Woodframe Project
Testing and Analysis Literature Reviews

Edited by
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2001
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Preface

The Northridge Earthquake of January 17, 1994 was in effect a full-scale dynamic test of thousands of wood structures. The Field Investigations element of the CUREE-Caltech Woodframe Project explores this information via Statistical Analysis (1) and this Case Studies report.

The Case Studies present observations of engineers who have investigated woodframe buildings damaged by the Northridge Earthquake. Their information is presented in this report in consistent content and format, designed to capture this valuable resource in a form that will help develop new design guidelines and code provisions.

The CUREE-Caltech Woodframe Project originated in the need for a combined research and implementation project to improve the seismic performance of woodframe buildings, a need which was brought to light by the January 17, 1994 Northridge, California Earthquake in the Los Angeles metropolitan region. Damage to woodframe construction predominated in all three basic categories of earthquake loss in that disaster:

- **Casualties:** 24 of the 25 fatalities in the Northridge Earthquake that were caused by building damage occurred in woodframe buildings (2);
- **Property Loss:** Half or more of the $40 billion in property damage was due to damage to woodframe construction (3);
- **Functionality:** 48,000 housing units, almost all of them in woodframe buildings, were rendered uninhabitable by the earthquake (4).

Woodframe construction represents one of society’s largest investments in the built environment, and the common woodframe house is usually an individual’s largest single asset. In California, 99% of all residences are of woodframe construction, and even considering occupancies other than residential, such as commercial and industrial uses, 96% of all buildings in Los Angeles County are built of wood. In other regions of the country, woodframe construction is still extremely prevalent, constituting, for example, 89% of all buildings in Memphis, Tennessee and 87% in Wichita, Kansas, with "the general range of the fraction of wood structures to total structures...between 80% and 90% in all regions of the US…” (5).

Funding for the Project is provided primarily by the Federal Emergency Management Agency (FEMA) under the Stafford Act (Public Law 93-288). The federal funding comes to the project through a California Governor’s Office of Emergency Services (OES) Hazard Mitigation Grant Program award to the California Institute of Technology (Caltech). The Project Manager is Professor John Hall of Caltech. The Consortium of Universities for Research in Earthquake Engineering (CUREE), as subcontractor to Caltech, with Robert Reitherman as Project Director, manages the subcontracted work to various universities, along with the work of consulting engineers, government agencies, trade groups, and others. CUREE is a non-profit corporation devoted to the advancement of earthquake engineering research, education, and implementation.
Cost-sharing contributions to the Project come from a large number of practicing engineers, universities, companies, local and state agencies, and others.

The project has five main Elements, which together with a management element are designed to make the engineering of woodframe buildings more scientific and their construction technology more efficient. The project’s Elements and their managers are:

1. **Testing and Analysis**: Prof. André Filiatrault, University of California, San Diego, Manager; Prof. Frieder Seible and Prof. Chia-Ming Uang, Assistant Managers
2. **Field Investigations**: Prof. G. G. Schierle, University of Southern California, Manager
3. **Building Codes and Standards**: Kelly Cobeen, GFDS Engineers, Manager; John Coil and James Russell, Assistant Managers
4. **Economic Aspects**: Tom Tobin, Tobin Associates, Manager
5. **Education and Outreach**: Jill Andrews, Southern California Earthquake Center, Manager

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Task 1.1.1 - Shake Table Tests of a Simplified Two-Story Single-Family House Literature Review

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Summary

Until recently, the majority of woodframe construction houses were built based on experience and tradition. Significant damage to residential woodframe construction from earthquakes and high winds has created a need to examine the current design practices and building codes for woodframe construction. During the last 50 years, there has been significant testing on individual woodframe building components such as shear walls and diaphragms. However, there has been little testing on full-scale testing of woodframe construction. It is important to quantify the interaction of the individual building components during lateral loading of full-scale woodframe construction. The full-scale testing of woodframe houses that has been completed can be divided into lateral wind load testing and shake table testing.

Table 1 summarizes the research conducted on full-scale testing of woodframe construction. Beginning in the 1950’s, researchers tested woodframe houses subjected to simulated wind loads. Houses tested in the United States were constructed using typical two-by-four wall construction. Houses tested in Japan were constructed using post and beam construction. The interior surfaces of the walls and ceilings were covered with gypsum wallboard or sheetrock. The exterior surfaces of the walls were sheathed with wood or metal siding. Diagonal braces at the corners of the building were used to resist lateral loads in the structures. In earlier testing, the roofs of these houses were sheathed with evenly spaced boards. In more recent testing, the roofs were sheathed with plywood. The houses were either loaded with several hydraulic rams producing concentrated lateral loads, several hydraulic rams in a whiffle tree arrangement producing distributed lateral loads, or an air bag producing distributed lateral loads. The majority of the full-scale woodframe structures tested with simulated wind loads preformed well without
catastrophic failure. The horizontal floor and roof diaphragms had small in-plane deformations suggesting they behaved more like rigid diaphragms. Non-structural finish materials greatly increased the lateral stiffness of the structures. The diagonal braces did not resist a significant portion of the lateral load when gypsum wallboard and exterior wall siding were installed.

Beginning in the late 1980’s, researchers began testing full-scale woodframe houses subjected to earthquake ground motions on a shake table. All of these shake table tests were performed outside of the United States in Japan, Korea, and Greece. These houses were typically post and beam construction with diagonal braces designed to resist the lateral inertia loads. The exterior wall surfaces were covered with wood or fiberboard siding or mortar stucco. The interior wall and ceiling surfaces were typically covered with gypsum wallboard. The floors and roofs of these structures were sheathed with plywood. The houses were tested with several structural configurations during the construction process. The houses were subjected to several ground motions from recent earthquakes and variable frequency motions to quantify the dynamic response of the structures. These structures performed well when subjected to the earthquake ground motions with only one catastrophic failure. Again, the non-structural finish materials greatly increased the lateral stiffness of the structures. Without gypsum wallboard and exterior siding, the diagonal braces were not capable of resisting the lateral loads in the structures. Researchers were able to predict the experimental response of the structures using a bi-linear hysteresis for shear walls in analytical models.

