall response of shear walls. Comparisons of the computed load-displacement behavior with experimental results from several full-scale plywood-, gypsum-, and flakeboard-sheathed walls showed generally good agreement.

In a parallel effort, Easley et al. (1982) developed explicit formulas for the analysis of wood frame shear walls. Their model was similar to that used for corrugated metal shear diaphragms. The formulas, which relate the shear load to shear deformation, were derived for the linear and nonlinear load-strain behavior of the wall. Experiments on eight full-scale plywood-sheathed walls [2.4 m high by 3.6 m wide (8 ft by 12 ft)] and linear and nonlinear finite element models were used to verify the formulas. The finite element model consisted of linear plane stress, eight-node isoparametric elements with a quadratic displacement field for the studs and sheathing in addition to linear/nonlinear spring elements of zero length for the nails. Comparisons between the results from the equations and those from the experiments and finite element analyses showed good agreement as shown in the shear load-strain relationships presented in Figure 4.1.



(a) walls with 0.41 m (16 in) stud spacing (b) walls with 0.61 m (24 in) stud spacing

Figure 4.1 Comparison of the Shear Load-Strain Relationships from Experiments, Formulas, and Finite Element Analysis (after Easley et al., 1982).

Naik et al. (1984) proposed a mechanical model which simulates the nonlinear behavior of shear walls. The model consisted of a hinged square frame of rigid bars stiffened with diagonal springs of equal stiffness which simulate the in-plane horizontal resistance of the wall. A single frame or multiple frames may be used depending on the wall characteristics. When the compressive force in any of the springs exceeds its buckling limit, the spring no longer resists the lateral forces. Therefore, the model was capable of representing the load-deflection behavior of the wall as piece-wise linear segments as shown in Figure 4.2. The characteristics of the springs can be determined from experimental results. Naik et al. (1984) indicated that their model can be used for static and dynamic analysis of wood shear wall buildings.

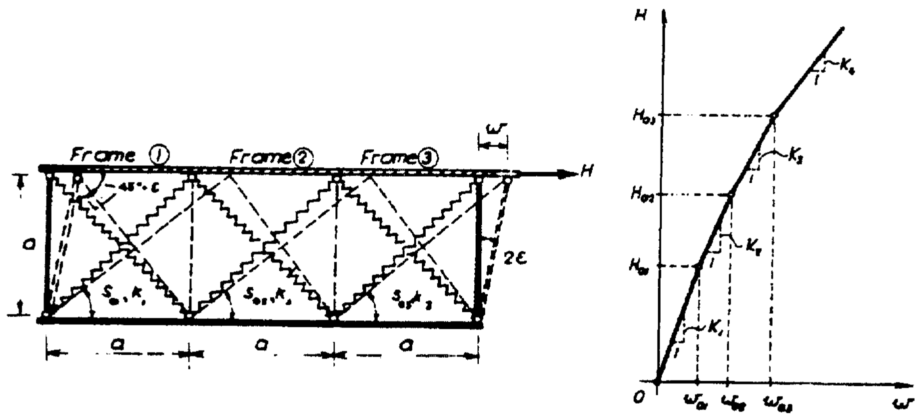
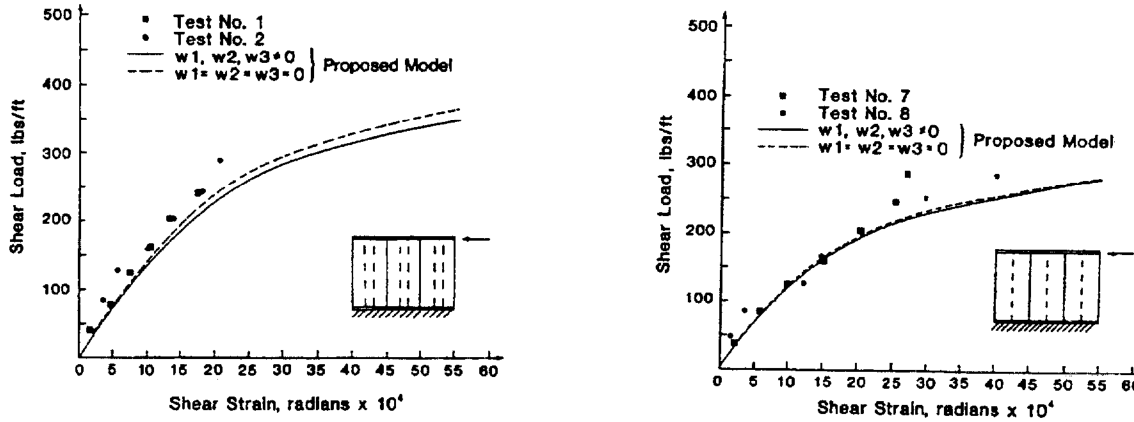


Figure 4.2 Three-Frame Model and its Load-Deflection Behavior
(after Naik et al., 1984).

Gupta and Kuo (1985) proposed a new model to represent the shear behavior of wood-frame walls using strain energy relationships. In their model, they assumed that under a horizontal load, the frame deforms into a parallelogram and the sheathing resists this deformation. As a result, the horizontal and vertical edges of the sheathing have different relative angles to the frame. A sinusoidal shape was assumed for the deformed studs based on the distribution of nail forces. Consequently, the motion of a single panel could be defined in terms of four generalized coordinates: 1) the rotation of the frame, 2) the relative rotation between the horizontal edges of the sheathing panels and horizontal members of the frame, 3) the relative rotation between the vertical edges of the sheathing panels and vertical members of the frame, and 4) the amplitude of the sinusoidal displacement of the frame. The behavior of the nail fasteners can be modeled as linear or nonlinear. The results from Gupta and Kuo's model were compared with those from experiments by Easley et al. (1982) and satisfactory agreement was observed as shown in Figure 4.3 which presents the load-strain relationship for the same walls shown in Figure 4.1. This close agreement was not surprising since Easley's finite element results were used as a justification for the use of the sinusoidal shape of the deformed studs as well as the nail force distribution. The results indicated that assuming the studs to be infinitely rigid in bending (case when $w_1 = w_2 = w_3 = 0$ in Figure 4.3 where w_i represents the sinusoidal amplitude of the deformed studs) had only a slight effect on the overall wall behavior. Later, Gupta and Kuo (1987a) improved their model by including uplift deformation in the studs due to horizontal loading. Their model showed the dependence of the shear load-deflection behavior of the wall on the vertical load.

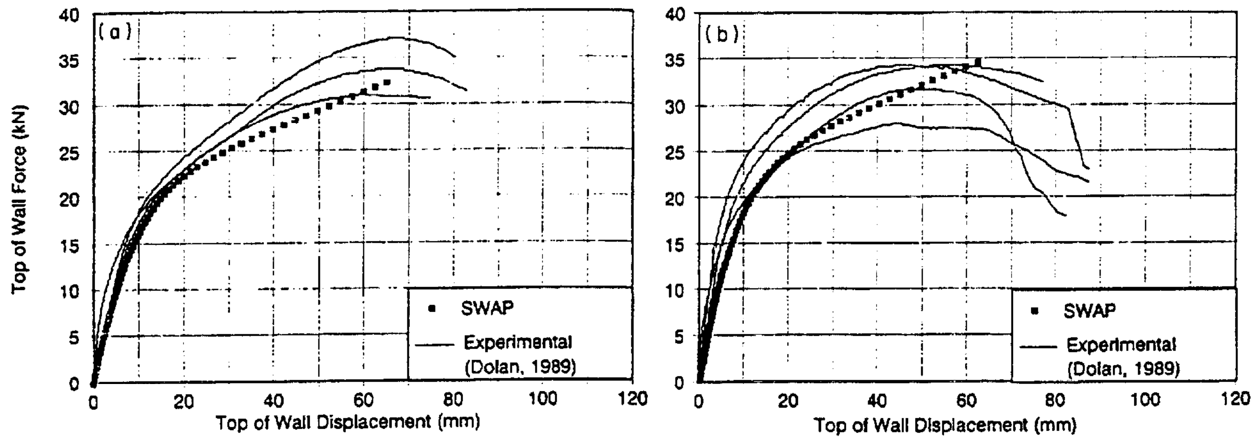


(a) walls with 0.41 m (16 in) stud spacing

(b) walls with 0.61 m (24 in) stud spacing

Figure 4.3 Comparison of the Results from Gupta-Kuo's Model with those from Easley's Tests (after Gupta and Kuo, 1985) ($w_1 = w_2 = w_3 = 0$ signifies infinitely rigid studs).

Filiatrault (1990) developed a simplified static model to predict the stiffness and ultimate load capacity of shear walls. The model, which was similar to that of Gupta and Kuo (1985) but with more degrees of freedom, was incorporated in a computer program called SWAP (Shear Wall Analysis Program). In this model, sheathing panels were modeled using four degrees of freedom: one for uniform shear deformation, two for rigid body translations, and one for rigid body rotation in addition to the lateral displacement of the top of the frame. As a result, the shear walls could be modeled using 4N+1 degrees-of-freedom where N is the number of sheathing panels. Nail lines connecting the sheathing to the studs were lumped into one element. The model included the nonlinear load-slip characteristics of the fasteners represented by a monotonic curve up to a maximum displacement (corresponding to maximum load) followed by a line with negative slope which approaches zero load at failure. The nonlinear load-slip curve was similar to that used by Dolan (1989), see Section 4.2.2 and Figure 4.8(a). The computed deflections and forces were compared with the experimental results presented by Dolan (1989) for seven 2.4 m by 2.4 m (8 ft by 8 ft) walls; three were plywood-sheathed and four were waferboard-sheathed. The comparisons indicated close agreement between the analytical and experimental results as shown in Figure 4.4.



(a) plywood-sheathed walls

(b) waferboard-sheathed walls

Figure 4.4 Comparison of the Results from Filiatrault's Model with those from Dolan's Static Tests (after Filiatrault, 1990).

