State of the Art: Seismic Behavior of Wood-Frame Residential Structures

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Abstract: There are about 80 million single-family dwellings (SFDs) in the United States, predominantly of wood-frame construction. Of these, 68% are owner-occupied. A home is typically the largest single investment of a family, and is often not covered by earthquake insurance, even where it is available. Of all the houses in America, 50% were built before 1974, and 76% built before 1990. Most wood-frame SFDs (WFSFDs) were built to prescriptive code provisions before seismic requirements were introduced. After the introduction of seismic design requirements, the importance of examining structures as an assembly of connected elements became more common. Much of the seismic design information on SFD construction is based on educated opinion or limited research. This review examines research that can be applied to WFSFD seismic analysis and the design and retrofit of existing WFSFDs. The review is intended to cover most readily available papers published in major U.S. journals and at major conferences in the area of seismic modeling, testing and evaluation. The state of the art is reviewed of seismic experimentation and seismic evaluation, and observations and recommendations are provided for future research. **DOI: 10.1061/(ASCE)ST.1943-541X.0000861.** *© 2013 American Society of Civil Engineers.*

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Introduction

Most of the building structures in the United States and Canada are single-family, residential dwellings (Ni et al. 2010). Like other structures, these buildings vary in size, configuration, age, and condition. These factors affect their use as habitable dwellings and also their performance in natural disasters. Predecessors of the International Building Code (IBC) (International Code Council 2011a) and the International Residential Code (IRC) (International Code Council 2011b) have addressed seismic issues in past editions. However, according to the American Housing Survey of the U.S. Census Bureau (2011), most single-family dwellings (SFDs) were constructed before these seismic provisions were introduced during the 1970s and 1980s. More than half of the existing housing inventory was constructed before 1974. Seventy-five percent of the inventory was constructed before 1990. About 70% of the housing inventory are individual dwellings. People spend more than 1/3 of their lives in these structures, usually asleep and not prepared to react to a potential disaster. The seismic performance of wood-frame, single-family dwellings (WFSFDs) is of vital importance to many of us.

The Association of Bay Area Governments commissioned a study (Perkins et al. 1996) which indicates that reasonably expected major earthquakes of approximately magnitude 7 could result in over 150,000 uninhabitable housing units and 360,000 people made homeless as a result. The 1994 Northridge Earthquake Buildings Case Studies Project (Holmes et al. 1996) examined a two-story house with moderate damage and concluded, "hundreds of thousands of existing houses similar to this case study are located in areas that can expect . . . similar or greater levels of damage, p. 389." During the 1994 Northridge Earthquake, a 6.7 magnitude event, 20 lives were lost in wood-platform-framed buildings according to Rainer and Karacabeyli (2000). Though the loss of life has been limited in WFSFDs, Holmes et al. (1996) also concluded, "Life Safety Performance as a minimum code requirement does not meet the expectations of those investing in housing (e.g., owners and lenders)." Therefore, improvements in performance (reduction in damage), in addition to improved life-safety, were primary motivations for conducting this study.

Extensive literature reviews were conducted by Carney (1975) and Peterson (1983) on wood diaphragm testing and design and by van de Lindt (2004) on shear-wall testing, modeling and reliability analysis. These collections will not be repeated here. This review will include more recent research in those areas and extend those reviews to cover whole-building testing, finite element analysis of these structures, research on post-frame roof diaphragms, and earthquake damage analysis and estimation methods. It is intended to cover most readily available papers published in major U.S. journals and presented at major conferences. This review will also discuss the state of the art in these areas, the general progress of research to date and offer an opinion on where additional research is needed.

This paper is limited to conventional construction materials and methods that are used within the United States and Canada. Conventional materials include sawn dimensional lumber, structural sheathing, and metal-plate-connected wood trusses, which are assembled with nails, screws, adhesive and proprietary sheet metal connectors. Typical systems in these areas use modular construction with typical floor-to-floor heights generally of 2.4 m (8 ft), walls made of studs 38 × 89 mm (nominal 2 × 4 in.) or 38 × 140 mm (nominal 2 × 6 in.), floors of joists 38 mm (nominal 2 in.) in width, all with a spacing of 400 mm (16 in.) or 600 mm (24 in.),

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and a roof composed of plate-connected engineered trusses or rafters 38 mm (nominal 2 in.) in width, spaced 600 mm (24 in.). There are methods of construction that are common in Africa, Asia, Australia, Europe, and South America that are not frequently used in the United States and Canada. Without discussion of the relative merits of these systems, we have concentrated on these traditional methods used in the United States and Canada.

Unique systems have been and are being developed in the United States and Canada which involve special materials or component mechanics, such as viscoelastomeric damping materials in shear walls, installing plywood in the center of the shearwall plane (mid-ply walls), seismic dampers, and similar nonconventional systems. Some of these systems are proposed, but not in regular use at present. Most of these systems remain experimental and are excluded from this review.

The present state of seismic research is presented in the following sections. The research is summarized in tabular form for brevity. The focus of the presentation is recent research, and the objectives in writing this paper are to:

1. Review the current state of the art with respect to WFSFD design and research for conventional materials and methods of construction;
2. Locate research areas where potential for additional research or improvement exists;
3. Compare, contrast, and synthesize conclusions between different areas of research; and
4. Provide conclusions and recommendations for future research in the seismic behavior of WFSFDs.

Progression of Research

Structural systems of various materials were developed using concepts of beams, columns, shear walls, and diaphragms. Though most materials use these concepts, not all systems are truly composed of individual components. Systems such as concrete or masonry often have continuity through joints and are cast on-site instead of being assembled from pieces shipped to the construction site. Conversely, steel, wood, precast concrete, and cold-formed steel are assembled from individual components fabricated in a factory and assembled at the construction site.

Although this general progression has been followed in all materials, wood construction has unique characteristics that do not affect the other construction materials. Concerns about the applicability of scaling factors to wood, a natural product with a cellular makeup, have led to efforts to test full-scale models. Though wood members can be trimmed to a scaled size, the fundamental fiber or cell size does not change, so scaling of some wood test models may not be valid (van de Lindt 2007). Wood is anisotropic due to this cellular composition, and thus it has different strength and stiffness properties in each orthogonal direction (Breyer et al. 2007). Wood horizontal diaphragms have been traditionally considered *flexible* rather than *rigid* (Breyer et al. 2007). The IBC (International Code Council 2011a) defines a flexible horizontal diaphragm as one which has more than twice as much mid-span deflection as the supporting story drift, and also includes an assumption that noncantilevered horizontal diaphragms with no more than 38 mm (1.5 in.) of concrete are flexible. A provision of ASCE 7 (2010) indicates, "... wood structural panels are permitted to be idealized as flexible if any of the following conditions exist ...". There is a growing list of peer-reviewed journal papers indicating that wood diaphragms often behave rigidly, rather than flexibly (Breyer et al. 2007). The authors are now seeing research that questions the assumption by which so many WFSFDs have been designed (Christovasilis and Filiatrault 2010; Phillips et al. 1993; Skaggs and Martin 2004). These distinctions lead to concerns that research into other materials and methods of construction or design may not fully apply to wood construction.

Quality control in the design and construction of WFSFDs is sometimes difficult to achieve. Traditionally, a WFSFD can be designed by unlicensed individuals unless the WFSFD is of significant size or complexity. If a structural engineer (SE) is involved, it is common for the SE to design only specific components (door or window headers, for example) and mark their design items on the architect's or designer's plans. This makes it hard for the construction team to know which component is engineered and which component is based on the prescriptive code. Field changes are occasionally made by the construction team without consultation with the designer. The relative ease with which wood members can be reshaped makes field changes more likely than with steel, for example. Basic elements [studs, plywood, and oriented-strand board (OSB)] are assembled into walls, floors, and roofs, and then stacked to assemble the structure. Thus, an SE may not be involved in the design of a specific WFSFD, and even if an SE is involved, the SE may not have designed the lateral force resisting system (LFRS) or the structure as a whole. Occasionally, significant components are omitted, either by faulty design or construction. Falk and Itani (1988) and Graf and Seligson (2011) recommend increased quality control from design through final inspection, engineered design of all new WFSFDs, engineering evaluation of older structures, and certain mandated upgrades. A survey of architects and engineers in California indicated that significant omissions of key seismic-resisting elements were missing in more than 40% of the buildings surveyed (Schierle 1996).

As late as the 1970s, WFSFDs were considered to be very safe in earthquakes (Li and Ellingwood 2007; Skaggs and Martin 2004). Traditional WFSFDs built before the 1970s were often regular in shape, usually one story in height, with a roof structure that was continuous throughout the structure. Though these simple dwellings suffered some types of damage during earthquakes, there was little loss of life. Changes in architectural style resulted in WFSFDs of more recent construction that have multiple stories, segmented roofs at different levels, and few long runs of shear walls. These more recent dwellings suffered more damage, which became noticeable in the 1971 San Fernando earthquake [moment magnitude (MM) 6.6], the 1989 Loma Prieta earthquake (MM 6.9), and very notably the 1994 Northridge earthquake (MM 6.7) (Graf and Seligson 2011). Minimum requirements for seismic connections between components began in the 1980s versions of the Uniform Building Code (UBC) (Breyer et al. 2007).