As a result of damage to residential woodframe structures from recent earthquakes in the United States, more shake table testing is required for these structures. There is a lack of full-scale testing of houses framed with two-by-four walls and sheathed with plywood or Oriented Strand Board (OSB). This shake table testing should examine the common construction methods used in the United States. Based on shake table testing in Japan, this testing should examine the influence of finish materials on the lateral stiffness of the structures. In addition, the testing should investigate the influence of window and door openings of the dynamic performance of the structure. Previous testing shows a need for information on the distribution of forces in the full-scale structure.
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In 1954, Dorey et al. (1957, pp. 29-49) at the Division of Building Research of Canada tested a full-scale single-story house subjected to simulated wind and snow loads. The researchers wanted to quantify the strength and rigidity of a full-scale house without exterior sheathing. The researchers also wanted to use the resulting experimental data in writing performance requirements for houses in Canada. Previous researchers had tested individual building components, but this testing did not investigate the interaction of the building components in a three-dimensional structure.

The house was built in the field in 1948 for experimental purposes, but not specifically for structural testing. The single-story house was 24 ft – 6 in (7.5 m) wide (direction of loading) by 36 ft – 8 in (11.2 m) long. The woodframe structure was built on a basementless block wall foundation that extended 3 ft to 4 ft (0.9 m to 1.2 m) below grade. Two-by-six sill plates were attached to the top of the foundation wall using anchor bolts. The floor system was constructed using two-by-eight joists at 16 in (406 mm) on center supported mid-span with a built-up timber beam. The floor was sheathed with 13/16 in (21 mm) thick diagonal boards with 3/4 in (19 mm) thick hardwood flooring. The exterior walls were constructed using two-by-four studs at 16 in (406 mm) on center with one-by-four let-in bracing at the corners. These walls were covered with 12-pound asphalt-saturated felt paper and interlocking aluminum siding on the exterior. The interior side of these walls was covered with 3/8 in (10 mm) thick gypsum wallboard. Most interior partition walls were constructed using two-by-four studs at 16 in (406 mm) on center with 3/8 in (10 mm) thick gypsum wallboard on both sides. As an experiment, a few interior walls were constructed using only 2 in (51 mm) thick solid plaster. The roof was framed using two-by-four rafters at 16 in (406 mm) on center with one-by-four collar ties. The roof framing was sheathed with one-by-six boards spaced 6 in (152 mm) apart with aluminum roofing. The ceiling was constructed with two-by-sixes supported mid-span by an interior partition wall and was sheathed with 3/8 in (10 mm) thick gypsum wallboard.

A rigid reaction frame was constructed using Bailey bridging to span across the structure. Hydraulic rams were used to apply loads to the roof and walls using a “whiffletree” arrangement. Hardwood loading pads were used at each point of loading to further distribute the load. The house was tested four times with increasing severity of load during each test. During the first test, the structure was loaded to simulate wind with internal suction representing a condition with windows open on the leeward side only. During the second test, the structure was loaded to simulate wind with internal pressure representing a condition with windows open on the windward side only. For the third test, researchers tested the structure with simulated wind loads and one-half of the design snow load. For these tests, the structure was loaded with simulated winds starting at 70 mph (113 km/h) and ending at 120 mph (193 km/hr) increasing in 10 mph (16 km/hr) increments. The load was maintained for one-half hour for each of the first three load increments and one hour for each of the final three load increments for each test. During the fourth test, simulated snow loads were applied to the roof in increments of 13 psf (0.6 kPa) (25% of the design snow load) to a maximum of 73 psf (3.5 kPa) (143% of the design snow load). Again, the load was maintained for one-half hour at each load increment.
Deflections of the structure were measured using a system of pulleys and wires. Deflection measurements were taken at several places on the transverse end walls (direction of loading) and intermediate transverse vertical planes. At each measurement location, piano wire was run over low-friction aircraft pulleys out a window to a instrument hut. Each of the 43 wires was tensioned with a one-pound weight strung across a deflection board. In addition to the deflection measurements, plaster telltales were installed at the ceiling level of several interior corners in the structure to observe damaging deformations. The plaster telltales were made by replacing the paint from the intersecting planes at a corner with several layers of gypsum plaster.

The structure performed well when subjected to wind loading. In the first test of simulated winds with internal suction, no damage resulted at equivalent wind velocities of 90 mph (145 km/hr). Cracking became apparent in the plaster telltales as the simulated winds were increased to 120 mph (193 km/hr). During this first test, the maximum lateral displacement of the structure was 0.12 in (3 mm) and the largest bending deflection in the roof rafters was 0.31 in (8 mm). In the second test of winds with internal pressure, a few of the existing cracks in the plaster telltales increased in width. The maximum lateral displacement of the structure was 0.08 in (2 mm) and the maximum bending deflection in the rafters was 0.80 in (20 mm). No further significant damage resulted from the third test with simulated wind and one-half of the design snow load. Since the snow load acted in the opposite direction of the simulated wind load on the roof, the largest deflection in the rafters occurred at the minimum wind load for this test. In the final test with simulated snow loads, a few collar ties failed at a load of approximately 73 psf (3.5 kPa) (143 percent of the design snow load). The allowable deflection of L/240 was exceeded on the front roof slope by 182 percent during this test. The wind tests performed on the structure showed that the structure was capable of withstanding 1.8 times the design wind load using only one-by-four let-in bracing and interior finish materials to resist lateral loads.

The Wood-Frame House as a Structural Unit
By Homer T. Hurst

Between 1963 and 1964, Homer Hurst (1965, pp. 3-27) conducted a study of a full-scale woodframe house in cooperation with the National Forest Products Association. The researchers studied the deflection of floors subjected to simulated gravity loads and the racking resistance of walls subjected to simulated wind loads. However, the discussion here will be limited to the experimental procedure and results developed from the lateral load testing of the structure. The main objective of the study was to determine the structural performance of a conventional full-scale, woodframe house under simulated wind loads acting diagonally on the structure. At the time of the experiment, residential wood structures were designed on a piece-by-piece basis without consideration of three-dimensional effects. The researchers wanted to investigate the structural contribution of various building components such as the roof, walls, partitions, and floors to the overall performance of the complete structure.