Filiatrault (1990) also modified his static model so that it could perform dynamic analysis of shear walls. In his model, the hysteretic behavior of the fasteners was represented by four exponential curves such that the envelope of the hysteresis loops used in the dynamic analysis coincided with the load-slip relationship for the static case. The hysteretic model was similar to that suggested by Dolan (1989), see Section 4.2.2 and Figure 4.8(b). Stiffness degradation and pinching were incorporated into the model and a constant mass and viscous damping matrices were utilized. Reasonable accuracy of the analytical results was reported when compared with the shake table tests performed by Dolan (1989) under earthquake ground motions as shown in Figure 4.5 for the 1952 Kern County earthquake.

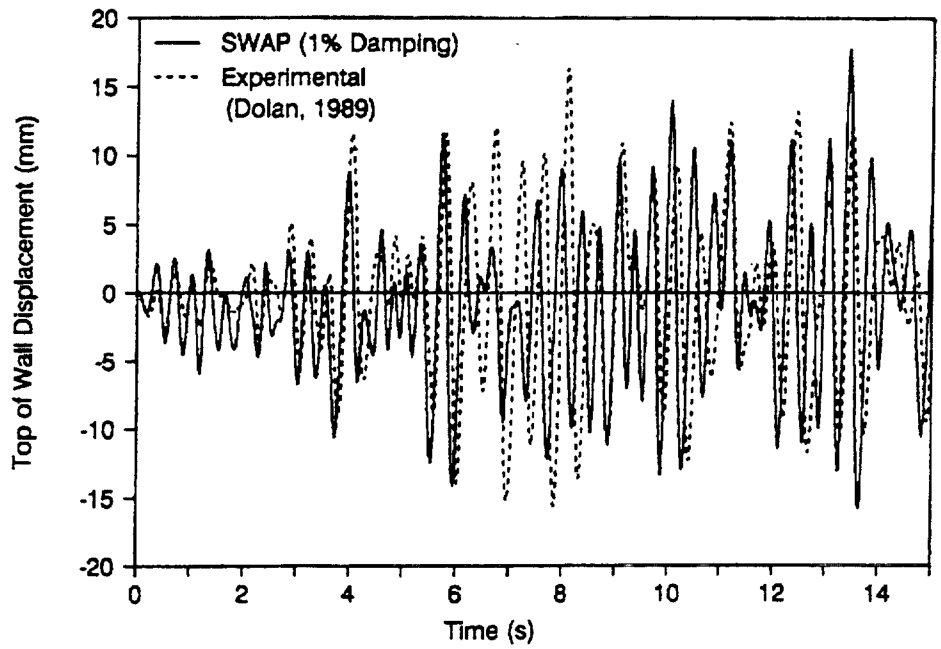


Figure 4.5 Dynamic Response of a Shear Wall Subjected to the 1952 Kern County Earthquake from Filiatrault's Model and from Shake Table Test by Dolan (after Filiatrault, 1990).

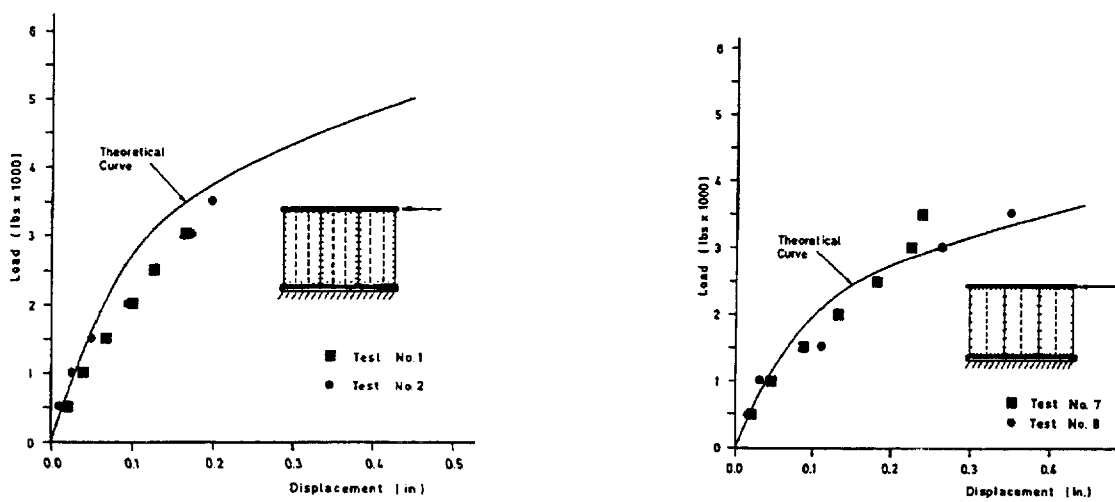
In a later study, Yoon and Gupta (1991) generalized the Gupta-Kuo model to obtain a closed form solution for the analysis of shear walls with and without uplift deformation and using a linear load-slip behavior for nails. For nonlinear load-slip relationships, the problem could be solved using a step-by-step iterative procedure. Comparisons with different analytical and experimental results were made to verify the model. Ductility analysis of several walls showed that shear walls do not possess much ductility unless the nail joints were considerably ductile. Based on the results of several analyses and possible failure modes, Yoon and Gupta (1991) introduced simple design charts for the design of shear walls.

4.2.2 Finite Element Analyses

Polensek (1976) presented a finite element model for wood stud walls subjected to flexural and compressive loads. The model consisted of I-beam-column elements representing studs and plate elements representing sheathing. Fastener stiffness was assumed to be distributed along the nail line. The model accounted for the partial composite action between studs and sheathing, the non-uniform stiffness distribution within the walls, and the nonlinear behavior of studs. A linear step-by-step procedure was used to account for the nonlinear relationship between the shear and slip of the fastener joints. The model was capable of accurately predicting deflections and stresses at service and ultimate loads. Polensek (1976) reported that the accuracy of his solution depended mainly on the accuracy of the nail load-slip relationship and the material properties.

In another study, Foschi (1977) developed a finite element model of walls to predict their behavior under shear loading. The model consisted of: two-dimensional (2-D) linear orthotropic twelve-node plane stress elements for the sheathing, linear beam-column elements for the frame members, and nonlinear spring elements for the connections. To reduce the complexity of the analysis, every nail line was grouped into one spring element. The analysis gave good estimates for the load-deformation characteristics of the walls when compared with the experimental results of a 6 m by 8 m (20 ft by 26 ft) wall sheathed with lumber decking and overlaid plywood that was tested at Oregon State University (Johnson, 1971).

Itani and Cheung (1984) developed a finite element model to predict the static load-deflection behavior of sheathed wood diaphragms under racking loads. The model consisted of linear beam elements for framing members (studs, header, and sill), linear 2-D plane stress 4-node quadrilateral elements for sheathing, and a nonlinear joint element for nail connections. The nonlinear joint element was a series of mutually perpendicular spring pairs representing the nail fasteners and connected the sheathing nodes to those of the frame. The program NONSAP (Bathe et al., 1974) was modified by Itani and Cheung to incorporate the nonlinear joint element. Reasonable agreement between experimental measurements from two-panel waferboard-sheathed walls tested by Foschi (1982a) and the three-panel plywood-sheathed walls tested by Easley et al. (1982), and analytical predictions were obtained. Figure 4.6 shows a comparison between computed and measured load-deflection curves for walls tested by Easley et al. (1982). Itani and Cheung (1984) concluded that the properties of nailed joints are the controlling factor of the performance of sheathed diaphragms.

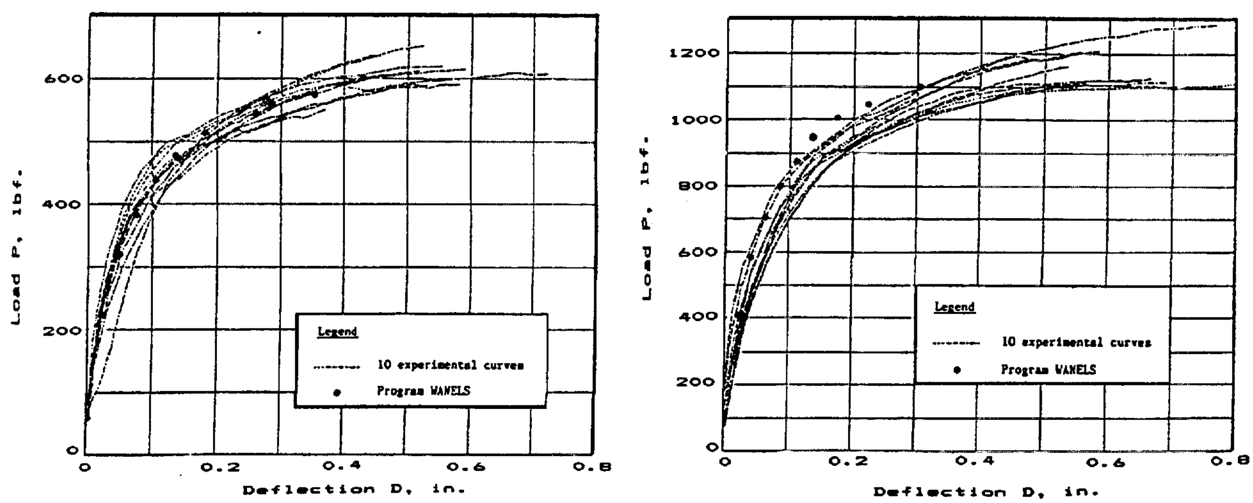


(a) walls with 0.41 m (16 in) stud spacing (b) walls with 0.61 m (24 in) stud spacing

Figure 4.6 Load-Displacement Relationships from Experiments and Finite Element Model (after Itani and Cheung, 1984).

In another study, Itani and Robeldo (1984) developed a finite element model of the wall loaded in shear. The model was incorporated in a computer program called PANFRA (Panel and Frame Analysis). The model consisted of beam elements for framing members, constant strain triangular elements for sheathing, and a nonlinear joint element for nail fasteners. Based on the comparisons with experimental results, they concluded that their model provided reasonable predictions of the behavior of walls with and without openings.