Research into WFSFDs began with individual elements and progressed to horizontal and vertical diaphragms and finally to assembled structures. The general progression of research is shown in Fig. 1. (Delineations between types of design or analysis are approximate.)

The damage and loss of life in wood-frame structures during the 1994 Northridge earthquake led to several major wood-frame research projects. These Megaprojects are discussed individually in the following two sections.

CUREE-Caltech Woodframe Project

The Consortium of Universities for Research in Earthquake Engineering (CUREE) worked with the California Institute of Technology (CalTech) to study earthquake hazard mitigation in wood-frame structures (CUREE 2002). This project was announced in 1998 as a $12.1 million, three-year study, funded by the Federal Emergency Management Agency (FEMA) and the

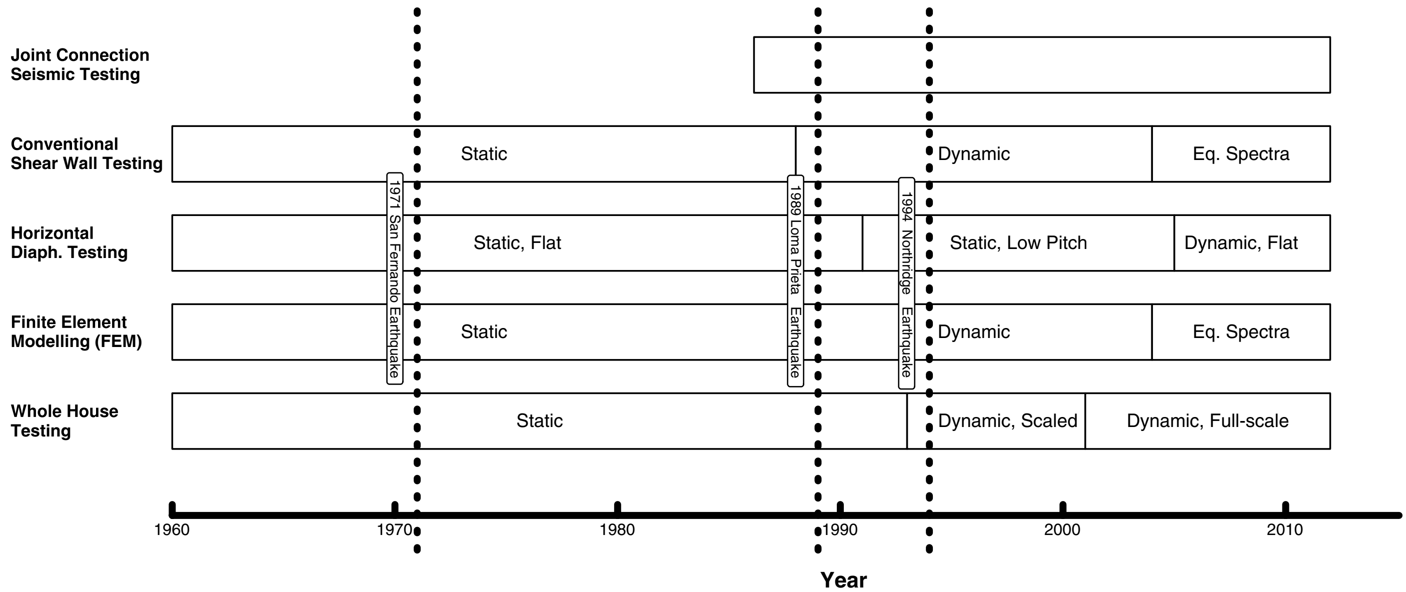

Fig. 1. Progression of wood-frame dwelling research and methods

California Governor's Office of Emergency Services. This project became known as the CUREE-CalTech Woodframe Project, and was charged with advancing the state-of-the-art in wood-frame analysis and design for seismic hazards. Under this project approximately 30 reports were produced, comprised of five elements: testing and analysis; field investigation; proposed revisions to the building codes; economic analysis and education and outreach. This was a coordinated program involving many universities, researchers and research efforts. Summaries of the results of those relating to this paper appear in Tables 1–3, where noted. Discussion of specific portions of the project will appear in the following sections: "Shearwall Testing and Analysis," "Wood-Frame Dwelling Testing," and "Earthquake Damage Surveys."

The CUREE-CalTech project was among the first to dynamically test full-scale dwellings and to perform analysis of those results by different methods for comparison. Though it produced answers to many of the questions of the day, it also provided direction for further research in areas that it could not answer within the project timeframe (Cobeen et al. 2004a, b). It further developed and standardized dynamic testing methods intended to better evaluate wood-structure performance.

NEESWood Project

The NEESWood project (Network for Earthquake Engineering Simulation—Wood) began in 2005 and was funded by the National Science Foundation (NSF) with a $1.2 million grant, as a multiyear project to study how wood-frame structures respond to seismic forces. NEESWood continued the work begun by the CUREE-Caltech project by performing and analyzing a series of experiments based on the CUREE prototype buildings (van de Lindt et al. 2006a, b), which were designed to provide a basis for experimentation and analysis that all the CUREE-CalTech researchers could use. NEESWood experiments included shake-table testing of two-story townhouses and culminated with shake-table testing of a six-story wood building through a 7.5 magnitude earthquake. Papers based on these tests appear in Table 2 and are discussed under "Wood-Frame Dwelling Testing" below. The performance of the six-story building has indicated that large residential buildings can be successfully designed to withstand expected seismic activity in any region. Additional analyses of the test results are being performed and further papers will be appearing, so the results of this project are not yet complete. Though much larger than typical houses, the experiments with this structure help to evaluate some of the wood structural systems and elements that are also used in houses.

Shearwall Testing and Analysis

Conventional shear walls are constructed of 38 × 89 mm (nominal 2 × 4 in.) or 38 × 140 mm (nominal 2 × 6 in.) studs with a structural sheathing consisting of plywood, OSB, Portland cement plaster, or other approved material. Research efforts on the static and dynamic properties of conventional structural panel shear walls are numerous. Table 1 provides a chronological list with a focus on the most recent 10 years of research for brevity. Table 1 also includes some notable previous studies that investigate brittle finishes or nonwood materials. The most common nonwood materials are gypsum wallboard panels and Portland cement plaster on lath or expanded metal mesh. The reader is also referred to an excellent review of shearwall research by van de Lindt (2004).

Early studies tended to be static loading tests based on ASTM E-72 (ASTM 2005) and more recently ASTM E-564 (ASTM 2006). The studies that followed 1990 often used cyclic loading protocols proposed by the Structural Engineers Association of Southern California (SEAOSC) and the CUREE protocols developed for the CUREE-CalTech Woodframe Project. Table 4 describes the characteristics of seismic loading protocols known as ASTM E-546 (ASTM 2006), ISO-16670 (ISO 1998), sequential phased displacement (SPD) (SEA 1996; Dinehart and Shenton 1998), FCC-Forintek (Karacabeyli and Ceccotti 1996), and CUREE-CalTech/CUREE-CalTech Near Fault (Krawinkler et al. 2001). Figs. 2(a–f) referenced in Table 4 show the shape of each of these loading protocols.

Cyclic testing has shown that concerns about the nonlinear performance of brittle or nonductile materials (mainly gypsum and cement plaster) are warranted (Falk and Itani 1987; Hart et al. 2008; Seaders 2004, Seaders et al. 2009a). Various studies indicate permissible elastic drift limits for brittle materials, but limited research has been conducted to determine whether these materials can be kept within elastic limits for WFSFDs design, or whether