The test structure was 36 ft (11.0 m) long by 28 ft (8.5 m) wide with three bedrooms, a living and dining room area, kitchen, and one and a half bathrooms. The house was built in a laboratory on concrete block walls that formed a basement. Two-by-eight floor joists spliced over a middle
laminated girder spanned the transverse direction of the house. The floor was sheathed with one-by-eight diagonal sheathing and the basement ceiling was covered with 1/2 in (13 mm) thick sheetrock. The walls were constructed using typical two-by-four construction and were sheathed with 3/8 in (10 mm) thick plywood on the exterior and 1/2 in (13 mm) thick sheetrock on the interior. Interior partition walls were framed using two-by-fours with 1/2 in (13 mm) thick sheetrock on both sides. The roof was framed with manufactured roof trusses spaced at 24 in (610 mm) and was sheathed with one-by-eight boards.

The wall and diaphragm systems were tested with simulated wind loads during 7 stages of dismantling of the structure. The completed structure sheathed with plywood on the exterior and sheetrock on the interior was tested during Stage 1. Prior to Stage 2 testing, the sheetrock joints were taped and plastered. The sheetrock was removed from all partition walls for Stage 3. The partition walls were removed for Stage 4. For Stage 5 testing, the sheetrock was removed from the exterior walls. The one-by-eight roof sheathing was removed prior to testing in Stage 6. For the last stage, Stage 7, the gypsum wallboard was removed from the roof ceiling except for a 4-ft (1.2 m) wide strip adjacent to the two leeward walls.

The structure was loaded cyclically with a simulated wind load acting diagonally on the house. Hydraulic load cylinders were used to simulate uniform wind pressure on the walls with concentrated loads at 16 in (406 mm) on center and on the roof with concentrated loads at 24 in (610 mm) on center. All roof and wall surfaces were subjected to the same simulated wind pressures up to a maximum of 16 psf (766 Pa) loaded in 4 psf (192 Pa) increments. For each load increment, the load was cycled on and off three times during six-minute cycles. For each load increment, a gravity load was imposed on the roof equal to the simulated wind pressure. Horizontal deflection recorders measured deflection at 19 points on the two leeward walls.

The structure performed well up to simulated wind loads of 16 psf (766 Pa). The researchers initially intended to test the structure to simulated wind loads much greater than 20 psf (958 Pa), but the foundation wall failed at a load of 20 psf (958 Pa). Maximum wall deflections occurred midway between the floor and top plates during all stages of testing. The researchers found that the taping and plastering of the walls prior to Stage 2 had little effect on the horizontal deflection of the structure. In addition, there was little change in the horizontal deflections as a result of removing the sheetrock from the interior partitions for Stage 3. Removing these partitions before Stage 4 had little effect on the horizontal deflections as well. After the sheetrock was removed from the exterior walls for Stage 5, again, there was little change in the horizontal movement of the building. However, when the roof sheathing was removed for Stage 6, there was a large increase in horizontal deflection of the structure. During Stage 7 with the sheetrock removed from the majority of the ceiling, a simulated wind load of 12 psf (575 Pa) caused the end wall to pull away from the 4-ft (1.2 m) wide strip of sheetrock. From these tests, the researchers concluded that the ceiling and the roof sheathing were very important structural elements in limiting horizontal movement. However, since the 3/8 in (10 mm) plywood sheathing was much stiffer than the interior sheetrock, researchers urged that conclusions should not be made about the effect of the taping and plastering of the sheetrock and the interior partitions on the racking resistance of the structure.
In 1972, Yokel et al. (1973, pp. 1-23) tested a full-scale two-story house that was representative of housing in the United States. A large producer of conventional housing donated a recently completed house located in Maryland for testing. In this investigation, the researchers wanted to determine whether existing drift limitations for medium-rise and high-rise structures were applicable to conventional low-rise housing. They also wanted to quantify the dynamic response characteristics of conventional housing to improve dynamic lateral load calculations in design.

The two-story woodframe test structure was 47 ft (14.3 m) long by 26 ft (7.9m) wide (direction of loading). The house contained a family room, bedroom, bathroom, and a garage on the lower story and a L-shaped living and dining room, kitchen, three bedrooms, and two bathrooms on the upper story. The exterior walls were constructed with two-by-four studs and the interior walls were constructed using a combination of two-by-four and two-by-three studs. All interior wall surfaces were covered with 3/8 in (10 mm) thick gypsum wallboard. The exterior wall surfaces were covered with 1/2” (13 mm) thick gypsum sheathing with asbestos shingles and 3/8 in (10 mm) thick beveled wood siding. The exterior walls were braced at the corners with one-by-four let-in bracing installed at a 45-degree angle. The second floor was framed with two-by-eights spaced at 12 in (305 mm) with 5/8 in (16 mm) thick plywood sub-flooring and 1/2 in (13 mm) thick gypsum wallboard on the ceiling below. The roof was framed with two-by-four trussed roof rafters at 24 in (610 mm) on center and was sheathed with 1/2 in (13 mm) thick plywood and asphalt shingles.

The test structure was loaded over a series of four static tests and one dynamic test. The static tests were intended to measure the stiffness of the house when subjected to simulated wind loads in the transverse direction. Four single-acting hydraulic rams with a load capacity of 10 tons were used in the static tests. During three of the tests, the concentrated loads were applied horizontally at the centerline of the second story floor joists. The concentrated loads were applied at the bottom chord of the roof trusses during the other static test. In each of these tests, the load was cycled producing maximum total loads between 2 kips (8.9 kN) and 10 kips (44.5 kN). The dynamic test was intended to measure the dynamic response characteristics of the structure. During this test, a 12” long piece of steel pipe was installed between the hydraulic ram and the house. After each ram was loaded, the steel pipe was removed with a sharp blow of a hammer and the structure was allowed to vibrate freely. This test had to be repeated several times because of limited recording equipment.