Gutkowski and Castillo (1988) developed a finite element model for single- and double-sheathed wood walls loaded in shear. The model was incorporated in a computer program called WANELS. The model consisted of beam elements with axial deformation for framing members, 2-D orthotropic plane stress elements for sheathing, and nonlinear non-dimensional fastener elements to model the interlayer slip between the stud frame and the sheathing. The discontinuity between sheathing panels (sheathing gap behavior) was approximated by springs placed between adjacent nodes of adjacent sheathing panels. The springs had a discontinuous load-deformation behavior, i.e. zero forces under tension and linear force distribution under compression. Experimental results from several gypsum wallboard- and plywood-sheathed small-scale walls tested by Patton-Mallory (1983) and Patton-Mallory et al. (1984) were used to verify the mathematical model and good agreement between the analytical and experimental results was observed as shown in Figure 4.7.



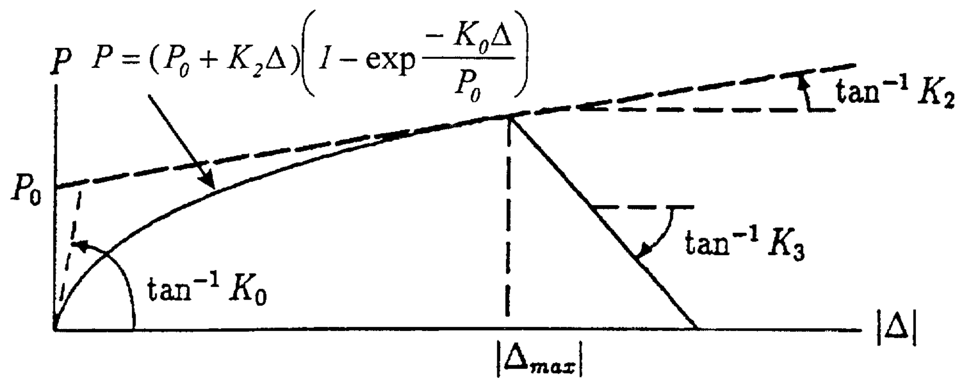
(a) gypsum board sheathing on one side

(b) plywood sheathing on one side

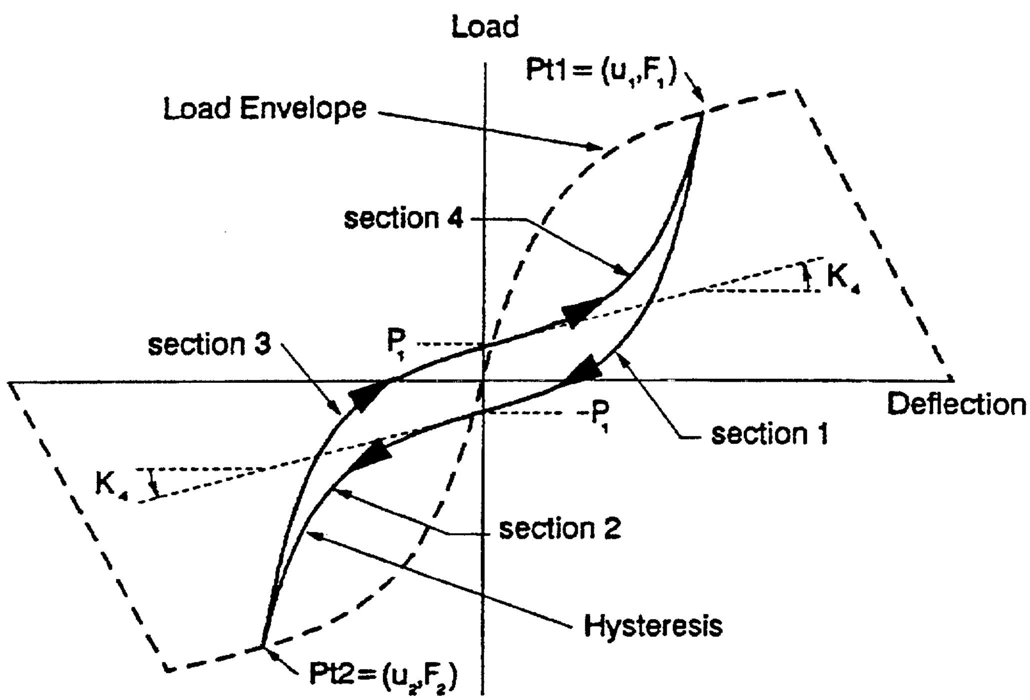
Figure 4.7 Load-Deformation Relationships from Experimental and Finite Element Analyses (after Gutkowski and Castillo, 1988).

For analyzing vertical and horizontal wood diaphragms, Falk and Itani (1989) developed a transfer element to account for the transfer of lateral forces through the fasteners from the 2-D plane stress elements representing the sheathing to the beam elements representing the framing. The transfer elements were used to account for the stiffness of individual fasteners through the use of nonlinear spring pairs. A reduction of 40% in the number of degrees of freedom was noted compared to the earlier Itani and Cheung (1984) model. The model was used in analyzing shear walls in addition to floor and ceiling diaphragms. A comparison of the finite element analysis with the experimental results indicated that the model was capable of predicting the diaphragm response to shear loading. Parametric studies indicated that the nail spacing had a greater effect on the diaphragm stiffness than the nail load-slip behavior.

Dolan (1989) developed a finite element model of shear walls that consisted of the following elements: 1) beam elements for the framing, 2) bilinear corner connector elements for the connection between the framing members, 3) plate elements for the sheathing, 4) sheathing connector elements consisting of nonlinear 3-D spring elements for the fasteners, and 5) bilinear bearing connector elements for the gap between adjacent sheathing panels. The model was used in the nonlinear static (computer program SHWALL) and dynamic (program DYNWALL) analyses of shear walls. Figure 4.8(a) shows the load-displacement curve for the nonlinear connector elements where the straight line portion with the negative slope represents the behavior of the connector after reaching the connection capacity at Δ_{max}. For the dynamic analyses, constant mass and damping matrices were utilized and an idealized hysteretic curve for the connector elements was selected to account for the energy dissipation in the system. The hysteresis loop, shown in Figure 4.8(b), is defined in four sections by four exponential equations representing loading and unloading. It can be observed that the static load-deflection curve (Figure 4.8a) is the envelope for the hysteretic curves (Figure 4.8b) of the connectors. Results from the static and dynamic full scale shear wall tests conducted by Dolan (1989), see Chapter 3, were used to verify the accuracy of the finite element model and close agreement was observed. In addition, a closed form mathematical model consisting of a nonlinear single-degree-of-freedom structure was introduced to predict the natural frequencies and the steady state response of the shear walls under harmonic base excitation and its accuracy was verified.



(a)



(b)

Figure 4.8 (a) Load-Displacement Curve and (b) Assumed Hysteresis Loop for Sheathing Connector Elements (after Dolan, 1989).

In an attempt to model a complete house, Kasal (1992) and Kasal and Leichti (1992a) developed a detailed 3-dimensional (3-D) finite element model of shear walls. The model consisted of: 1) linear 2-D orthotropic rectangular shell elements (six-degree-of-freedom per node) for studs and sheathing, 2) nonlinear one-dimensional spring elements (one for withdrawal and two for shear resistance) for fasteners, 3) 3-D brick-type elements (three translational degree-of-freedom per node) for headers above openings, and 4) gap elements between sheathing panels. Verification of the detailed model was performed for combined compression and out-of-plane flexural tests conducted by Polensek (1975) and for in-plane shear tests by Easley et al. (1982) where reasonable agreement was observed as shown in Figure 4.9 for compression and flexural loading. The results indicated that when the wall was loaded in flexure by wind pressure and at the same time acting as a shear diaphragm, there was no strong coupling (interaction) between bending, shear, and axial behavior.

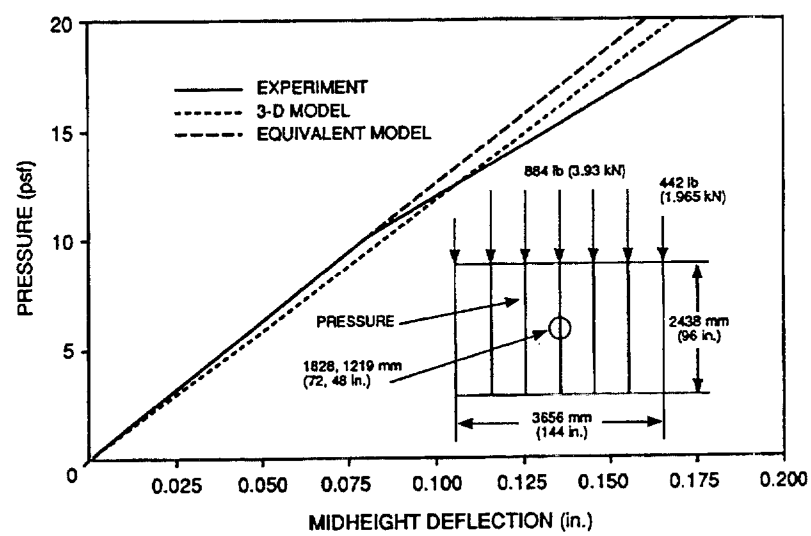


Figure 4.9 Experimental and Analytical Results for Wall Tested by Polensek (1975) under Bending and Compression (after Kasal and Leichti, 1992a).

Kasal (1992) and Kasal and Leichti (1992a) indicated that while the detailed 3-D model is reliable, it cannot be implemented in the analysis of a complete building due to the enormous number of degrees-of-freedom. Therefore, an equivalent model with a minimum number of degrees-of-freedom was recommended and required to respond in the same manner as the detailed model. The equivalent model was based on the idea that the work done by the external forces on the reduced model is equal to the work required to deform the original structure. Since in the full structure, reaction forces and deformations along the boundaries are of interest, attention was focused on the boundaries where walls are connected. The equivalent model consisted of: 1) beam elements for edge studs, top and sole plates, and header and sill plates around openings, 2) truss elements for internal studs and vertical studs above and below openings, 3) 2-D orthotropic plate elements for bending and torsional rigidity of the wall, and 4) nonlinear diagonal spring for shear behavior. The properties of the diagonal springs can be obtained from experimental results or a detailed finite element model. Figure 4.10 shows a detailed finite element model of a wall in addition to its equivalent model. The commercial finite element software package, ANSYS (Kohnke, 1989), was used in the analysis of the detailed and the equivalent models, and a good correlation between the two models was reported while a dramatic reduction in the number of nodes, elements, and degrees-of-freedom was achieved. Figure 4.9 shows the response of the equivalent model under compression and flexural loading where good agreements with experimental and detailed analysis are observed.