Table 1. Conventional Wood Shearwall Testing and Analysis

Reference	Method/loading	Focus of research
Oliva and Wolfe (1988); Oliva (1990)	ASTM E564. monotonic, static cycles at 1 Hz, dynamic at 5 Hz	Tested 59 gypsum SW for racking resistance. 2.4 m (8 ft) long walls confirmed codes, but longer walls, and horizontal sheets were better. Gluing increased
Thurston and King (1994)	Racking resistance	Ten SW, varying wall returns, openings and materials w/o hold-down
Seible et al. (1999)	Analytical study	CUREE workshop on testing, analysis and design
Karacabeyli et al. (1999)	Static and dynamic	Compares static and dynamic SW test results
Merrick (1999)	cyclic, nonincreasing	7 tests of plywood, OSB, gypsum wallboard SW to evaluate energy dissipation
Salenikovich and Dolan (2000, 2003a, b)	Monotonic, cyclic at 0.25 Hz (ISO 1998)	Investigates the strength of anchored SW, 2.4 m (8 ft) tall, 4:1, 2:1, 1:1, 2:3 aspect ratios
NAHB Research Center (2001)	Static	Strength and deflection of SW with corners and openings
Gatto and Uang (2002, 2003); Uang and Gatto (2003)	Dynamic and cyclic	Standard construction 2.4 × 2.4 m (8 × 8 ft) wood-frame shear walls were tested using: monotonic, CUREE-Caltech standard (CUREE), CUREE-CalTech near-fault, SPD, and International Standards Organization test protocols
McMullin and Merrick (2002)	Cyclic	Six shear walls of grade CD plywood, OSB and gypsum wallboard, includes tests of different types of drywall screws
Kim (2003); Rosowsky and Kim (2004a, b); Kim and Rosowsky (2005a, b)	Reliability analysis	Develops fragility curves for various SW materials
Langlois et al. (2004)	Static, cyclic	Applied monotonic (ASTM E564) and cyclic (CUREE) testing protocols to SW
Ni and Karacabeyli (2004)	Analytical study	Presents equations for evaluating deflection of unblocked SW and horizontal diaphragms
van de Lindt et al. (2004); van de Lindt and Rosowsky (2004)	Reliability analysis	Tested 12 SW designed and evaluated for reliability with ASCE 16
Seaders et al. (2004, 2009a, b)	Monotonic (ASTM E564), cyclic and earthquake loads	Two sets of tests of eight partially and two fully anchored 2.4 × 2.4 m (8 × 8 ft) SW with 38 × 89 mm (nominal 2 × 4 in.) DF studs at 610 mm (24 in.) o.c. 2 OSB with 8d nails and GWB
van de Lindt (2004)	Literature review	Details 31 SW tests, modeling and reliability analysis
Williamson and Yeh (2004)	SPD (SEAOSC, FME = 3 cm)	SW w/openings (portal frames)
Dean and Shenton (2005)	ASTM E564 modified to exceed design allowable before the final half-cycle	Ten 2.4 × 2.4 m (8 × 8 ft) SW with 11 mm (7/16 in.) OSB and applied vertical load
Lebeda et al. (2005)	Static, cyclic	Thirteen 2.4 × 2.4 m (8 × 8 ft) SW with misplaced hold-downs (CUREE)
White (2005); White et al. (2009, 2010)	Earthquake records	Tested 34 identical 2.4 × 2.4 m (8 × 8 ft) walls of 38 × 89 mm (nominal 2 × 4 in.) kiln-dried DF. Studs were spaced at 610 mm (24 in.) o.c. Half were partially anchored, half fully anchored
Johnston et al. (2006)	Cyclic	Compares effects of vertical load and hold-down placement
Seaders et al. (2004); White (2005); van de Lindt and Gupta (2006); White et al. (2009)	Three SAC response spectra	2.4 × 2.4 m (8 × 8 ft.) SW with 11.1 mm (7/16 in.) OSB and 12.5 mm (1/2 in.) gypsum panels
Leichti et al. (2006)	CUREE	Tested SW with different nail strengths
Mi et al. (2004, 2006)	Monotonic and ASTM E2126	Eight 4.9 × 4.9 m (16 × 16 ft.) SW with 12.5 mm (1/2 in.) plywood
Winkel (2006); Winkel and Smith (2010)	Static	14 tests of shear walls with combined racking, uplift and bending loads
Yasumura et al. (2006)	1940 El Centro	Two-story 3 × 3 × 6 m (9 × 9 × 18 ft.) 7.5 mm (5/16 in.) plywood with openings
McMullin and Merrick (2007)	Monotonic and CUREE-CalTech	11 tests. Discusses seismic damage thresholds for gypsum wallboard
Ni and Karacabeyli (2007)	ISO 16670, ASTM 2126	16 SW w/ diagonal or transverse horizontal lumber sheathing and gypsum sheathing varying hold-downs, vertical load, and width of sheathing
van de Lindt (2008)	Shake-table tests	24 shake-table tests of SW, some w/gypsum, some w/ corner walls
Hart et al. (2008)	Cyclic, varying by author	Discusses 195 drywall and stucco sheathing tests done by APA, Merrick, City of Los Angeles and McMullen and Pardoen for CUREE
McMullin and Merrick (2008)	Cyclic CUREE-Caltech	17 tests w/ screws and nails w and w/o window openings
Sinha (2007); Sinha and Gupta (2009)	Monotonically (ASTM E564)	Tested 16 standard 2.4 × 2.4 m (8 × 8 ft) walls, 11 were sheathed with OSB on one side and GWB on the other, and 5 walls were tested without GWB. Digital image correlation was used for data acquisition and analysis which is a full-field, noncontact technique for measurement of displacements and strains
Zisi (2009)	Monotonic and cyclic w/increasing amplitude	Tested brick veneer on wood-frame walls with OSB and gypsum
Ni et al. (2010)	Monotonic and cyclic (ISO 16670)	Tested 20 configurations of 1.22, 2.44, or 4.88 m long SW with 9.5 mm OSB or 12.7 mm GWB, some 4.88 m SW with a 2.44 opening, some 2.44 m walls with 1.22 or 0.61 m perpendicular bracing walls

Table 1. (*Continued.*)

Reference	Method/loading	Focus of research
Goodall and Gupta (2011); Goodall (2010)	Monotonically (ASTM E564)	Tested 14 shear walls, 2 of each of 7 different designs. Six walls had 1105 × 610 mm window openings, eight did not. All walls were 2.4 × 2.4 m (8 × 8 ft) and built from 38 × 89 mm (nominal 2 × 4 in.) DF studs at 610 mm o.c. Tests stopped at deflections of 4.0, 8.0, 12.0, 16.0, 20.0, 24.4, 48.8, and 73.2 mm (5/32, 5/16. 5/8, 3/4 , 1, or 2 in.) to record damage

Note: DF = Douglas Fir; FEM = Finite Element Model; FME = First Major Event, defined as an event sufficient to bring the structure to the yield point; GWB = Gypsum Wallboard; SAC = A joint venture of the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and CUREE; SEAOSC = Structural Engineers Association of Southern California; SPD = Sequential Phased Displacement; SW = Shear Wall.

Table 2. Definition of Shearwall Testing Methodologies

Protocol type	Standard or common name	Classification	Description	Figure	Reference
Static	Monotonic (ASTM E564)	Linear increasing	Ramps load to incremental limits	Fig. 2(a)	ASTM (2006)
Cyclic	ISO-16670	Full cycle reversing	Initial increasing sequence, then three cycles at each increasing displacement level	Fig. 2(b)	ISO (1998)
	SPD		SPD	Fig. 2(c)	SEA (1996); Dinehart and Shenton (1998)
	FCC-Forintek		Similar to SPD	Fig. 2(d)	Karacabeyli and Ceccotti (1996)
	CUREE		CUREE-CalTech	Fig. 2(e)	Krawinkler et al. (2001)
	NF		CUREE-CalTech Near Fault	Fig. 2(f)	Krawinkler et al. (2001)
Dynamic			Usually a set of scaled historic seismic records		

Note: NF = Near field.

Table 3. Horizontal Wood Diaphragm Testing and Analysis

Reference	Methods	Focus of research
Countryman (1952)	Static, dynamic	Plywood diaphragms, mostly blocked. Tested six quarter-scale models [1.5 × 3.0 m (5 × 10 ft)] and four full-scale models 3.7 m or 6.1 × 12.2 m (12 or 20 × 40 ft). Dynamic load was static load increased by 1/3 and cycled five times
Countryman and Colbenson (1954)	Static, dynamic	Plywood diaphragms, about half blocked. Tested 15 full-scale models 7.3 × 7.3 m (24 × 24 ft). Dynamic load was static load increased by 1/3 and cycled six times, then single cycles of increasing amplitude to failure
Johnson (1955)	Static	Tested 3.7 × 18.3 m (12 × 60 ft) plywood diaphragm
Johnson and Burrows (1956)	Static	Gable roofs were tested and found stronger than flat diaphragms w/ no boundary reinforcing and weaker than flat diaphragms w/ boundary reinforcing
Tissell (1967)	Static	Tested 18 diaphragms 4.9 × 14.63 m (16 × 48 ft)
Carney (1971)	Analytical study	Presents development of the general theory of folded plates as it applies to plywood roof diaphragms
Johnson (1971)	—	Tested 6.1 × 18.3 m (20 × 60 ft) roof section sheathed w/ plywood overlaid on decking
Falk et al. (1984)	SOTA	Reviews literature on low-rise wood diaphragms. Concludes more research is needed on roofs and dynamic behavior
Walker and Gonano (1984)	—	Tested gypsum and asbestos cement ceiling panels. Determined independent panels should be modeled for shear, solidly connected panels for flexure
Falk and Itani (1988)	Analytical study	Compares deflection model to previous flat diaphragm tests
Kamiya (1988)	Pseudodynamic tests	Simple hysteretic loop model
Mahaney and Kehoe (1988)	Analytical study	Traditional tributary area methods and rigid diaphragm methods may be unconservative. Presents a generalized linear shear stiffness method for plywood diaphragms to distribute the shear between lateral resisting elements
Falk et al. (1989)	SOTA	Reviews literature on low-rise wood diaphragm modeling
Falk and Itani (1989)	Analytical study	Compares finite-element model to previous flat diaphragm tests
Alsmarker (1991)	Static load tests of 3 flat diaphragm panels. Load parallel to ridge/eaves	Gypsum fasteners ultimately fail at far above allowable load. Design for elastic fastener failure
Tissell and Rose (1993)	Static load tests of 5 low pitch trussed diaphragms	Used 8 mm (5/16 in.) plywood on 50 × 50 mm (2 × 2 in.) trusses 406 mm (16 in.) o.c. Maximum slope was 2:12. No ceiling material was used in the tests. Includes some MPCT

Table 3. (*Continued*.)