Thirty-two displacement transducers were used to measure the overall building movement and the racking deformation of the walls. Two of the transducers were more sensitive for use in the dynamic response test of the house. Since the structure was tested during a time of inclement weather, all of the recording devices and equipment had to be installed on the interior of the structure.

During the static tests, the researchers found that the walls behaved nearly elastically. Thus, the results of the upper story testing and the lower story testing could be superimposed on one
another. A wind load of approximately 15 psf (718 Pa) was simulated using a total ram load of 2.82 kips (12.5 kN) at the bottom chord of the roof truss and a total ram load of 5.64 kips (25.1 kN) at the second story floor joists. The simulated wind load on the structure was calculated in accordance with ANSI Standard A58.1-1955. As a result of these tests, the researchers believe that the drift at the upper story was much less than that calculated using design criteria for medium-rise and high-rise structures. The let-in braces used at the corners resisted the majority of the racking loads because only a small portion of the exterior wall distortion was transmitted to the interior gypsum wallboard. The roof diaphragm underwent significant in-plane deformations suggesting it behaved more like a flexible diaphragm. However, the second floor diaphragm acted more like a rigid body during loading of the structure. During the dynamic tests, the natural frequency of the house was around 9 Hz. The percentage of critical damping ranged from four to nine percent of critical damping with an average of six percent. However, the researchers urged that the conclusions regarding the dynamic properties should be questioned because of the resolution of the dynamic recording equipment.

**Testing of a Full-Scale House Under Simulated Snowloads and Windloads**

By Roger L. Tuomi and William J. McCutcheon

Tuomi et al. (1973, pp. 1-32) tested a full-scale single story house under simulated snow and wind loads in a laboratory. The researchers wanted to determine the response of a typical woodframe construction house subjected to horizontal wall loads and vertical roof loads. They narrowed their main objective to four specific goals. Firstly, the researchers wanted to quantify the racking resistance of shear walls subjected to horizontal concentrated loads during several stages of construction. Secondly, they wanted to investigate the serviceability limits in which door and windows became inoperable and cracks in the wall coverings became prevalent. Thirdly, they wanted to correlate the racking resistance of a structure subjected to concentrated horizontal loads at the shear walls with distributed horizontal loads on a wall perpendicular to the direction of loading. Lastly, the researchers wanted to determine the ultimate cause of structural failure.

The test structure was 24 ft (7.3 m) long (direction of loading) by 16 ft (4.9 m) wide. The house was constructed with typical materials used in 1973 meeting the minimum size requirement according to the Federal Housing Administration Minimum Property Standards, FHA No. 300. The 24-ft (7.3 m) length of the house was typical of residential construction. However, the 16-ft (4.9 m) width of the house was only about one-half of the width of a typical house. The two 24-ft (7.3 m) long gable-end walls (direction of loading) each contained a window and a door. In the west gable-end wall, the door opening was located near the north end of the wall whereas the door opening in the east gable-end wall was located near the south end of the wall. The two north and south end walls did not contain openings. Four wide flange beams supported the four-wall box-shaped test structure. Two-by-six sill plates were bolted to the wide-flange beams with 1/2 in (13 mm) diameter bolts at 8-ft (2.4 m) intervals. The walls were constructed using two-by-fours (stud grade Douglas-fir) at 16 in (406 mm) on center with double top plates and intermediate wall blocking. The walls were sheathed with 3/8 in (10 mm) plywood with 6-penny nails spaced at 6 in (152 mm) around the perimeter of the panels and at 12 in (305 mm) to
intermediate supports. The interior side of each wall was covered with 1/2 in (13 mm) gypsum wallboard with 1-1/4 in (32 mm) ring-shank nails at 8 in (203 mm) on center. Western red cedar siding was installed horizontally to the exterior of the structure. The roof was constructed with manufactured roof trusses spaced at 24 in (610 mm). The roof was sheathed with 3/8 in (10 mm) plywood with 6-penny nails spaced at 6 in (152 mm) around the perimeter of the panels and at 12 in (305 mm) to intermediate supports. The ceiling was covered with 1/2 in (13 mm) gypsum wallboard using 1-1/4 (32 mm) ring-shank nails at 6 in (152 mm) on center.

The house structure was tested with lateral loads at six stages during construction. In the first five tests, shear wall deflections were limited in an effort to keep the walls in their elastic range. Concentrated loads were applied to the top of the longitudinal shear walls for the first five tests. During Stage 1, the initial stiffness of the longitudinal walls was measured with only 3/8 in (10 mm) plywood sheathing on the exterior. Windows and doors were installed in Stage 2 to evaluate the reduction of stiffness with openings. The interior gypsum wallboard and cedar siding were added for the testing in Stage 3. During Stage 4, the transverse walls were constructed to evaluate the influence of the walls perpendicular to the direction of loading. The roof system was constructed for Stage 5 to quantify the racking resistance of a complete three-dimensional structure. For Stage 6, a uniform load was applied to the south transverse wall to determine the stiffness, ultimate strength, and load distribution for a structure subjected to wind loading. Since there was little structural failure from to the six lateral tests, vertical load tests on the roof were also conducted. The roof was loaded with a “whiffletree” arrangement using four jacks to produce 32 equal concentrated loads.

During the concentrated lateral load testing, loads were applied in 0.025-in (0.6 mm) displacement increments with hydraulic jack actuators. The load was applied at a rate greater than or equal to two minutes per increment. After each new level of deformation, the load was reduced to zero and residual displacements were measured after two minutes. The maximum lateral displacements were kept between 0.125 in (3 mm) and 0.150 in (4 mm) for the first five test stages. For the uniform load testing of Stage 6, an air bag system was used to apply lateral loads to the south transverse wall. The transverse wall was loaded in increments of 5 psf (0.24 kPa) up to a maximum pressure of 40 psf (1.9 kPa). Once this maximum pressure of 40 psf (1.9 kPa) had been reached, the load was cycled three times to this level. Following the third cycle, the lateral uniform load was increased until failure. For the vertical load roof testing, the roof was loaded to 16,000 lbs (71.2 kN) (approximately 45 psf) in 4,000-lb (17.8 kN) increments and then cycled four times at this level. The load was then increased again in 2,000-lb (8.9 kN) increments until failure.