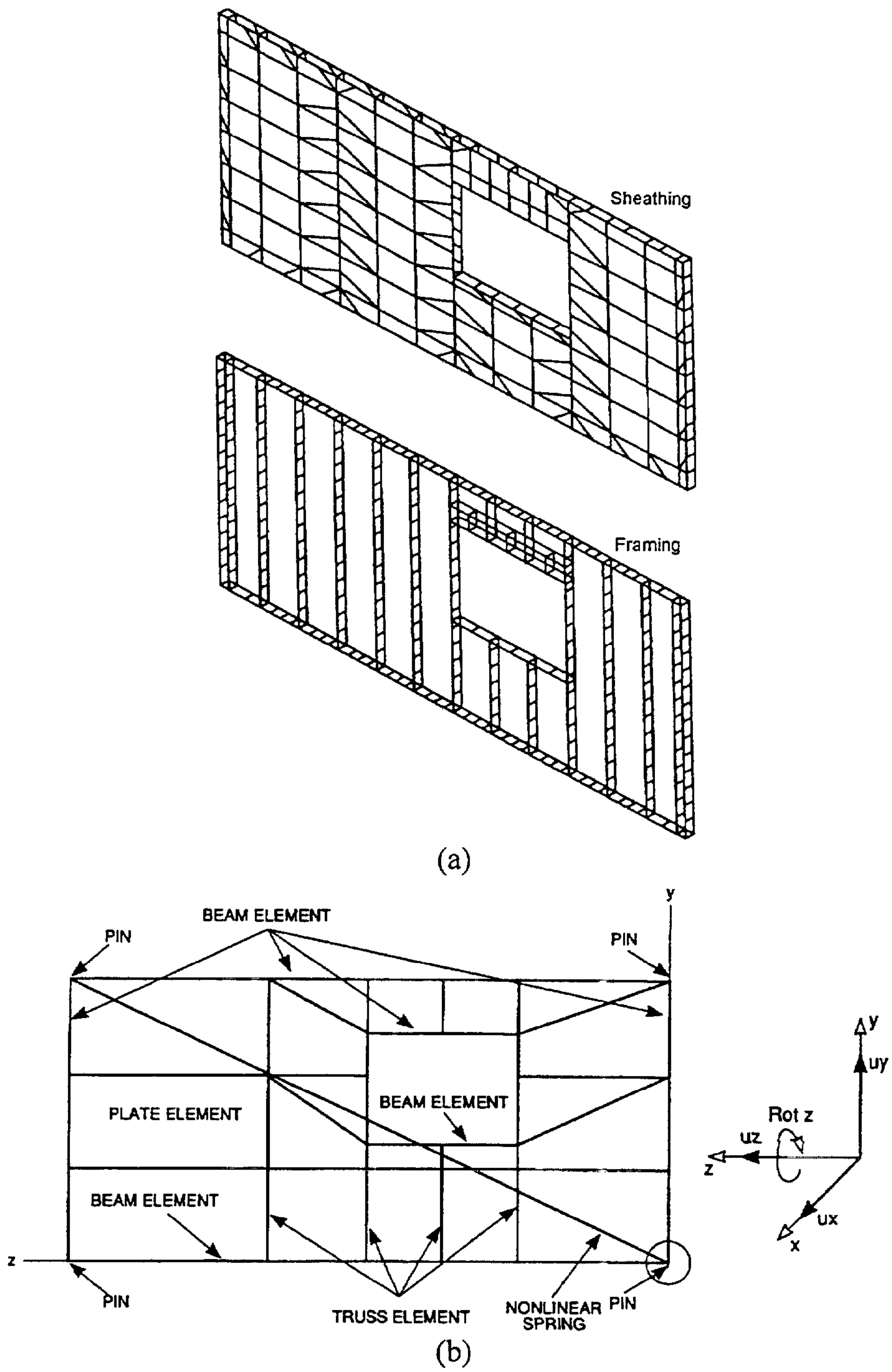


Figure 4.10 (a) Detailed and (b) Equivalent Finite Element Mesh of a Shear Wall (after Kasal and Leichti, 1992a).

More recently, White and Dolan (1995) extended the model proposed by Dolan (1989) and developed a computer program, WALSEIZ, to perform static and dynamic analyses of shear walls with a reduced number of degrees-of-freedom compared to Dolan's model. In their model, the walls are modeled using four elements: 1) 2-D linear beam elements for framing, 2) linear orthotropic plate elements for sheathing, 3) nonlinear springs for fasteners, and 4) bilinear springs to model the gap between adjacent sheathing panels. For static and dynamic analyses, the load-slip behavior and hysteresis loops, respectively, of the springs representing fasteners were the same as those suggested by Dolan (1989), see Figure 4.8. The program was validated using experimental results for two walls tested by Dolan (1989) under monotonic and dynamic loadings where good agreement was observed.

4.3 FLOORS AND ROOFS

Floors in wood-frame structures are typically made of a system of parallel wood beams (joists) whose size and spacing are selected to sustain the expected loads. The joists are covered with sheathing panels made of plywood or wood composites that are nailed to the joists. They carry gravity loads and transfer the in-plane and out-of-plane forces to the vertical substructures and to the foundations. When subjected to a uniform vertical load, floors act as complex two-way structural systems with incomplete composite action due to the slip between layers and discontinuities in the plane of the sheathing due to gaps between different panels. The following summarizes research work related to the bending behavior of floors.

Thompson et al. (1975) presented a linear finite element model of layered wood systems that considered the effect of interlayer slip, variable material properties, and gaps between sheathing panels. In their analysis, the floor was idealized as a set of crossing beams: T-beams (a joist and a composite flange representing the sheathing) along the joist direction, and sheathing strips in the perpendicular direction. The beams were subdivided into finite elements and the deformations of the T-beams and the sheathing strips were matched at the points of intersection. The method was incorporated in the computer program FEAFLO (Finite Element Analysis of Floors). Good agreement between the computed deflections and those from experiments of eleven floors was reported. Sazinski and Vanderbilt (1979) used the FEAFLO program to analyze floors in flexure with linear material and connection properties. Based on their analyses, they recommended design procedures for floor systems.

In another study, Foschi (1982b) developed a linear model based on a combined Fourier series and finite element procedure. The model considered the lateral and torsional deformation of the joists as well as gaps between sheathing panels. The results of the analytical study were compared with those from experiments conducted on three floors under two concentrated loads where good agreement between the analytical and experimental results was observed.

To investigate the effect of nonlinear nail stiffness, Wheat et al. (1983) added nail connection nonlinearity into the Thompson et al. (1975) finite element model and implemented their model into the program NONFLO (Nonlinear Floor Analysis). A direct iterative procedure was used to solve the system of nonlinear equations. When compared with the experimental results for floors under concentrated and uniform loads, the analysis showed satisfactory agreement. The results showed that the deflection at failure (defined as first joist rupture) for the nonlinear model was 10 to 15% larger than that for the linear model, see Figure 4.11, indicating that the linear approximation of nail connection behavior can result in satisfactory estimates of the floors capacity.

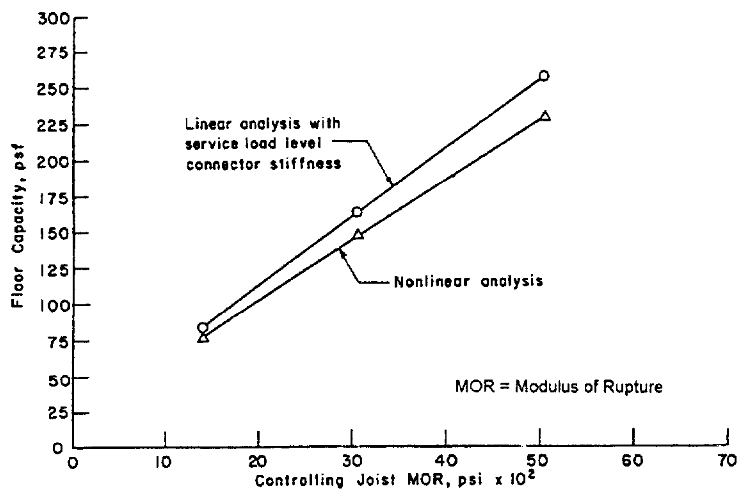


Figure 4.11 Floor Capacity versus Joist Modulus of Rupture for Linear and Nonlinear Analyses (after Wheat et al., 1983).

Roofs are used to transmit gravity loads to the supporting walls and act, together with ceilings, as a horizontal diaphragms. Different roof constructions ranging from rafter systems to pre-fabricated trusses have been used. Wood trusses with metal plate connections have been widely used for roof framing. Unlike shear walls, most roof truss models utilize the linear approach as indicated from experiments (see Wolfe and McCarthy, 1989). Cramer and Wolfe (1989) developed a simplified model to estimate the response and load distribution characteristics of wood trusses under vertical loading. The model represented each truss as simply supported and pin connected. Increased moments of inertia of the top chord were assumed to account for the composite action between the truss members and sheathing. Although the model did not consider the connector plate rigidity, it was capable of accurately predicting the load distribution and deflections of the roof as compared to experimental results.

Lafave (1990) and Lafave and Itani (1992) developed a 3-D finite element model for roof truss wood assemblies. The truss elements were modeled as 3-D beam elements with twelve degrees-of-freedom while the plated truss joints were modeled as semi-rigid connection elements. The computed deflections were in good agreement with those from the experiments. It was concluded that the load-deflection response of the roof trusses was linear up to twice the design loads.

4.4 INTERCOMPONENT CONNECTIONS

Intercomponent connections such as nails, anchor bolts, or metal plates (hangers and T-straps) are used in wood-frame housing construction to connect different substructures such as walls, floors, roof, and foundation, and to transfer forces between them. The behavior of intercomponent connections is complex because of the nonlinearity of the materials, interlayer gaps, material variability, and diversity of construction techniques. As discussed in Chapter 2, damage to or failure of wood-frame buildings during earthquakes and hurricanes are, in most cases, due to failure of intercomponent connections. Intercomponent connections, however, have not been widely studied either experimentally or analytically. The following summarizes research work related to the analytical prediction of the behavior of intercomponent connections.