Reference	Methods	Focus of research
Foliente (1994, 1995)	Analytical study	Hysteresis model includes nonlinearity, strength and stiffness degradation, pinching and historical loading
Tarabia and Itani (1997)	Analytical study	Nonlinear 3-D FEM. Concludes diaphragm rigidity is a significant factor in determining loads on building elements
Yancey et al. (1998)	SOTA	NIST review of the state of residential design research
Carradine et al. (2004b); Dolan et al. (2003)	Monotonically increasing, cyclic CUREE	Presents research on the deflection of horizontal diaphragms
Collins et al. (2005)	Analytical study	Develops 3-D finite element model for a house
Bott (2005)	Dynamic-elastic load test of 6 flat diaphragm panels. Load perpendicular to ridge/eaves	Shear stiffness increased by: foam adhesive/blocking 259%, blocking 135%, foam adhesive 89%; relative to unblocked diaphragm
He and Li (2012)	Analytical study	FEM of nine flat horizontal diaphragms

Table 4. Finite Element and Analytic Models of Wood-Frame Dwellings

Reference	Analysis method	Focus of research
Falk and Itani (1988)	2-D nonlinear FEM	Nonlinear elements model the connections between the fasteners, sheathing and framing members
Kamiya (1988)	Simple hysteretic loop model	Pseudo-dynamic tests
Kataoka and Asano (1988)	Nonlinear stiffness model	Compared model with tests for a two-story Japanese post and beam structure
Mahaney and Kehoe (1988)	Linear	Traditional tributary area methods and rigid diaphragm methods may be unconservative. Presents a generalized shear stiffness method for plywood diaphragms to distribute the shear between lateral resisting elements
Moss and Carr (1988)	—	New Zealand building code. Tested timber portal frames and excluded shear walls. Seismic response
Kasal and Leichti (1992)	Program *ANSYS* nonlinear FEM	Wood stud wall with openings
Foliente (1994, 1995)	Nonlinear	Hysteresis model includes nonlinearity, strength and stiffness degradation, pinching and historical loading
Kasal et al. (1994)	Nonlinear	Model of one-story house tested by Phillips (1990)
Kasal et al. (1999)	Nonlinear	Hybrid dynamic model including hysteretic and stochastic methods
He et al. (2001)	Program *Lightframe3D* nonlinear FEM	Presents FEM which includes individual nail connections
Masaki and Kenji (2002); Kenji et al. (2002)	Nonlinear FEM	Dynamic model of Japanese house demonstrates 45% increase in loads due to eccentricity
Lam et al. (2002)	Nonlinear FEM	Dynamic model of individual nail connections in the diaphragm system. Verified with a simple box structure
Symans et al. (2004)	Nonlinear FEM	Modeled behavior of a house using viscous dampers
Collins et al. (2005)	Program *ANSYS* nonlinear FEM	Modeled hysteretic behavior of a house
Li (2005)	Program *CASHEW* nonlinear FEM	Used to develop fragility information for light frame shear walls
Winkel (2006)	Nonlinear FEM	FEM using uncoupled spring model for sheathing-framing and framing-framing nail connections is compared to test data
Xu (2006)	Program *ABAQUS* nonlinear FEM	General hysteretic model, BWBN, was modified for nailed joints, embedded in ABAQUS and compared with the test data
Li and Ellingwood (2007)	Programs *CASHEW*, *OpenSees*	Models of three typical shearwall types demonstrate applicability of this technique to general WFSFD structures. Concludes that this method can predict WFSFD response and assist in evaluating retrofit methods
Blasetti et al. (2008)	Program *ANSYS* nonlinear FEM	Modeled hysteretic behavior of shear walls
Osteraas et al. (2008)	Nonlinear FEM	Uses programs *SAWS* and *SAPWood* with laboratory test data (COLA, CUREE-CalTech, CUREE-EDA) compared with documented damage of two buildings due to the Northridge Earthquake
Pei and van de Lindt (2009)	Program *SAPWood*	Model using Bayesian predictive distribution fragilities to simulate damage and repair cost. Applied to one story ranch and two-story houses, concluding the method provides reasonable results
Pang et al. (2009)	Programs *CASHEW*, *SAWS*	Fragility analysis of six buildings of two foundation types with OSB and gypsum sheathing in Central US. Concludes that one-story WFSFDs have good life safety response but can have significant financial loss, two-story WFSFDs may need additional nailing and hold-downs
Black et al. (2010)	Programs *SAPWood*, *MATLAB*	Empirical seismic loss model applied to a two-story WFSFD. Concludes loss analysis can help evaluate loss, help define performance objectives and guide objective WFSFD design
Christovasilis and Filiatrault (2010); Christovasilis (2011)	Nonlinear FEM	A 2-D FEM with rigid floors including explicit connection elements

Table 4. (*Continued.*)

Reference	Analysis method	Focus of research
Li et al. (2010)	Programs *CASHEW*, *SAWS*	Compares collapse probabilities of WFSFDs in Western United States with Central and Eastern United States and concludes existing ASCE 7 seismic maps do not result in uniform risk
Pei and van de Lindt (2010)	Programs *SAPWood*, *Nail Pattern*	Develop fragility curves based on differing possible construction quality and relating the damage to economic loss. Concludes that retrofits are of limited use in either large or small earthquakes and construction quality has major impacts
Yin and Li (2010)	Programs *CASHEW*, *SAWS*	Examines collapse risk due to uncertainties in ground motion and in shear wall resistance in a Monte Carlo simulation to a single story. Concludes these uncertainties result in significant variation in outcome
Goda et al. (2011)	Program *SAWS*	Examined 1,415 houses in Richmond, BC using seismic hazard model of Geological Survey of Canada. Estimates sensitivity of analysis to differing assumptions of hazard models, spatial correlation model, uncertainty in ultimate seismic capacity and spectral shape
Pei and van de Lindt (2011)	Nonlinear FEM	FEM including hysteretic and anchorage behavior is compared to shake-table tests of a six-story apartment building