The researchers found that the racking resistance of the structure was more than adequate. The load-displacement relationships were found to be curvilinear as opposed to the linear elastic relationship that was assumed. After the first two cycles of load, there was residual set in the walls due to the inelastic nail-slip. As a result of the ultimate uniform load testing of Stage 6, component failure first occurred when sole plate on the loaded transverse wall broke loose at a lateral uniform load of 63 psf (3.0 kPa). In addition, component failure occurred at a sill plate joint at lateral uniform load of 123 psf (5.9 kPa). At a load of approximately 70 psf (3.4 kPa) (corresponding to a wind speed of 165 mph), a center stud on the loaded transverse wall failed in
bending. After comparing the concentrated load testing to uniform load testing, the researchers discovered that three-fourths of the lateral load was resisted by the shear walls and the other one-fourth of the lateral load went directly into the foundation. In terms of serviceability, one of the doors began to bind at a lateral deflection of 0.10 in (2.5 mm). At one inch of bending deflection in the loaded transverse wall, the gypsum wallboard showed no significant damage. However, the gypsum wallboard joints were not taped. The trusses performed very well when subjected to simulated snow loads with a failure load of 135 psf (6.5 kPa).

**Full-Scale tests of Manufactured Homes Under Simulated Wind Loads**

By Andrew H. Stewart, Aron Kliewer, James R. Goodman, and Edward M. Salsbury

In 1988, Stewart et al. (1988, pp. 97-111) tested two manufactured homes under simulated wind loads. The main objective of this testing was to evaluate the lateral load capacity of a manufactured home. Specifically, the researchers wanted to determine how the transverse walls (direction of loading) provided racking resistance under lateral loads. They also wanted to determine the interaction between the roof diaphragm and the shear walls. Lastly, the researchers wanted to evaluate the loading protocol for use in future testing of manufactured homes.

The two identical 13.7 ft (4.2 m) wide by 66 ft (20.1 m) long homes were constructed under standard production line conditions. However, non-structural components such as cabinets, plumbing, and electrical components were not installed for ease in instrumentation of the structure. The transverse wall system was designed to resist a lateral load of 25 psf (1.2 kPa) through five shear walls. The roof of each manufactured home was framed using trusses and covered with metal roofing and a fiberboard ceiling.

A total of 78 electronic instruments were used to record the behavior of each structure under lateral loads. Electronic transducers were used to measure horizontal and vertical deflection of the shear walls and roof diaphragm. Load cells and pressure sensors were used to measure applied loads and internal loads. Strain gages were used to monitor loads in connector straps and lag bolts that connected the floor of the homes to the chassis below. Lastly, eight dial gages and 2 pendulums were used to monitor box rotation.

The structures were loaded in two different sequences. The first sequence of loading involved applying concentrated loads with hand-operated hydraulic actuators to each of the five transverse walls individually. The loads were applied slowly to allow researchers to record loads and deformations at each increment. Loads up to a maximum of 3,500 lbs (15.6 kN) were applied to the top and bottom of each of the shear walls. This concentrated load testing was used to estimate the in-place stiffness of the shear walls. In the second sequence of lateral loading, uniform lateral loads were applied to the longitudinal side of the building to simulate wind loading. Air bags pushing against a reaction wall and the structure provided the distributed load up to 75 psf (3.6 Kpa). The pressure in the air bags was measured using a water manometer and a pressure sensor. In addition, eight load cells were used to verify the pressure measurements.
The concentrated load tests indicated that the roof diaphragm had a high stiffness compared to the five shear walls. Since the diaphragm acted rigidly, the majority of the lateral load was transferred to the stiffer end walls. The uniform load tests revealed that both test structures had a much higher ultimate capacity than the design value of 25 psf (1.2 kPa). The first test house had an ultimate lateral capacity of 75 psf (3.6 kPa) producing a safety factor of 3.0. Failure occurred in one of the end shear walls, but the structure continued to resist load since the failure was not catastrophic. The second test structure had an ultimate lateral capacity of 71 psf (3.4 kPa). Testing was stopped at this point due to the failure of a tie-down strap. During these tests, the researchers found that the shear walls exhibited a nearly linear, elastic response. They concluded that the current design practice considering the interaction of the shear walls, roof diaphragm, floor diaphragm, and chassis in the design of manufactured homes was satisfactory. They also concluded that the testing protocol worked well and should be used in future testing.

Full-Scale Test on A Japanese Type of Two-Story Wooden Frame House Subjected to Lateral Load
By Hideo Sugiyama, Naoto Andoh, Shigeru Hirano, Takayuki Uchisako, and Noboru Nakamura

In 1988, Sugiyama et al. (1988, pp. 55-61) tested a full-scale Japanese style two-story house subjected to lateral loads. The main objective of this experimental investigation was to evaluate the influence of commonly ignored structural factors on the racking resistance of a structure. The researchers wanted to quantify the influence of wall sheathing above and below door and window openings on the racking resistance of the wall. In addition, they examined the effect of shear frames placed perpendicular to the direction of lateral loading.

The test structure was a full-size two-story house 7.28 m (24 ft) wide (direction of loading) by 10.01 m (33 ft) long. Four shear wall frames resisted lateral loads in the direction of loading during experimentation. The frames were constructed using 12 cm by 12 cm (4 3/4 in by 4 3/4 in) wood columns connected with 12 cm by 33 cm (4 3/4 in by 13 in) beams at the second floor. The frames were connected with 12 cm by 30 cm (4 3/4 in by 12 in) sub-beams spaced at 1.82 m (6 ft). The second floor was framed with 4.5 cm by 9.0 cm (1 3/4 in by 3 1/2 in) joists and spaced at 22.5 cm (9 in) and sheathed with 3-ft by 6-ft (91 cm by 183 cm) sheets of 1.2 cm (1/2 in) thick lauan plywood. A large opening was left in the second floor for a staircase. Studs with dimensions of 12 cm by 3 cm (1 1/4 in by 4 3/4 in) were installed between posts at 45.5 cm (18 in) on center. Let-in bracing in each of the shear wall frames resisted lateral loads. The walls were sheathed with exterior calcium silicate wall siding during certain stages of testing. The hip roof was sheathed with 3-ft by 6-ft (91 cm by 183 cm) sheets of 1.2 cm (1/2 in) thick lauan plywood. Windows, doors, interior wall sheathing, and ceiling coverings were not installed during testing.