Polensek and Schimel (1986 and 1988) developed a 2-D nonlinear finite element program "COMPCON" to analyze the rotational capacity of the wall-to-floor connection. The program accounted for the nonlinear behavior of wood materials and nailed joints as well as intercomponent gaps. It included orthotropic rectangular and triangular plane strain elements to represent the behavior of lumber and sheathing materials in addition to a set of two dimensionless and mutually perpendicular springs to represent nail joints and contact surfaces between adjacent materials. The program's accuracy was verified by the close agreement between the analytical and experimental results of nine connection systems. The study indicated a slight nonlinear behavior of the connection. It was also pointed out that the wall stiffness and resistance can be significantly increased by adding a few nails into the exterior sheathing, sill plate, and header.

Groom (1992) and Groom and Leichti (1991, 1994) conducted a detailed finite element study to investigate the behavior of various intercomponent connections typical of wood-frame structures. Their analyses included roof truss-to-wall connection, partition-to-exterior wall connection, exterior wall-to-exterior wall connection, and several metal framing anchors. Their models included material nonlinearities and gap behavior. The method consisted of reducing the number of degrees of freedom from a detailed finite element model of an intercomponent connection with appropriate boundary conditions to a set of energetically equivalent nonlinear springs that exhibit the same moment-rotation or load-displacement relationships of the original connection. Good agreement between analytical results and experiments conducted by Groom (1992), where the connections were similar to those of the full scale house tested by Philips (1990) and Philips et al. (1993), was reported. Figure 4.12 shows the finite element model of the connection between the roof and the top plate of the wall in addition to the computed and experimental moment-rotation relationship (Groom, 1992). Due to its simplicity, the reduced model was incorporated in the 3-D finite element analysis of a complete house studied by Kasal (1992).

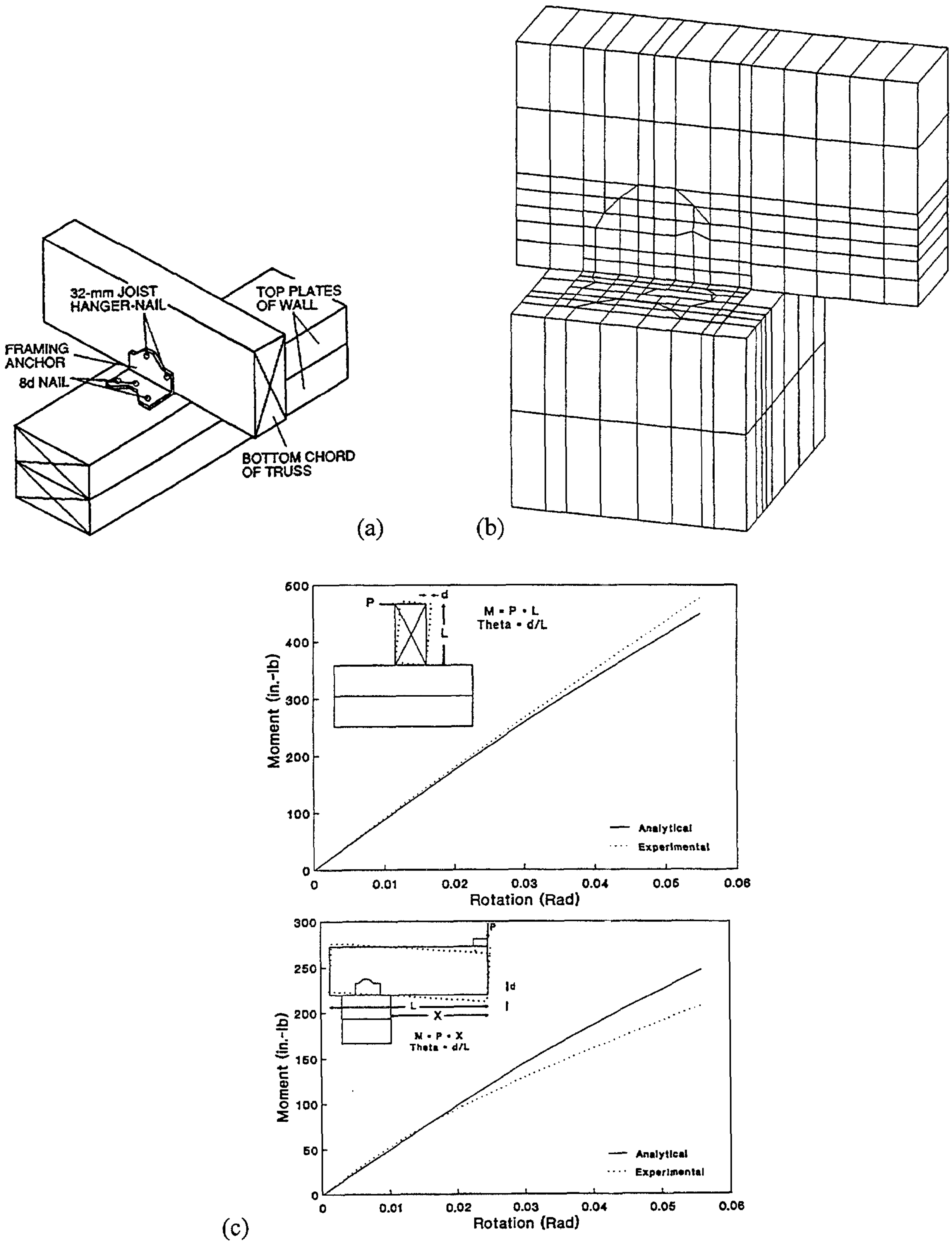


Figure 4.12 Connection between the Roof and the Top Plate of Wall (after Groom, 1992)
(a) connection details (b) finite element model (c) moment-rotation relationships.

4.5 FULL STRUCTURE ANALYSIS

Experimental investigations of full-scale wood-frame structures are limited because of the high costs and difficulties associated with experimental testing. As discussed in Chapter 3, only a few full-scale houses have been tested in the United States, Canada, Japan, and Australia. Analytical procedures which can accurately represent the behavior of a complete house, can be efficient tools for understanding and predicting the behavior of wood-frame houses. Parametric studies involving numerous variations of a building can be performed more cost-effectively than full-scale testing. In 1983, Itani and Cheung (1983) reported that no analytical models were available for the three-dimensional analysis of complete houses. Since then, several investigators have attempted to analytically study the behavior of complete houses, predict their behavior under different loading conditions, and compare the computed results with those from the experimental studies. The following is a brief discussion on analytical studies performed on full housing units.

Gupta and Kuo (1987b) generalized their shear wall models (Gupta and Kuo, 1985 and 1987a) into a macro-element representing any wall or diaphragm in a wood-frame house. They used their model to analyze the building tested by Tuomi and McCutcheon (1974), see Chapter 3. In their analysis, a model of the shear wall which included uplift was used as a basic element. Rigid joints were assumed between vertical diaphragms, and constraint equations were used to ensure rigid body motion of the roof trusses. A limited number of global degrees of freedom was used in the analysis, as shown in Figure 4.13, and several simplifications were employed. The simplifications included ignoring the bending and axial deformation of studs as well as the shear deformation of the sheathing. Reasonable agreement with the test results was obtained. The linear model, however, did not capture the entire load-deformation path.

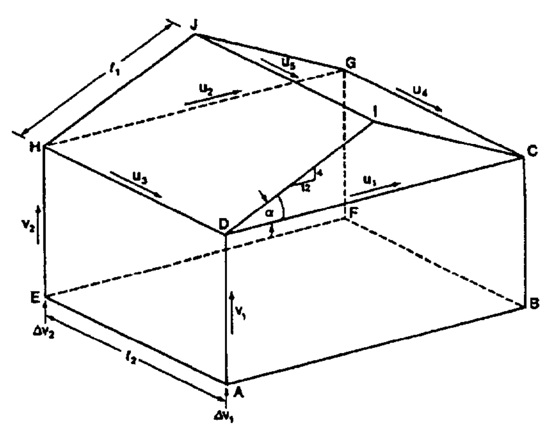


Figure 4.13 Global Degrees-of-Freedom for the Gupta-Kuo Model (after Gupta and Kuo, 1987) (v_i and u_i signify vertical and horizontal degrees-of-freedom, respectively).

While the first program was written specifically to analyze the Tuomi-McCutcheon test house, Kuo and Gupta (1989) later developed a general purpose program, called HOUSE, capable of analyzing different building configurations using their earlier linear model. In a later study, Yoon and Gupta (1991) modified the program HOUSE to compute factors of safety against possible failure modes such as buckling of sheathing panels and slippage of the connecting nails, and added a step-by-step iterative nonlinear analysis capability. This resulted in a new version of the program, called N-HOUSE. Comparisons between the experimental results for the Tuomi-McCutcheon house and those computed by Yoon and Gupta (1991) are shown in Figure 4.14 where good agreement is observed.

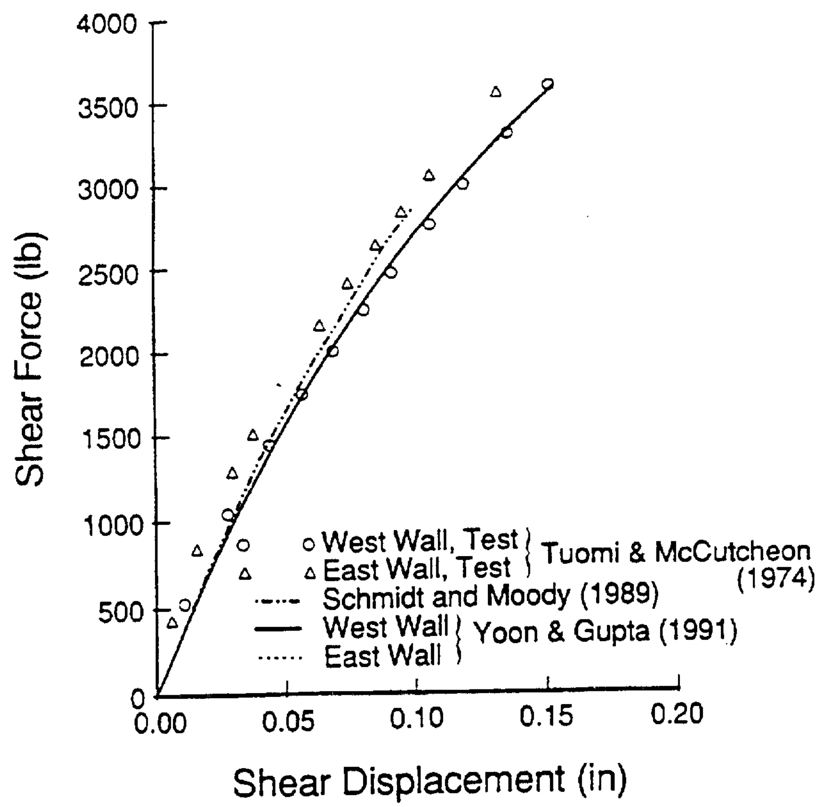


Figure 4.14 Experimental and Analytical Load-Deformation Curves for the Tuomi-McCutcheon House (after Yoon and Gupta, 1993).