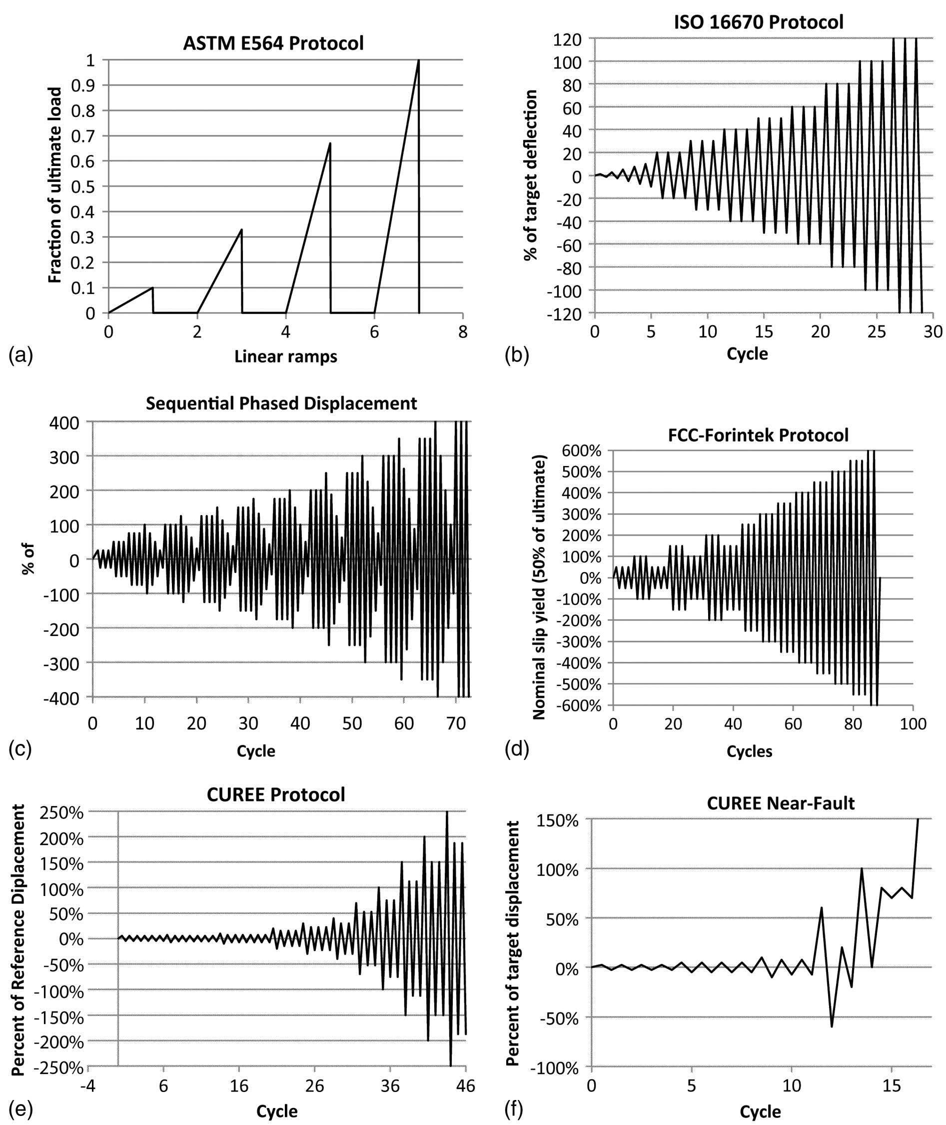

Fig. 2. Common shearwall testing protocols: (a) monotonic loading; (b) ISO-16670 loading; (c) sequential phased displacement; (d) FCC-Forintek loading; (e) CUREE loading; (f) CUREE near-fault loading

more attention to fastener or connection ductility may lead to improved methods of construction with brittle materials or in walls with openings (Merrick 1999; Uang and Gatto 2003; Rosowsky and Kim 2004a, b). Further, use of ductile framing or elastic adhesives to support brittle materials may allow more effective use of their strength without pushing these materials into the nonlinear range.

Horizontal Wood-Diaphragm Research

Research on residential diaphragms can be divided into two groups; floor (flat) diaphragms and roof diaphragms that usually have a slope or pitch (which may also be flat). Horizontal diaphragm studies are listed chronologically with the specific research focus in Table 5. Design practice has not typically differentiated between flat and pitched diaphragms (Breyer et al. 2007). Many excellent studies have been published on horizontal floor or roof diaphragms, however, there is very limited research on pitched roof diaphragms (Johnson and Burrows 1956; Tissell and Rose 1993) or roof diaphragms that include gypsum board ceilings (Walker and Gonano 1984; Alsmarker 1991). Significant numbers of these studies involve analytical models, rather than laboratory experiments.

Wood horizontal diaphragms have been traditionally considered flexible rather than rigid (ASCE 2010). Studies by Phillips (1990), Phillips et al. (1993), and Tarabia and Itani (1997) indicate that the assumption of a flexible diaphragm may be unconservative.

Thirty papers were found investigating different aspects of gypsum shearwall design, but only three considering the effect or contribution of the gypsum ceiling in the horizontal diaphragm design. The IBC (International Code Committee 2011a) Table 2306.3(3) contains 17 lines of gypsum board, gypsum lath and plaster, or Portland cement plaster shearwall design values, using only staples, but no diaphragm design values for gypsum products used on a ceiling. Though shear capacities of walls having both plywood and gypsum cannot be summed, the IBC permits the designer to use twice the lesser shear capacity (usually the gypsum), which treats the shear wall as if gypsum existed on both sides.

The restriction on combining gypsum and wood-panel sheathing strengths is based on the understanding that the stiffer material will provide most of the lateral resistance. This is true with respect to wood shear walls. However, with roofs that are framed with dimension lumber or metal-plate-connected trusses (MPCT), the top and bottom truss chords (or joists and rafters) are distinct and separate components, so it is unclear if some additional capacity or efficiency may be obtained through a combination of the ceiling and roof sheathing. Table 5 shows two papers on pitched roof diaphragms (Johnson and Burrows 1956; Tissell and Rose 1993) and only one paper on MPCT (Tissell and Rose 1993). Also, the greatest pitch tested was 4:12, which is less than in many current roofs. The majority of the wood-roof diaphragm research involves roof pitches of 3:12 or less, often with plywood overlaid on tongue and groove roof decking several inches thick and trusses of 2, 3, or 4 members bolted at joints. The roof diaphragm stiffness or strength for differing pitches of light 2-MPC truss configurations typically used today (6:12 or higher pitches) remain untested or unreported. There may be significant opportunities to improve the performance of MPCT with gypsum ceiling and structural panel diaphragms.

Finite-Element Modeling

Static testing of wood dwellings occurred primarily before 1990, with tests performed by Yokel et al. (1973), Yancy and Somes (1973), Tuomi and McCutcheon (1978) and Boughton and Reardon (1982). Dynamic testing of full-scale models of residential buildings was rare until the early 1990s, due to several problems. The expense of constructing a complete dwelling is great. There is also a limited number of shake tables capable of testing a full-sized model. As an alternative method of analysis, researchers have used finite-element (FE) models to test their analytical understanding of material and connection behavior, with model complexity ranging from simple static, linear-elastic models to complex 3-D nonlinear models analyzed with seismic excitation data. These analyses are listed chronologically including the specific focus in Table 6.

Yancey and Somes (1973) indicate that research is needed on torsional behavior, post-ultimate load behavior, and simplified, practical analytical models. They stated that "the available studies are either too complicated and time consuming or too simplified that their accuracy is questionable, p 9."

Recent research has included structural reliability or fragility analysis combined with probabilistic seismic hazard models to determine damage risk (Li et al. 2010; Kim and Rosowsky 2005a, b; Li 2005; Li and Ellingwood 2007). These methods require thousands of model evaluations to produce reasonable results. Typical studies use a few tested structures or components and then perform the required analysis calibrated to the physical tests.

Much effort has been spent developing various models without reaching a consensus on the methods and elements to be used. For example, there is a general consensus in the mechanics of modeling concrete and steel with various connections and fixity. Commonly used software does not require that the designer implement an element from scratch. With WFSFDs, there is no consensus on the methods used to model connections, shear walls or diaphragms. Many studies have been performed, independently of the others, and there are dozens of finite element model approaches for wood structures. However, it is difficult to compare the accuracy of models of structures using different elements and techniques. Research comparing these elements may assist in determining which would be most useful to the practitioner.

Wood-Frame Dwelling Testing

A limited number of full-scale experiments have been performed on WFSFDs, as summarized chronologically in Table 2. In static tests, loads or displacements are applied to a dwelling at specific locations to test deflections of diaphragms or shear walls (Yokel et al. 1973; Yancy and Somes 1973; Tuomi and McCutcheon 1978; Boughton and Reardon 1982). Dynamic tests can be performed using computer-controlled hydraulic rams attached to the structure or by securing the structure on shake tables capable of generating earthquake-level accelerations. The first full-scale shake-table test was reported by Fischer et al. (2000), with most of the subsequent studies being performed on ever-larger shake tables.

A small residential structure was designed and tested on a shake table at the University of California at San Diego (UCSD) as a portion of the CUREE-Woodframe Project (Fischer et al. 2000). The structure was heavily instrumented and many configurations for wall construction were examined. One objective for the project was to obtain as much data as possible on component and system deformations for potential study by other researchers.

Recent tests from the NEESWood project, which continues some of the work of the CUREE-Woodframe project, by Christovasilis et al. (2006), van de Lindt et al. (2006a, b), Pang et al. (2007), and Filiatrault et al. (2007) investigated the performance of a complex townhouse on two coupled shake tables.

The most recent project focuses on prediction, testing, and evaluation of a six-story wood-frame building tested on a shake table in Japan. A key component of the investigation was