The structure was tested at six stages during construction. During Stage 1, let-in bracing was installed in both directions parallel and perpendicular to the lateral load and a portion of the second floor sheathing was omitted. The sheathing that was omitted during Stage 1 was installed for testing during Stage 2. For Stage 3 testing, the shear wall frames parallel to the direction of
During test Stages 1 through 5, each shear wall frame was loaded individually using hydraulic jack actuators at only the second floor to measure the initial stiffness of each shear wall frame. The concentrated lateral load was cycled increasing in magnitude with each cycle. During Stage 6 testing, the entire structure was loaded at once with two partially-distributed loads at the second floor using a cyclic push-pull loading that increased in magnitude with each cycle. Fifty-nine electric dial gages were used to record the displacements in both horizontal directions of points along the second floor at the sill plates. In addition, the vertical displacements of columns with let-in bracing were recorded.

Since the load was applied to each shear wall frame individually during the first five test stages, four simultaneous equations could be solved to provide the stiffness of each wall. The researchers found that the total stiffness of the first floor walls in Stage 1 and Stage 2 were nearly the same suggesting that the larger opening in the floor diaphragm had little effect on the total wall stiffness. The total stiffness in the walls of Stage 3 was about 50% greater than that of Stages 1 and 2. This suggests that the addition of the calcium silicate siding greatly increases the lateral stiffness in the direction parallel to loading. There was negligible difference in lateral stiffness between Stages 3 and 4 suggesting that the application of the calcium silicate siding on the walls perpendicular to the direction of loading had little effect on the stiffness of the walls parallel to the direction of loading. However, the researchers believe this behavior was due to very small racking deformations in the walls parallel to the direction of loading and could change at higher racking deformations. They believe more tests should be conducted in this area. The total stiffness during test Stage 5 was about 10% to 15% greater than that of test Stage 4 indicating that the application of sheathing materials above and below the door and window openings increases the wall stiffness. During the entire structure testing of Stage 6, local failure of the house occurred as a result of the buckling of a let-in brace on an interior shear wall frame at an applied structure load of about 32,000 lbs (142 kN). From measurements of the second floor diaphragm deformations, the researchers concluded that the in-plane rotation was very small.

Experiments on a Three-Storied Wooden Frame Building Subjected to Horizontal Load
By Motoi Yasamura, Isao Nishiyama, Tatuo Murota, and Nobuyoshi Yamaguchi

In 1988, Yasamura et al. (1988, pp. 262-273) investigated the lateral resistance of a three-story woodframe house. A revision of Japanese Building Law in 1987 permitted woodframe buildings to be constructed three stories high. The main objective of the experiment was to determine if three story woodframe houses were structurally safe when subjected to lateral loads. The
researchers wanted to obtain experimental data for use in calculating the horizontal resistance of the structure. The shear resistance of each story was compared to theoretical calculations based on non-linear load-slip relations of a nail joint.

The three-story house was 7.28 m (23.9 ft) wide by 9.1 m (29.9 ft) long (direction of loading). In the longitudinal direction, the two exterior shear walls at each floor were sheathed with 9.5 mm (3/8 in) thick plywood and 12 mm (1/2 in) thick gypsum wallboard. The interior shear walls located at the center of each floor were sheathed with 12 mm (1/2 in) thick gypsum wallboard on both sides. For all wall sheathings, nails were spaced at 10 cm (4 in) around the perimeter of panels and at 20 cm (8 in) to intermediate supports. The sub-floor for each floor was sheathed with 12 mm (1/2 in) thick lauan plywood. The roof was sheathed with 9.5 mm (3/8 in) thick lauan plywood.

The structure was tested under three different sheathing configurations during construction. During the testing of Specimen A, the longitudinal walls were sheathed, but the sub-floor and the transverse walls were not sheathed. The longitudinal walls and the sub-floor were sheathed during the testing of Specimen B. The longitudinal walls, transverse walls, and sub-floor were all sheathed during the testing of Specimen C.

The structure was loaded using three hydraulic jacks at the top of the third story at the two exterior shear walls and the interior shear wall. Electric transducers measured horizontal and vertical displacements of each story and tension forces in each holdown bolt. In the testing of Specimen A, the load was applied monotonically producing equal displacements at each of the three loading points. Specimens B and C were tested with an increasing cyclic load at each of the three shear walls. In these last two tests, the load applied to the interior wall was one and a half times the load applied to the exterior shear walls. In addition, a rotation mass generator at the roof was used for forced vibration tests of Specimens B and C.

As a result of the experimental testing, the researchers discovered that the shear resistance of the north longitudinal wall was approximately one and a half times the shear resistance of the south longitudinal wall due to a larger number of openings on the south wall. However, this difference in shear resistance had very little effect on the torsional deformation of the structure since the transverse walls had sufficient stiffness to resist torsional deformation. The shear deformations were relatively small in the horizontal diaphragms suggesting that they could be considered rigid. The researchers found that the horizontal load was distributed approximately in proportion to the length of shear walls without openings. Much of the failure in all of the specimens occurred on the second story, particular in the south and middle walls. The exterior plywood and interior gypsum wallboard buckled on the compression side around openings and was torn on the tension side around openings.

Through the forced vibration tests, several important patterns were noticed in the natural frequencies of the structure. After subjecting Specimen C to horizontal loads and causing damage, the first mode of the natural frequency decreased from 5.8 Hz to 3.1 Hz. The finished house with all interior and exterior finish materials had a natural frequency of 4.8 Hz. In addition,
the application of sheathing to the transverse walls resulted in an increase in the torsional natural frequency from 4.8 Hz to 8.8 Hz.