Schmidt and Moody (1989) extended the Tuomi-McCutcheon (1978) model for the analysis of shear walls by including the nonlinear behavior of the fasteners and used this model to predict the response of 3-dimensional wood-frame structures subjected to lateral loading. The main assumption of their 3-D model was that the ceiling and roof diaphragms were sufficiently rigid such that the shear walls in each story can be combined into a three-degree-of-freedom system: two horizontal translations and one rotation. The model was validated using the results of two experimental studies. In the first, the analytical predictions were in good agreement with the experimental results from the full-scale house tested by Tuomi and McCutcheon (1974) as presented in Figure 4.14. In the second, the analytical results were in reasonable agreement with the experimental results reported by Boughton and Reardon (1984) when the building was subjected to a uniform lateral load. When the building, however, was loaded with an eccentric point load at the top corner, the analytical results differed from the experimental results. This may be due to the oversimplifications of the model that neglects the out-of-plane stiffness of the walls, the slippage of intercomponent connections, and the shear stiffness of areas above and below the openings, and assumes rigid ceiling and roof diaphragms.

Ge (1991) developed a nonlinear model for analyzing wood-frame houses subjected to lateral loading. In this model, Ge replaced the shear stiffness of the diaphragm element by two nonlinear diagonal elements. The nonlinear load-deformation characteristics of the diagonal elements were calculated from a separate analysis of the diaphragm elements. The model was used in the analysis of the house tested by Tuomi and McCutcheon (1974) and good agreement was reported.

Kasal (1992) and Kasal et al. (1994) developed a nonlinear 3-D finite element models of complete wood-frame structures and used the model in the analysis of the full-scale house tested by Philips (1990), see Chapter 3. The model was designed as an assembly of substructures (walls, roof, and floors) joined by intercomponent connections. The model, thus, consisted of: 1) an equivalent nonlinear shear wall model as illustrated in Section 4.2.2 [Figure 4.10(b)], 2) linear super-elements representing the roof and floors, and 3) nonlinear one-dimensional elements representing the intercomponent connections as suggested by Groom (1992), see Section 4.4. The super-element concept means that the interior degrees-of-freedom of the domain are eliminated through a condensation process and the degrees-of-freedom on the boundaries are retained and connected to the rest of the structure through intercomponent connections. Before condensation, the roof was modeled using truss element for truss members, beam elements for the lower chord, and 2-D orthotropic shell elements for plywood roof sheathing and gypsum board ceiling, while the floors were modeled using 2-D orthotropic shell elements for joists and sheathing and spring elements for nails. Two 3-D finite element models of the house tested by Philips, a coarse and a refined mesh, were developed using the commercial software ANSYS (Kohnke, 1989) and are shown in Figure 4.15. The experimental and analytical wall reaction forces as well as load-deformation curves for the four walls, when the four walls were loaded, are shown in Figure 4.16 where reasonable agreement is observed. When only two adjacent walls out of the four were loaded, the model did not yield accurate results for the unloaded walls. This inaccuracy may be due to the transfer of forces through the roof diaphragm and intercomponent connections. The results indicated that there was no significant difference between the response of the coarse and refined meshes and that the distribution of loads among the shear walls depends on the combination of shear wall stiffness, roof diaphragm action, and intercomponent connections stiffness.

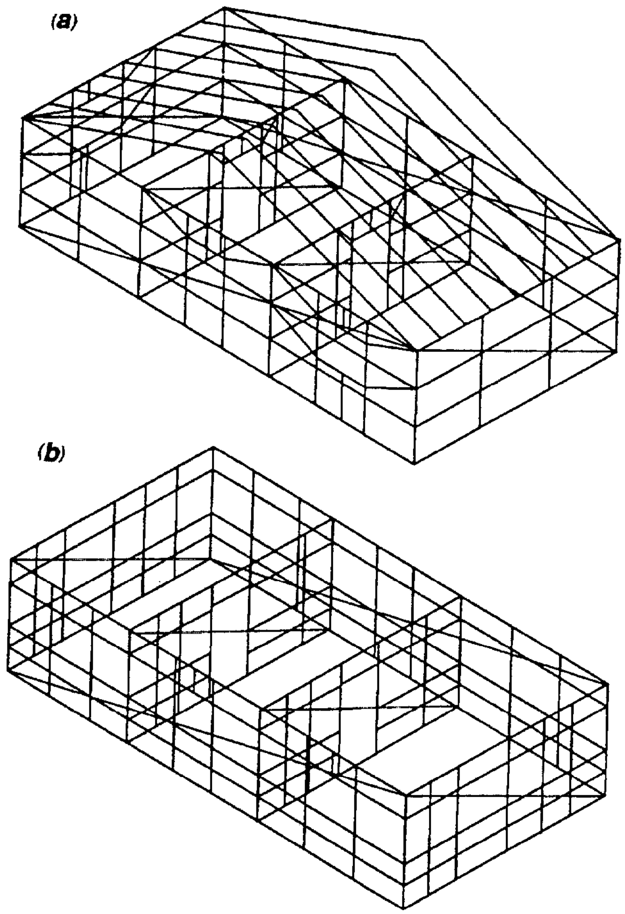


Figure 4.15 Finite Element Model of the Philips House (after Kasal and Leichti, 1992a)
(a) coarse mesh with roof (b) refined mesh without roof.

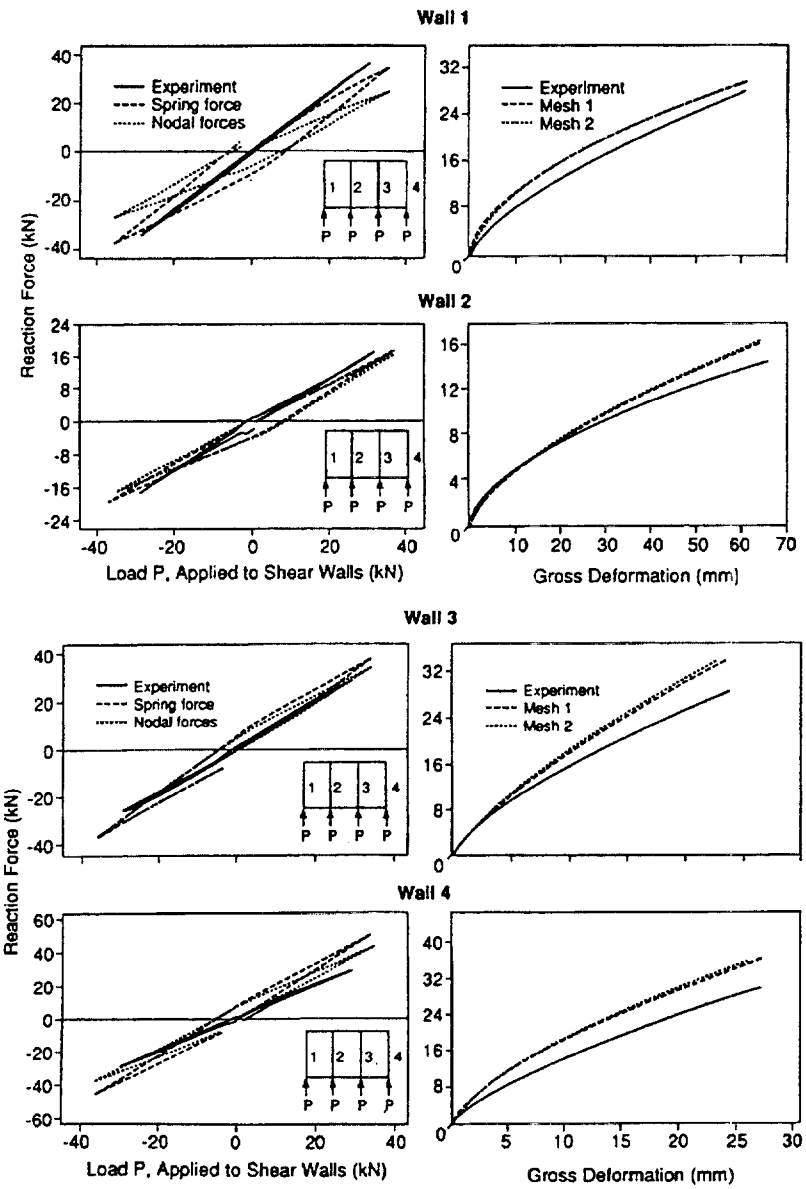


Figure 4.16 Experimental and Analytical Wall Reactions and Deflections
mesh 1: coarse mesh   mesh 2: refined mesh (after Kasal and Leichti, 1992a).

Based on the finite element analyses, Kasal (1992) and Kasal and Leichti (1992b) developed a simplified model for the analysis of wood-frame houses. The model considered the house as a rigid beam, representing the roof diaphragm, resting on elastic supports, representing the shear walls. Reasonable accuracy was obtained using the simplified model compared to the finite element analysis results.