Table 5. Wood-Frame Dwelling Testing

Reference	Loading	Focus of research
Yokel et al. (1973)	Concentrated, static and cyclic	Two story house before occupancy. Gypsum wall sheathing, trusses w/ plywood roof sheathing. Measured damping, natural frequency and drift
Yancey and Somes (1973)	Static, cyclic	Two story HUD Operation Breakthrough modular unit. Gypsum wall sheathing, trusses w/ plywood roof sheathing
Tuomi and McCutcheon (1978)	Static racking at various stages of construction	Component interaction study. One-story. Plywood wall and roof sheathing. Trusses with gypsum on bottom chord
Boughton and Reardon (1982)	Static	USAF building from 1940s converted to house. Applied loads to portions of the house to determine system load distribution
Sugiyama et al. (1988)	Static at specific locations	Tested stiffness and deformation of Japanese house under static loading
Phillips (1990); Phillips et al. (1993)	Cyclic, ASTM E72	Single-story rectangular house. Study indicates roof behaves as rigid diaphragm
King and Deam (1998)	Dynamic testing	New Zealand code. Evaluated the post-elastic performance of wall panel, used to develop a “dependable lateral load resistance rating, p 24”
Kasal et al. (1999)	3-D FEM nonlinear	Uses statistical properties of building components in FEM to distribute seismic forces to the lateral resisting elements. Then uses SDOF shear model to calculate displacements
Fischer et al. (2000)	Dynamic uniaxial shake table	CUREE-CalTech two-story single-family woodframe house was tested at UC San Diego. It was 4.9 × 6.1 m (16 × 20 ft.), 38 × 89 mm (nominal 2 × 4 in.) with OSB and oriented such that shaking occurred along the short dimension of the structure. Tested at 10 different phases of construction
Fischer et al. (2000)	Dynamic uniaxial shake table	Four types of shake-table tests were performed for quasi-static in-plane floor diaphragm tests, frequency evaluation tests, damping evaluation tests, and seismic tests, at up to five levels of increasing in amplitude.
Foliente et al. (2000); Foliente et al. (1998); Paevere and Foliente (2002); Phillips et al. (1993); Paevere et al. (2003)	Wind loading, static, dynamic and destructive	Tested single story L-shaped house containing required structural elements, with interior finishes. Concluded tributary area method was least accurate, and FEM gave most accurate results
Kohara and Miyazawa (1998); Miyazawa and Kohara (1998)	Dynamic	Tested two-story Japanese house
Ohashi et al. (1998)	Dynamic	Tested 5.4 × 3.6 × 2.9 m (17 × 12 × 8 ft) tall model house
Kharrazi (2001)	Shake-table and field tests	Vibration and damping tests on shake tables and houses in the field
Folz and Filiatrault (2001)	Cyclic SDOF FEM	CUREE Development of CASHEW model of displacement and energy dissipation in wood shear walls
Filiatrault et al. (2002)	Dynamic uniaxial shake table	CUREE UCSD house. Rectangular, two-story. Different configurations of sheathing, finish and mass distribution
Malesza et al. (2004)	Static	Applied static load to house center with cables and measured floor diaphragm deflection. FEM 1.45–2.54 times measured deflections, and rigid diaphragm model 1.84–4.92 times measured deflections
van de Lindt and Liu (2006)	Uniaxial shake table	Six tests of a one-story house with: (1) the exterior wood shear walls with only OSB and no nonstructural finishes, (2) the exterior wood shear walls with OSB and drywall, and (3) the exterior wood shear walls with OSB and drywall and a nonstructural partition wall
van de Lindt (2007)	Uniaxial shake table	Tested full-scale and half-scale house models. Determined that scaling was not reliable in wood-frame structures
Filiatrault et al. (2008)	Dynamic uniaxial shake table	CUREE two-story townhouse. Part of NEESWood project
Lu et al. (2006)	3-D shake table	Tested two-story wood-frame structure with I joists, OSB
van de Lindt et al. (2008, 2007); van de Lindt and Liu (2006)	Uniaxial shake table	Simple one-story box model, 24 tests of four specimens with six ground motions
van de Lindt et al. (2010, 2012); Pang et al. (2010); Pei and van de Lindt (2011)	Shake table	Five tests of six-story light framed apartment building. Examines damage, drift and performance of largest full-size structure to date. Part of NEESWood project
Kang et al. (2009)	ISO 16670 (cyclic, increasing)	Tests of nine full-scale one- and two-story, light-framed structures
Christovasilis (2011); Filiatrault et al. (2007, 2010, 2008)	Triaxial shake tables	Full-scale, two-story, light-frame wood townhouse building tested at MCEER on two triaxial shake tables. Part of NEESWood project
van de Lindt et al. (2011)	Shake table	Report of testing a six-story wood building on a one-story steel frame. Concludes structure performed well in testing, a first story SMF is a viable option to add commercial space at ground level, and DDD produced better performance than would have been expected under current IBC requirements. Part of NEESWood project

Note: DDD = Direct Displacement Design, a method of performance based design; HUD = U.S. Dept. of Housing and Urban Development; MCEER = Multidisciplinary Center for Earthquake Engineering Research, University of Buffalo, New York; SDOF = Single Degree of Freedom; SMF = Special Moment Frame, a type of steel structure; USAF = U.S. Air Force.

Table 6. Post-Frame Diaphragm Testing

Reference	Experiment	Conclusions
Hoagland and Bundy (1983)	Corrugated aluminum and steel attached with screws	Developed strength and stiffness values
Gebremedhin and Irish (1984)	—	Aluminum- and steel-clad, timber-framed, and screw-fastened diaphragms were tested as deep beams. Variables include direction of ribs, size of supporting grid, diaphragm width to length ratio, fastener spacing, and effect of an *opening*
Gebremedhin and Bartsch (1988)	Corrugated aluminum and steel panels with urethane foam inserts	Strength and stiffness increased 3–7 times with foam. Failure was sudden when foam sheared
Anderson and Bundy (1990)	Corrugated steel with openings	Plane truss analog under or over predicts stiffness by ∼10%. Number and type of fasteners have significant effect
McFadden and Bundy (1991)	Compares cantilever and two-bay diaphragm tests	Both tests gave similar values if the corners of the cantilevered test panel were reinforced
Bohnhoff et al. (1991)	25 steel diaphragm with rigid foam between steel and framing	Addition of insulation layer reduces stiffness and strength. Deformation of screws controlled failure mode
Woeste and Townsend (1991)	19 cantilevered panels	Cantilevered tests need framing stiffeners and out-of-plane restraint to be consistent
Gebremedhin et al. (1992)	Full-scale post-frame building with static loading	Endwall stiffness highly significant
Bohnhoff (1992a)	Analytic study	Demonstrates method of calculating frame stiffness and eave loads
Gebremedhin and Price (1999)	Full-scale post-frame building tests	Data show that the roof diaphragm halves act as a unit rather than two independent parts

to verify the applicability of performance-based design (PBD) for wood structures. See van de Lindt et al. (2010, 2011, 2012), Pang et al. (2010), and Pei and van de Lindt (2011) for further information.

Paevere et al. (2003) performed experiments on a full-scale L-shaped house using static and dynamic loading applied to the structure with a hydraulic ram. Displacements were measured at key locations and uplift forces were measured at locations where anchor bolts would typically be installed into a foundation.

In some cases, a specific structure was tested, but it was either a small, research-sized building or an individual, unique dwelling. Researchers used the testing protocol that they believed most important or most practical at the time. These tests cannot be easily compared due to these issues. From these limited tests, there has not been enough consistent, comparable data to permit evaluation of the significance of building geometric factors on the behavior of the structure. Recent projects have both performed experimental research and performed analysis, and so have resulted in models and techniques that are correlated. In historic cases where researchers performed well-documented testing, it was often difficult for subsequent researchers, not involved in the specific experiment series, to produce accurate models without making many assumptions. Recently, there have been projects such as the NEESWood Project, which involve many researchers simultaneously working on different aspects of the research. This helped improve communication among the researchers and is a favorable trend that should result in more useful results.

Early research on shear walls, diaphragms, and other components (e.g., straps, tie-downs) was primarily interested in determining yielding behavior, rather than system deformation or deformation-based damage. Whole-structure testing on shake tables is beginning to yield useful information on deformation and system effects. Full-scale testing has not, however, resulted in substantially more practical information on the design of individual WFSFD components, such as shear walls or diaphragms. Testing of WFSFDs has been of limited use thus far in design, in part because there are many variations in geometry and materials that have prevented development of accurate, general-purpose design methods.

Post-Frame Buildings

Unique to post-frame design is the roof-diaphragm shear reduction factor. No such reduction is used in WFSFDs, thus a discussion of post-frame research is merited and a brief discussion is included here. Post-frame buildings use the moment connection capacities of timber connections and the flexural capacities of columns with a fixed base to provide the lateral force resisting system for these structures Gebremedhin et al. (1986). These buildings can be heavy timber resort lodges or SFDs, but may also include many agricultural buildings. Typical construction of an agricultural post-frame building consists of corrugated metal siding and roofing over a timber framework.

Much of the agricultural post-frame design research consists of analytical studies rather than experimental programs (see chronological list with conclusions in Table 7). Generally, post-frame buildings have pitched roofs rather than flat, horizontal diaphragms. In Gebremedhin et al. (1986), an equation is used to calculate a reduction in the stiffness of horizontal diaphragms for pitches other than strictly horizontal. This is unique to post-frame construction and is not a part of typical design practice in WFSFDs. Post-frame testing programs specifically examining roof-diaphragm stiffness are summarized with their principal conclusions, chronologically in Table 8. Experiments generally used heavy trusses (due to the size of the structures) with corrugated sheet metal roofs, so it is not clear that these experiments are relevant to WFSFDs.