The experimental results obtained were compared to theoretical results using Tuomi’s racking model. This model predicts a load-deformation relationship for shear walls using the height and width of the shear panel, nailing, load-slip, and shear strain of the panel. A least squares method was used to develop load-deformation relationships for the experimental results of the plywood and gypsum wallboard shear walls. The researchers found that the theoretical horizontal resistance predicted by Tuomi’s racking model exceeded the experimental horizontal resistance. However, if a modification was made to the load-slip relation used in Tuomi’s racking model, the theoretical racking resistance was very close to the experimental racking resistance.

Full-Scale Dynamic Test of a Two-Storied Timber Frame Construction House using a 6-DOF Earthquake Simulator
by P. G. Carydis and E. A. Vougioukas

Carydis et al. (1989, pp. 17-21) tested a two-story timber frame construction house using a 6-degree of freedom earthquake simulator at National Technical University of Athens, Greece in 1989. The purpose of this experimental study was to demonstrate that wood houses constructed according to the Timber Frame Construction (T.F.C.) method were adequately resistant to earthquakes.

The two-story test structure was 3.6 m (11.8 ft) wide by 3.6 m (11.8 ft) long with a first story height of 2.9 m (9.5 ft) and a second story height of 2.6 m (8.5 ft). The Canadian Code of T.F.C. was followed for construction and the Greek Code was followed for dead and live loads. The exterior walls of the house were covered with asbestos cement over a bituminous paper with a chicken net metal. On two adjacent walls, plastic fibers were used to reinforce the cement plaster. In order to provide an adequate dead and live load in the structure, sacks filled with sand were placed on the second floor and roof.

The main shock of the Kalamata earthquake was used to excite the structure in the three principle directions simultaneously. The Kalamata earthquake struck the city of Kalamata on September 13, 1986 with a magnitude of 6.2 on the Richter scale. The test structure was tested with 40 repetitions of the Kalamata earthquake. The structure was instrumented with a few accelerometers to record the response.

Overall, the test structure performed well during the testing. After approximately 3 to 4 repetitions of the earthquake, nails at the base became loose and required repair. The plaster on the exterior cracked, but this cracking did not correspond to any damage to the wood substructure. After the 40th repetition of the Kalamata earthquake, the largest crack in the plaster had a width of the order of 2 mm (1/16 in). During repetitions 20 to 40, the cracks did not significantly enlarge. After the 20th repetition, a beam in the second floor failed at the location of an existing knot due to relatively high vertical accelerations.
After the 1st repetition of the earthquake, the measured natural period of the structure was 0.18 seconds in the longitudinal and transverse directions, and 0.16 seconds in the vertical direction. After the 20th repetition, the natural period was 0.21 seconds in the longitudinal direction, 0.20 seconds in the transverse direction, and 0.17 seconds in the vertical direction. After the last repetition, the natural period was 0.22 seconds in the longitudinal and transverse directions, and 0.17 seconds in the vertical direction. The maximum acceleration of various parts of the structure was of the order of 2 g. The damping in the structure varied from repetition to repetition. The damping of the structure was around 17% of critical during the 15th repetition of the earthquake.

Lateral Load Sharing by Diaphragms in Wood-Framed Buildings
by Timothy L. Phillips, Rafik Y. Itani, and David I. McLean

Phillips et al. (1993, pp. 1556-1571) tested a full-scale single story wood-framed structure subjected to lateral loads in 1990. The researchers wanted to determine the load-sharing characteristics of different building components through measured internal forces and displacements. This experimental study determined how a horizontal diaphragm affects the distribution of the load into shear wall elements. In addition, the study evaluated the hysteretic response of the test structure under cyclic lateral load. The researchers also determined the stiffness of the wood shear walls with different sheathing materials.

The 16-ft (4.9 m) wide (direction of loading) by 32-ft (9.8 m) long single-story test structure was constructed in a laboratory. The floor of the test structure was constructed with two-by-ten joists spaced at 16 in (0.4 m) with 5/8 in (16 mm) thick plywood sheathing. Four 16-ft (4.9 m) transverse shear walls divided the house into three rooms. Each shear wall was designed to produce a different lateral stiffness by using different sheathing materials and varying sizes of door and window openings. The shear walls were framed with two-by-fours spaced at 16 in (0.4 m) with a single two-by-four sill plate and a double two-by-four top plate. The exterior west and east shear walls were sheathed with 19/32 in (15 mm) thick T1-11 exterior siding on the outside and 1/2 in (13 mm) plywood on the inside. The interior west shear wall was sheathed with 1/2 in (13 mm) thick gypsum wallboard on both sides. The interior east shear wall was sheathed with 1/2 in (13 mm) thick plywood on both sides. The longitudinal walls were constructed similarly to the shear walls with 19/32 in (15 mm) thick T1-11 exterior siding on the outside and 1/2 in (13 mm) thick gypsum wallboard on the inside. The roof was constructed with nine manufactured trusses spanning the structure in the longitudinal direction. The roof was sheathed with 1/2 in (13 mm) thick exterior plywood and the ceiling was sheathed with 1/2 in (13 mm) gypsum wallboard.

Two servo-controlled hydraulic actuators were used to apply concentrated loads at the top of each of the four shear walls. A steel I-beam was used to divide the load from each hydraulic actuator into point loads at the top of two shear walls. The rate of loading and unloading of the walls was 1.78 kN/min. (400 lb/min.) as specified by ASTM E72. The structure was tested in four stages during the construction process. Stages I and II tested the racking resistance of the transverse shear walls prior to the construction of the longitudinal walls or roof diaphragm. During Stage I, the four shear walls were tested with sheathing on one side only. During Stage II,
the shear walls were tested with sheathing installed on both sides. The four transverse shearwalls and the longitudinal walls were constructed for Stage III testing. During Stage IV, the complete structure with all walls and roof diaphragm was tested with all walls and roof diaphragm. Stage IV test loads were considerably larger to represent overload conditions.