Tarabia (1994) and Tarabia and Itani (1997a) developed a 3-D finite element model for the static and dynamic analyses of wood-frame buildings. In their studies, buildings were idealized as a group of diaphragm elements connected by intercomponent connections. Master degrees-of-freedom were assigned to the connecting nodes among the diaphragm elements, between the diaphragm and intercomponent elements, and to the nodes where lumped masses are added while slave degrees-of-freedom were eliminated by condensation during the analysis. The development of the mathematical model consisted of three phases. In the first phase, the bending or out-of-plane (4-node plate elements for sheathing and grid elements for framing) and the shear or in-plane (sheathing with or without openings, framing, nonlinear fasteners, framing connector, and sheathing interface -gap- elements) stiffnesses of the diaphragms were formulated. In the second phase, the intercomponent connections were modeled as a series of nonlinear (hysteretic) springs connecting the different diaphragms. In the third phase, the structure mass was lumped at the master nodes and the damping matrix was considered to be mass-dependent for dynamic analysis. The hysteresis model for fasteners and intercomponent connections consisted of a skeleton curve representing the monotonic shear resistance and serving as an envelope for the hysteretic cycles as shown in Figure 4.17. The unloading portion of the loops was represented by a straight line and pinching was modeled with a straight line with low stiffness. The accuracy of the different components of the 3-D model was verified using the results of refined finite element analyses and several monotonic, quasi-static, and dynamic tests conducted by Easley et al. (1982) and Dolan (1989).

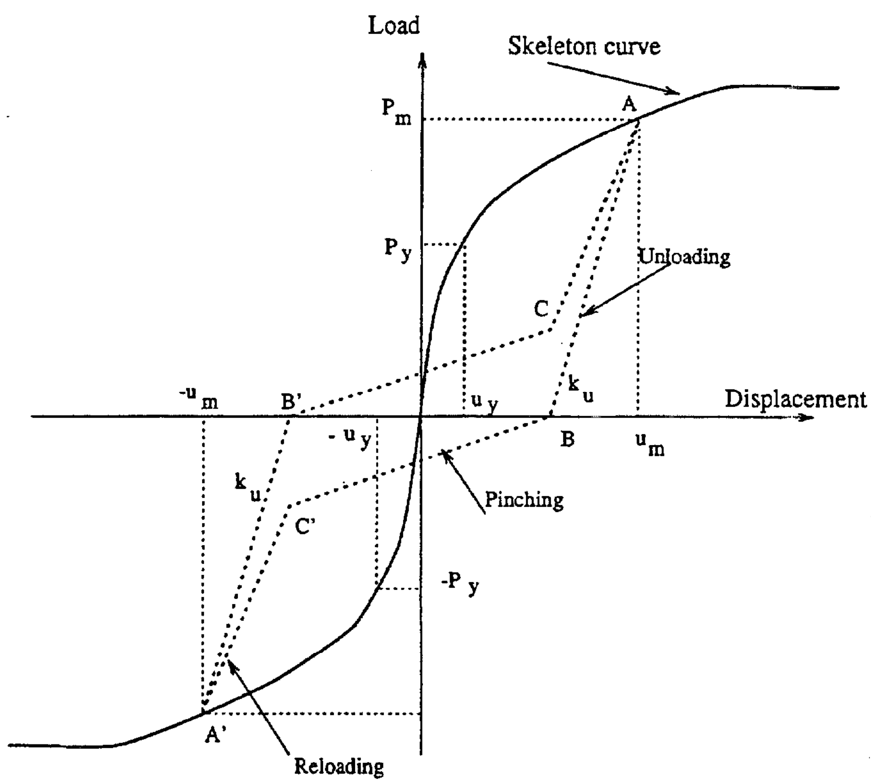


Figure 4.17 Hysteretic Model for Joints and Intercomponent Connections (after Tarabia, 1994).

Tarabia (1994) and Tarabia and Itani (1997b) analyzed different structural configurations using two earthquake accelerograms: Taft record of the 1952 Kern County earthquake and El Centro record of the 1940 Imperial Valley earthquake. The parametric studies examined the effects of the asymmetric in-plane stiffness, dimensions of horizontal diaphragm, presence of partition walls, and ground motion intensity on the overall building response. The analyses emphasized the importance of including sheathing interface elements in the model and the accurate modeling of the hysteretic nail joint behavior. The results of the dynamic analysis indicated that the rigidity of the horizontal diaphragm is inversely proportional to its aspect ratio, and that the use of strong hold-down connections reduces the deformations but increases the forces induced in the shear walls. The studies also showed that partition walls can resist a substantial portion of the seismic forces (depending on their stiffnesses and the aspect ratio of the horizontal diaphragm) and that transverse walls can resist 12 to 17% of the total lateral forces.

4.6 SUMMARY

Significant progress has been made in recent years in the analytical modeling of wood-frame building components, especially shear walls and diaphragms. Several investigators have proposed models that accurately capture the nonlinear behavior of the shear wall diaphragms under monotonic and cyclic lateral loads. Analytical models have also been developed for floors and roofs under uniform pressure. Only limited research, however, has been directed toward the analysis of a complete building or the intercomponent connections under different loading combinations. Table 4.1 summarizes the different finite element studies that have been performed on the various wood-frame components and complete structures.

Table 4.1 Summary of Finite Element Programs for Wood-Frame Housing Components.

Reference	Component	Program Name	Static	Dy-namic	Comments
Polensek (1976)	Shear walls		X		Walls were subjected to flexure and compression
Foschi (1977)	Shear walls		X		
Easley et al. (1982)	Shear walls		X		
Itani and Cheung (1984)	Shear walls	NONSAP*	X		
Itani and Robeldo (1984)	Shear walls	PANFRA	X		
Gutkowski and Castillo (1988)	Shear walls	WANELS	X		Gap elements used to model the discontinuity between sheathing panels
Falk and Itani (1989)	Shear walls		X		
Dolan (1989)	Shear walls	SHWALL DYNWALL	X	 X	Model included gap elements
Kasal (1992)	Shear walls	ANSYS*	X		Detailed 3-D model of the wall subjected to shear, flexure and compression
Kasal (1992)	Shear walls	ANSYS*	X		Equivalent model to be used for analysis of a complete structure
White and Dolan (1995)	Shear walls	WALSEIZ	X	X	
Thompson et al. (1975)	Floors	FEAFLO	X		Linear analysis of floors under flexure
Wheat et al. (1983)	Floors	NONFLO	X		Nonlinear analysis of floors under flexure
Lafave (1990)	Roofs		X		3-D analysis of wood trusses
Polensek and Schimel (1986 and 1988)	Intercomponent connections	COMPCON	X		2-D analysis of the rotational capacity of the wall-to-floor connection
Groom (1992)	Intercomponent connections	ANSYS*	X		Analysis included different intercomponent connections
Kasal (1992)	Complete house	ANSYS*	X		Used for analysis of Philips' (1990) house
Tarabia (1994)	Complete house		X	X	3-D model of a complete house

* Commercial software

Analytical research, similar to that presented in this chapter, has been conducted to investigate the response of manufactured houses to lateral loading. Some of this work may be found in Creighton (1997), Goodman et al. (1996), Jablin (1995), and Jablin and Schmidt (1996).

4.7 RESEARCH NEEDS

Based on the literature review presented in this chapter, further research in the following areas is needed to provide accurate and reliable analytical procedures for wood-frame structures that can be used in the development of performance evaluation criteria as well as guidelines for performance based design:

4.7.1 Refined and Simplified Analytical Procedures

1. *Study the coupling between bending, shear, and axial stiffness of shear wall diaphragms:* In addition to carrying axial loads, walls may be subjected to out-of-plane bending under wind pressure while acting simultaneously as in-plane shear diaphragms. An investigation of the wall behavior under the combined loading condition is needed to accurately model the in-plane and out-of-plane wall behaviors.

2. *Model the hysteretic behavior of nails and intercomponent connections:* When a building is subjected to earthquake motion, nail fasteners and intercomponent connections respond inelastically and dissipate a portion of the excitation energy. Research is needed to accurately model the hysteretic behavior of the joints and connections. The models should include pinching, slip, and stress and stiffness degradation.

3. *Investigate the torsional behavior of wood-frame structures:* Designs of new single family houses often call for unsymmetric plans with long spans, and large openings. Under such designs, structures subjected to lateral loads will experience torsional moments. The analytical procedures should be capable of modeling the torsional stiffness and strength of the building and investigate the influence of torsional moments on the structural response.

4. *Predict post-ultimate load behavior*: The analytical tools should be capable of capturing the influence of the degrading behavior of the nailed joints and connectors on the overall performance of the shear walls, intercomponent connections, and complete houses prior to failure and after reaching the maximum loads.

5. *Model multi-story houses:* As illustrated in Chapters 3 and 4, most experimental and analytical research has been conducted on single-story houses. Additional research is, therefore, required to investigate the behavior of two- and three-story houses under lateral loading. The effort should include detailed experimental and analytical investigations of the behavior of the connections between stories.

6. *Develop simplified analytical methods for a complete building:* While significant progress has been made in the analysis of shear wall diaphragms, additional research is required for the analysis of complete wood-frame structures. The available studies are either too complicated and time consuming or too simplified that their accuracy is

questionable. Further research is needed to develop simple and accurate, static and dynamic procedures to predict the building performance and load distribution among the different components. The simplified procedures should not require excessive time and effort while maintaining accuracy and reliability.

4.7.2 Performance, Measurements, and Standards

1. *Evaluate analysis procedures and design factors:* Current seismic design provisions specify linear or nonlinear, static or dynamic procedures to be used for the seismic design of buildings. Comparisons between the linear static, linear dynamic, and nonlinear static procedures and the more sophisticated nonlinear dynamic procedure are required to assess the reliability of the simplified procedures for analysis and design of wood-frame houses. Furthermore, for the linear procedures, building codes recommend the use of modification factors, that account for energy dissipation through inelastic behavior, to reduce the elastic seismic forces. The validity of the modification factors recommended for wood-frame buildings should be investigated.

2. *Conduct performance and parametric studies:* The parameters that significantly affect the response of a wood-frame house to different loading conditions should be identified. The influence of material properties and fastener/connection behavior as well as building configuration (shape, height, symmetry, and wall openings) and irregularities on the safety and serviceability of wood-frame buildings should be studied.