Earthquake Damage Surveys

Surveys of damage from major earthquakes in the United States include those shown in Table 3, which shows the survey reports or papers chronologically with seismic event and conclusions. Typical surveys review either substantial amounts of data at a limited level or a few specific cases in depth. Many studies (10 out of 13 in Table 3) are based on structures damaged in California. State laws that protect both the owners' privacy and the copyrights of the architect and engineer also limit California building surveys. Signed releases must be obtained from all these parties to gain

Table 7. Post-Frame Design Methods

Reference	Experiment	Conclusions
Gebremedhin and Woeste (1986)	Analytic study	Using diaphragm stiffness to redistribute loads resulted in smaller post sizes
Gebremedhin et al. (1986)	Analytic study	Demonstrates design using diaphragm stiffness to optimize member sizes
Gebremedhin (1988)	Analytic study	Describes the methods used in *METCLAD* design program
Gebremedhin et al. (1989)	Analytic study	Describes Met-X-PERT program design methods
Anderson and Bundy (1990)	Corrugated steel with openings	Plane truss analog under or over predicts stiffness by ~10%. Number and type of fasteners has significant effect
Bender et al. (1991)	Analytic study	Shows rigid diaphragm analysis results are similar to elaborate ASAE EP484.1 flexible analysis method
Bohnhoff (1992b)	Analytic study	Demonstrates method of calculating frame stiffness and eave loads
Niu and Gebremedhin (1997)	Analytic study	Demonstrates method of analyzing post-frame structure in a 3-D model
Carradine et al. (2000)	Analytic study	Demonstrates application of post-frame design methods to timber framed dwelling
Carradine et al. (2004a)	Analytic study	Demonstrates application of post-frame design methods to timber-framed dwelling with SIP panels

Note: SIP = structural insulated panel, a sandwich panel made from OSB glued to an insulating core.

Table 8. Previous Earthquake Damage Surveys

Reference	Earthquake	Dates	Plans	Conclusions
Kochkin and Crandall (2004)	New Madrid	1811–1812	No	Studied damage to historic homes. No current construction methods
Berg (1973)	Great Alaska	1964	—	Most damage to dwellings due to earth movement or tsunami
Moran and Bockemohle (1973); Pinkham (1973); Steinbrugge and Schader (1973)	San Fernando	1971	—	Contains detailed surveys of damaged wood-frame buildings
Falk and Soltis (1988)	California	1980s	No	Reviews wood-frame building performance in California earthquakes in 1980s
EQE Engineering (1989)	Loma Prieta	1989	No	Damaged homes generally pre-1940s with cripple walls, modern irregular homes, and apartments with soft first story
Holmes and Somers (1996)	Northridge	1994	No	Concludes, "the . . . earthquake should dispel the myth that wood construction is largely immune to earthquake shaking p 172"
Holmes et al. (1996)	Northridge	1994	Yes	Includes plans and photos of two damaged houses with analysis
Comerio (1997)	Loma Prieta	1989	No	Estimated that approximately 12,000 housing units were severely damaged or destroyed, and 30,000–35,000 incurred some minor damage
Thywissen and Boatwright (1998)	Northridge	1994	No	Examined database of ATC-20 surveyed structures. Concluded homes were mostly resistant
Yancey et al. (1998)	Great Alaska, San Fernando, Loma Prieta	various	No	Summarizes recent literature on damage surveys specifically related to house engineering
Poland and Scawthorn (2000)	Northridge	1994	No	ATC-38 study of 500 buildings <1,000 ft from fault
NAHB Research Center (1994)	Northridge	1994	—	Concludes most single-family dwellings had no structural damage to the roof or walls, but approximately 50% suffered some damage to the interior or exterior finishes, 7% suffered moderate or high damage to the finishes
Schierle (2003)	Northridge	1994	Yes	Includes plans and elevations of four damaged houses with analysis (CUREE)

access to the plans. So, in many cases, these are not available. Further, when an organization is charged with conducting the study, the work is generally targeted towards the final report and its conclusions, rather than concentrating on extensive details that would be useful to subsequent researchers. For example, plans were rare in the early studies, but more frequent in later studies. Elevations showing the sizes of openings in interior and exterior walls are nonexistent.

The 1971 San Fernando earthquake study contains valuable information on the state of seismic design as well as the results of field surveys of damaged buildings by Moran and Bockemohle (1973), Pinkham (1973), and Steinbrugge and Schader (1973). These studies provide the examination and opinions of the researchers, but lack plans and details sufficient for further analysis. Many of their recommendations have been implemented in the appropriate building codes.

The 1989 Loma Prieta earthquake was the largest event in California since the 1906 San Francisco earthquake. In a survey of damage, EQE Engineering (1989) noted that ". . . [wood-frame] buildings have generally performed well in past earthquakes," except "older (especially pre-1940s) homes, because these lack positive connections to their foundations or have raised floors supported by relatively weak cripple walls," and "some of the more irregularly shaped newer homes that lack clear load paths due to complex geometry or are built without enough wall area to resist the seismic forces, p 32." Additional serious problems included multi-story apartment buildings with garages on the first floor. The survey data did not include plans or details for the WFSFD examined.

The Applied Technology Council (ATC) (Poland and Scawthorn 2000) produced a study of 500 buildings located within 0.3 km (1000 ft) of the 1994 Northridge earthquake fault. Though this study is quite detailed, plans were excluded in the distributed electronic database. Therefore, the database is not useful for analyzing the design of those structures. It can only be used on a gross scale to compare building damage by type or location, for example.

The Earthquake Engineering Research Institute (EERI) published a report detailing damage to different types of structures and facilities by Holmes and Somers (1996). It details some of the types of damage seen in various WFSFDs during the 1994 Northridge earthquake. Holmes et al. (1996) include evaluation of two WFSFDs damaged in the Northridge earthquake. The first report describes damage to a two-story WFSFD constructed in 1958 and located within 0.8 km (1/2 mile) of a strong motion seismograph. Damage to this structure was nonstructural and the WFSFD was considered suitable for immediate occupancy. Notably, the cost of repairing the damage to the structure was actually so great that it was considered a total loss. The second WFSFD was a single-story home built in 1911 and seismically retrofitted three months before the earthquake. This WFSFD experienced minimal damage in comparison to WFSFD of similar construction in the immediate neighborhood. The report included plans and details, but no elevations or schedules that would show window and door opening sizes, therefore further analysis would depend on significant assumptions about the construction.

In a general survey, Crandell and Kochkin (2003) reviewed the history of WFSFD construction and related current design concerns to engineering practice. Engineering design uses the seismic provisions of the IBC (International Code Council 2011a). The IRC (International Code Council 2011b) is a prescriptive code, and is based on traditional methods of construction. Though related to engineering and construction practice, this code is not necessarily easily linked to engineering principles and calculations. The authors identified the following differences between engineering design practice and conventional prescriptive construction methods:

1. Lateral force resistance (shear walls and diaphragms), including perforated shear walls and rigid diaphragm behavior;
2. Connection design, discussing cross-grain tension and toe-nailing;
3. System effects, where loads are redistributed in the system, increasing its redundancy;
4. Safety margins and performance objectives, addressing the absence of a commonly understood level of performance for WFSFDs; and
5. Design loads, addressing differences between engineering loads and prescriptive design standards.

Subsequent work on the IRC (International Code Council 2011b) has attempted to address these issues (Crandell 2007; Crandell and Martin 2009).

Schierle (2003) provides engineering surveys of damaged residential and commercial buildings affected by the Northridge earthquake. Floor plans and elevations including categorization of the damage were included, along with the engineer's written evaluation. Since there were no elevations included, some assumptions need to be made regarding exact heights of windows and doors if this study is used for further analysis.

Damage surveys have shown the types of damage that have been problematic in WFSFDs. However, without significantly more detail in the surveys, it will be difficult to use these structures for further analysis. It is important to include more information in the future because these structures are of typical construction and have gone through major natural events, characteristics not necessarily true of WFSFDs constructed for laboratory research. Further refinement in methods and data collection will await the next major U.S. event.

Authors of some damage surveys suggest that correctly following building codes and engineered plans would mitigate or reduce seismic damage. It is certainly true that omitting one or two fasteners on each diaphragm will reduce its capacity. Further, the building codes that were once booklets that contractors could easily carry have become large tomes that are difficult to interpret. Within the damage surveys shown in Table 3, most indicate a number of design problems with the structures, e.g., cripple wall bracing (Falk and Soltis 1988; EQE Engineering 1989; NAHB 1994), but also one or two quality control items on each structure, e.g., no anchor bolts (Falk and Soltis 1988). Part of the problem is that without the original plans for the WFSFD, it is difficult to know if a hold-down or anchor bolt is missing because it is not on the plans or because it was omitted during construction. So, differentiating between design errors and construction defects has been difficult for these surveys. Nevertheless, the quality of WFSFD construction is generally not as good as commercial construction and more quality control would help.

Damage surveys also frequently conclude that seismic strengthening efforts are effective. Much attention is usually paid to ensuring that new construction adheres to the current code, whereas upgrading older construction is considered *elective* (Holmes et al. 1996).

Damage Estimation Methods

There are a number of different damage-estimation methods and strategies that have been developed by different researchers and organizations. Table 9 summarizes these methods and their specific purposes. These strategies are largely based on the accepted traditional basis for design, life-safety. Buildings constructed to the code requirements in the United States are intended "...to minimize the hazard to life and improve occupancy capability of essential facitities after a design level event or occurance" (International Code Council 2011a). Under these strategies, a building will most likely suffer significant damage to the structural system and need to be significantly repaired or replaced due to the economics of repair.

In recent seismic events, some wood-framed WFSFDs which were judged habitable were nevertheless considered total losses by the insurance companies (Holmes et al. 1996). The damage was nonstructural, limited to cracking of walls and finishes. The cost of repairing building finishes was too great relative to the value of the WFSFD. None of the existing damage-estimation methods can accurately predict the level or cost of damage because the methods are directed towards evaluating and obtaining the life-safety standard.

Lucksiri et al. (2012) adapt the basic philosophy of rapid visual screening to the unique characteristics of WFSFDs, emphasizing plan geometry, and validate the method by a comparison of 480 representative models.

Generally, damage-estimation methods seem to be well-developed at present. These methods were mainly developed after the Northridge earthquake. Additional opportunities for research in this area will require further comparison to concurrent experiments or await the next major event in the United States. Since this is most likely to be in California, amendments to state laws allowing access to building plans by researchers would be very useful. Additionally, involving an analysis component by researchers whose primary focus is in wood construction would help to expose information gaps and omissions. Similar to the experiences of dynamic structural testing, it would be useful to perform detailed or FEA analysis simultaneously with damage investigation, so that useful comparisons with existing design methods can be obtained.

Table 9. Current Damage-Estimation Methods

Reference	Name of Method	Purpose
ASCE (2006)	ASCE/SEI 41-06 Prestandard and commentary for the seismic rehabilitation of buildings	Employs linear/nonlinear, static/dynamic analyses. Objective is to avoid individual component failure
FEMA (2002)	FEMA-154 Rapid visual screening of buildings for seismic hazards	Provides a method for a quick visual survey of general structures. Only nine possible ratings for any dwelling regardless of age, material or complexity of construction. Identifies structures to receive a more detailed analysis
ASCE (2003)	ASCE 31 Handbook for the seismic evaluation of buildings (previously FEMA 31.)	Uses calculations based on simple methods and assumptions. Does not consider system effects of redistribution of forces. Intended to check common potential component deficiencies that might contribute to collapse
International Code Council (2011a)	2012 International building code	Provides the basis for engineering design of new structures, prescriptive design of residential and modifications to existing structures. Sometimes these are used to evaluate seismic conformance of existing structures
van de Lindt (2005)	Reliability model for drift performance	Damage-based seismic reliability model for light-frame wood structures subject to earthquake load
Baxter (2004); Baxter et al. (2007)	—	Compares different screening, evaluation, rehabilitation and design provisions for wood- framed structures
Lucksiri et al. (2012)	Rapid visual screening of wood-frame dwellings with plan irregularity	Approach to screening for seismic hazards in wood houses with plan irregularity is developed. Plan shape, number of stories, plan area, cutoffs in area, and wall openings are investigated

Research Challenges and Future Directions

Knowledge needs to be created to ensure that WFSFDs can be designed and built to resist seismic loads to the level expected by building owners, civil authorities and society expectations. By producing research that improves accurate modeling of different WFSFD configurations, designers will understand what components are truly required and what level of performance can be expected. For example, designers currently design roof structures based on data developed in the 1950s, when the cost of lumber was relatively low due to sale of inexpensive federal timber in the national forests and old growth lumber was readily available. Presently, the decision on whether these configurations are cost-effective remains with the architect, engineer, and the owner, not with the researcher. Consequently, cost of construction is rarely a reported factor in WFSFD research.

Research has not addressed many areas in seismic behavior of WFSFDs. Improvements in the following areas are crucial to improving seismic performance of WFSFDs:

- Innovative methods: Conventional construction methods were developed to be cost-effective and easily installed. For example, the use of short or *pony* walls to span vertically from a short concrete foundation to the first level of a house built on sloping terrain. But these methods have been difficult to analyze and research has shown some of them to be ineffective. Thus, there is a significant need to develop new and innovative construction materials, connections, fasteners, and techniques to overcome the limitations of wood, such as increasing system ductility.
- Brittle finishes: Present research has concluded that brittle materials are of limited value in providing seismic resistance. There has been limited research to improve seismic performance of brittle materials, such as gypsum wallboard, including the effects of openings, nor to improve ductility in the construction of shear walls designed with brittle materials. Brittle finishes may have stiffness and strength that can be exploited if ductile methods of connection can be developed. Use of elastomeric sheets, resilient channels or ductile fasteners could be routes to achieve this. Examination of using all of the gypsum walls in an SFDs may result in elastic (nondamaging) performance. There is also substantial opportunity to study the behavior of MPCT in WFSFDs lateral-force-resisting systems (LFRS), as wells as combinations of gypsum board ceilings with structural wood sheathing on the MPCT and on flat roofs.
- Horizontal/pitched diaphragms: Abundant data exist on rectangular horizontal or low-pitch gable roofs particularly with a heavy timber supporting framework. Different configurations (L, T, and U shapes, for example) need to be tested, as do roofs of differing pitches and hip roofs. It needs to be determined whether a shear stiffness or strength reduction factor similar to post-frame design is applicable to WFSFDs. OSB and structural insulated panel (SIP) roofs should also be tested to verify whether existing data are applicable to their design. Assumptions of flexible diaphragm behavior continue to persist in the building codes in spite of research indicating that the assumption is not valid for all structures; therefore additional research is needed to show that the assessment of diaphragm flexibility needs to be made in each case by the designer. If pitch results in a stiffness reduction factor, some rigid roofs could be flexible or semirigid or rigid at different pitches. Research should address how a stiffness reduction factor, if any, affects design of SFD horizontal diaphragms.
- Finite-element methods: Researchers have contributed much effort in finite element modeling of wood structures, but have not yet developed consensus methods and elements that should be used. PBD may result in better designs for buildings than the present code based methods. However, to date, different researchers have used different methods of analysis and design. As a result, it is difficult to compare the accuracy of models of structures using PBD, different FE elements, and techniques. Research comparing these methods may assist in determining which would be most useful to the practitioner. For many practitioners, PBD methods will need to be codified to result in widespread use. But at present, few of these methods have been adopted or provided in the commercially available finite element software, limiting use by design practitioners. Synthesizing the existing research and disseminating this research to the designers is the greatest challenge here.
- Whole-structure testing: Historic tests of WFSFDs have been of limited use because it has been so difficult to completely quantify the structure so as to allow an independent researcher to refine their analysis methods. To date, there have not been enough consistent, comparable data to permit evaluation of

the significance of building geometric factors on the behavior of the structure. Whole house testing had primarily measured damage to the WFSFD components, rather than determining whether a limiting behavior has been reached by the WFSFD as a whole. Therefore, such testing is not easily correlated with the testing of individual components. Research on shear walls, diaphragms and other components is usually based on yielding performance as a method of determining whether life-safety goals are being met. Such tests generally do not measure the amount or type of damage at various loading intervals. Substantial recent progress in testing large structures has been made. Understanding and integrating the measured results into present analysis methods remains the major challenge.

- Damage-estimation methods: Damage-estimation methods seem to be well developed at present, and are mainly products after the Northridge earthquake. Additional opportunities for research in this area will require further comparison to concurrent experiments (such as application to a shake-table structure before testing) or await the next significant earthquake in the United States.
- Damage surveys: More complete reports of damaged WFSFDs are needed. Open access to California plans and documents on WFSFDs for research would assist this effort greatly. (California is not the only state affected by earthquakes, but earthquakes are common and the laws restricting release of the original plans affect researcher's access to data that might improve design.) There is a challenge to define the required document sufficiently to permit detailed analysis while protecting the designer from the risk of losing their intellectual property within the plans. It is important to include more information in the future because these structures are of typical construction and have gone through major natural events, characteristics not necessarily true of WFSFDs constructed for laboratory research. Further refinement in methods of documenting the existing structure and communicating that data to future researchers is needed so that the present or future PBD models can be applied to real structures with real damage. It would be helpful to test some structures or portions of structures using the different testing protocols developed to date, to determine which protocol(s) best simulate(s) actual seismic stresses, deflection and damage. Application of FEA to sample damaged structures before demolition, would allow more accurate modeling to be performed.
- Collaboration: Research continues along paths that seem most likely to improve design and evaluation of WFSFDs. The following trends seem very positive: full-scale shake-table tests of large structures; comparison of tested structure performance with results from finite element design programs for both strength and prediction of deformation of components; and multi-researcher projects where test results have been analyzed, and finite element models produced by researchers either from the same institution or operating under the same grant, thus ensuring access to sufficient structural detail to permit accurate modeling. There is a strong need to develop a better understanding of the effects of WDSFD components, attachment, LFRS and how loads are distributed to these elements within the structure. A consensus needs to be developed on performance objectives for WDSFDs with respect to damage and repair costs, including new design techniques which balance life-safety with the effects of damage to building finish materials. Finite element analyses of seismically damaged WFSFDs would lead to a better understanding of component performance and allow evaluation of seismic testing protocols.

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