3. *Predict damage states and failure modes of the building:* The analysis procedure for wood-frame buildings should include the capability of predicting the structural and nonstructural damage state of the building as well as its failure mode(s) based on an expected level of seismic or wind excitation. Damage to structures should be quantified using a damage index. This will greatly benefit home-owners and insurance companies as it provides them with accurate measures of expected earthquake and hurricane losses. Such studies will also be used to develop performance criteria and to relate the design of buildings to their performance; issues that are crucial in the development of performance based design procedures for wood-frame buildings.

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5.0 SUMMARY AND CONCLUSIONS

5.1 GENERAL

Although recent hurricanes and earthquakes in the United States have resulted in few casualties, they have caused substantial damage, significant economic loss, and disruption of social and commercial activities. In many cases, damage to single-family houses has been significant and the performance of such houses has not met the occupants' expectations. To improve the performance of houses in natural disasters and prevent the escalating costs of repairs and rehabilitation, improved design and construction practices are needed. The vast majority of single-family houses in the United States are constructed of wood. Wood is an excellent building material and wood-frame houses, if properly constructed, can resist natural hazards. In addition, other materials such as lightweight concrete, light gage steel, and possibly composites may provide cost effective design alternatives, especially for enhancing disaster resistance.

Performance based seismic design has received considerable attention in the earthquake engineering community in the past few years (SEAOC, 1995; FEMA, 1996 and 1997). The objective of performance based seismic design is to make the performance of structures in earthquakes predictable and provide structures that meet the expectations of the designer and owner with greater economic efficiency. In performance based design, the structure is designed to satisfy life safety requirements (current code philosophy) and to maintain the intended functionality (operational, immediate occupancy, etc.) for different hazard levels. The conceptual framework for performance based design includes: definition of performance and hazard levels, selection of performance objectives, quantification of performance requirements, and development of prescriptive construction provisions. The performance based design concept can also be applied to single-family housing. To formulate performance requirements, identification of hazard levels (wind speed and loads, earthquake ground motion, etc.), quantification of damage in past natural disasters, as well as determination of design parameters such as interstory drift, inelastic deformation, energy dissipation, and strength requirements are needed. To develop performance and acceptance criteria for single-family houses and to be able to assess the feasibility of using alternative construction materials, a better understanding of the 1) performance of houses in extreme winds, hurricanes, and earthquakes, and 2) behavior of housing components and subassemblages to lateral loads simulating such events is needed. This report discusses the performance of single-family, wood-frame houses in past earthquakes and hurricanes (Chapter 2). It also summarizes the experimental (Chapter 3) and analytical (Chapter 4) studies of full scale houses as well as housing components and subassemblages. While each chapter includes conclusions and recommendations for further research, the major points are summarized below:

1. The performance of single-family houses in recent U. S. hurricanes and earthquakes indicates that wood-frame houses generally met the life safety objective. Many, however, experienced structural and nonstructural damage which prevented immediate occupancy and/or the intended operational functions, and resulted in expensive repairs. The majority

of damage to wood-frame construction resulted from the failure of connections between walls, walls and roof, and walls and foundation. While the knowledge exists to build single-family houses that can resist extreme winds and earthquakes, damage still occurs. The damage from these events, however, can be reduced with improved design, good construction practices, and quality inspection especially if immediate occupancy is required after a hurricane or an earthquake. Consequently, design procedures and alternative construction materials that are cost effective and enhance the resistance of houses to natural hazards are needed.

2. In order to develop performance criteria for single-family houses, performance of houses in past earthquakes and hurricanes needs to be reviewed and damage assessment data standardized to allow damage classification and development of damage indices. Measurements of earthquake ground motion and wind speed data as well as instrumentation of selected single-family houses (accelerometers and pressure sensors at different locations in the house) are needed to determine the loads, verify procedures for modeling of houses, and study their behavior. Although instrumenting houses and collecting data are expensive, the information will be invaluable in verifying analytical models, and assessing and developing performance requirements.

3. Only a few full scale single-story, wood frame houses have been tested under lateral loads to determine the distribution of loads to interior and exterior shear walls, and the applicability of extending drift criteria for medium- to high-rise buildings to low-rise buildings and single-family houses. Several investigators have tested shear walls, the majority without openings. Most of these walls were subjected to static monotonic loading with only a few studies involving cyclic loading. Most of the shear wall tests, as well as tests of full scale houses, were not carried out to failure. Testing of single- and multi-story shear walls with openings, different aspect ratios, and under monotonic and cyclic loading to failure are needed to obtain a better understanding of their behavior and performance characteristics. Component testing of intercomponent connections, such as sill anchorage, hold-downs, and shear connectors, are also needed since connection failures are the major causes of damage in houses during earthquakes and extreme winds.

4. Significant progress in analytical modeling of wood-frame building components, especially shear walls and diaphragms has been made in recent years. Only limited analytical studies, however, have been directed toward complete houses with intercomponent connections under different loading combinations. Further studies of complete houses with the modeling of hysteretic behavior of joints, intercomponent connections, and torsional behavior are needed to assess the performance of houses under different loading combinations and to develop performance baselines for comparing houses constructed with non-traditional vs. traditional materials. Also needed for the development of performance criteria is improved analytical models to predict structural and non-structural damage states and failure modes for houses under a given seismic or wind excitation.

5.2 RECOMMENDED RESEARCH

Based on the review of the performance of single-family, wood-frame houses in past earthquakes and hurricanes, and the review of experimental and analytical studies relevant to analysis, design, and behavior of full scale houses and housing subassemblages, the following plan is proposed to accomplish the stated objective for the NIST project.

Using refined three-dimensional finite element models, structural performance criteria for complete housing units will be developed. The analytical model will include the interaction of various components such as shear walls, floors, and roof as well as intercomponent connections. The model should be capable of predicting structural damage states of houses as well as their failure mode(s) for earthquake or extreme wind loads. Once the performance criteria for typical houses with traditional construction materials are established, they will serve as a baseline for examining the performance of single and multi-story houses constructed with non-traditional materials.

The study will further examine the data on the performance and behavior of housing components built with traditional and non-traditional materials, and identify areas where further information is needed for the finite element modeling. Component testing will then be conducted. Among the areas that require further information are single- and multi-story shear walls with openings and different aspect ratios, and intercomponent connections under monotonic and cyclic loading. Since few shear walls have been tested to failure, the experiments will be extended to failure to determine failure modes and damage states. The results from these experiments will be used to refine the finite element model.

A chart describing various tasks that should be completed to formulate performance criteria for single-family housing constructed with traditional or non-traditional materials is presented in Figure 5.1.

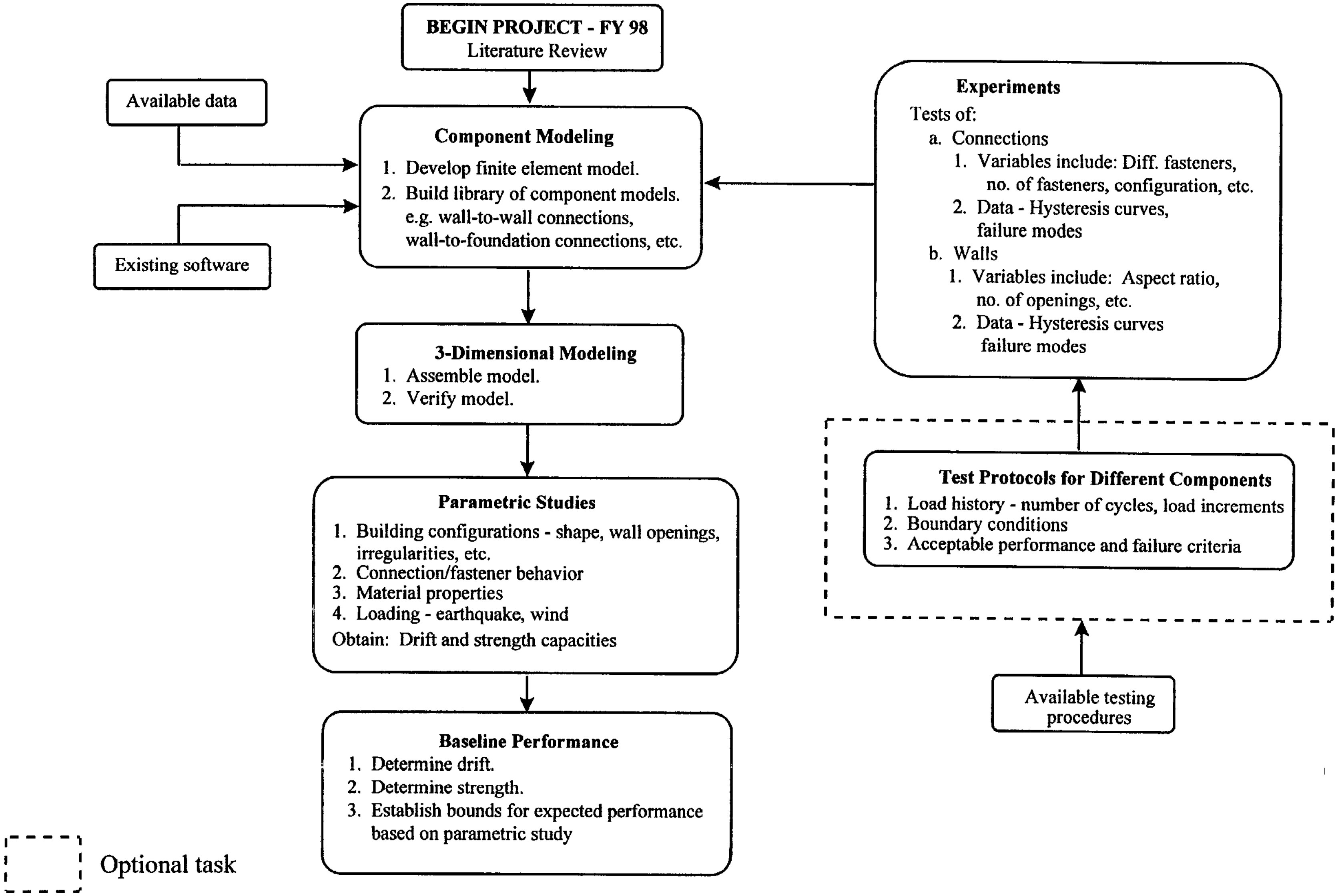

Figure 5.1 NIST's Research Plan for Structural Performance of Single-Family Housing

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