The structure was instrumented with four load cells at each point of load application. Each of the four shear walls was instrumented with four horizontal and four vertical load cells to measure the internal forces at the base of the walls. Linear variable differential transducers (LVDTs) measured the gross displacement at the top of each wall, the sill plate slippage, and the sill plate uplift for each shear wall.

During Stage I and II, load-deformation relationships were developed for each shear wall. The resulting relationships showed that the stiffness of the shear walls was additive with additional layers of sheathing. The hysteretic loops exhibited a pinching behavior common in timber shear walls. During Stages III and IV, the loads were first applied to adjacent pairs of shear walls and then to all four shear walls simultaneously. This loading sequence allowed the researchers to determine the load sharing characteristics of the structure with and without the roof diaphragm. During Stage III, applied loads were not transferred to unloaded walls without the roof diaphragm. The exterior shear walls carried a larger percentage of the total applied load compared to the interior shear walls due to the corner connections. During Stage IV, the roof diaphragm transferred some of the load into the unloaded walls. During these tests, the longitudinal walls carried up to 23% of the applied load, but this percentage decreased at higher levels of loading. The experimental results of the load distribution were compared to calculations following the 1991 UBC. From these calculations, the researchers concluded the roof diaphragm behaved more like a rigid diaphragm than a flexible diaphragm.

### Shaking Table Test of Full-Scale Wood-Framed House

By Yuki Tanaka, Yoshimitsu Ohasi, and Isao Sakamoto

Between November and December 1995, Tanaka et al. (1998, pp. 2487-2492) tested a full-scale, two-story, post and beam woodframe house using a shake table. The 1995 Kobe Earthquake caused significant damage to post and beam woodframe houses in Japan resulting in loss of life. From the testing of this house, the researchers wanted to assure the safety and seismic performance of woodframe housing. In addition, they wanted to examine the influence of nonstructural sheathing materials on the dynamic response of the structure. The researchers developed an analytical model in an effort to predict the experimental response.

The two-story house was 7.28 m (23.9 ft) wide by 7.28 m (23.9 ft) long with a total floor area of 53.0 m² (570 ft²). Each story was approximately 2.95 m (9.7 ft) high and the total height of the structure at the ridge was 7.54 m (24.7 ft). The structure was designed using a seismic shear coefficient of 0.28. The exterior of the building was sheathed with siding and the interior was covered with gypsum wallboard. Diagonal braces were used to resist the lateral forces in the shear walls. The total length of shear walls in the test structure was approximately 1.5 times the required length of shear walls established by the building code. If the structure were to be
simplified as a two-degree-of-freedom system, the mass at the roof would be 9.0 tons and the mass at the second floor level would be 9.3 tons.

The test structure was excited using a 15 m (49.2 ft) wide by 15 m (49.2 ft) long shake table capable of motion in one horizontal and one vertical direction. The test structure was excited using the 1995 Kobe Earthquake recorded at the Japan Meteorological Agency (JMA) Kobe station and the 1940 El Centro Earthquake with an amplitude scale factor of 1.5. For both earthquakes, the North-South component and the vertical component of the motion were used to excite the structure. In addition to ground motion testing, sweep frequency testing was also performed with a constant acceleration of 30 gal (0.03 g) to measure the natural frequencies of the house. The structure was tested during three phases of construction with different sheathing materials for a total of 8 experiments with different input motions. For Phase 1, the exterior siding, interior gypsum wallboard, and structural frame were installed for testing. The exterior siding was removed for testing during Phase 2. For Phase 3, only the structural frame with diagonal braces was tested. Damage to the gypsum wallboard was repaired between Phases 1 and 2 and the diagonal braces were replaced between Phase 2 and 3. The test structure was instrumented with accelerometers, displacement meters, and strain gages.

By removing sheathing materials with each phase of testing, the researchers found how this affected the damage in the structure. During the first phase of testing with the JMA Kobe ground motion and the El Centro ground motion, damage was limited to minor cracking in the exterior siding and gypsum wallboard around window openings. During Phase 2, cracks again appeared in the gypsum wallboard around window openings. Three of the fourteen diagonal braces on the first floor failed by buckling or by tensile failure. There was no damage to the second floor diagonal braces. Ten of the fourteen first floor diagonal braces failed during Phase 3. In addition, all eight of the second floor diagonal braces failed during this phase of testing. These trends in damage to the structural frame suggest that the nonstructural finish materials resist a significant portion of the lateral forces in the structure.

During Phase 1 of testing, a maximum acceleration recorded at the roof level was 2123 gal (2.16 g), equal to 2.6 times the peak input acceleration. The maximum acceleration recorded during Phase 2 was smaller than Phase 1. The acceleration response during Phase 3 was very small because the diagonal braces failed which greatly reduced the lateral stiffness of the structure. By using the same JMA Kobe ground motion for all three phases of testing, the researchers were able to compare the story drift levels for each phase. For Phase 1, the first story drift was 1/30 rad (3.3%) and the second story drift was 1/89 rad (1.1%). The first story drift was 1/15 rad (6.7%) and the second story drift was 1/40 rad (2.5%) for Phase 2. During the last phase of testing, the first story drift was 1/18 rad (5.6%) and the second story drift was 1/17 rad (5.9%). These drift levels suggest the exterior siding is very effective in resisting lateral deformation. The maximum total base shear during testing was about 20 tons representing a seismic coefficient between 1.0 to 1.1. The maximum axial compression was 3 to 4 tons and the maximum axial tension load was 1 to 2 tons in the diagonal braces. The axial load in the braces was considerably larger during Phase 3 than during Phase 1 because the gypsum wallboard and exterior siding resisted a significant portion of the load during Phase 1. From these results, the researchers concluded that the nonstructural sheathing materials added considerable stiffness to the structure.
An analytical model was developed to predict the experimental response. A simple two degree-of-freedom shear model was used with lumped masses at the second floor and roof levels. Bilinear hysteresis loops were used for the shear wall elements. The analytical model matched the experimental hysteresic loops well. In addition, the story drift predicted by the analytical model at the first and second floor matched the experimental drift.

Full-Scale Shaking Table Test of Two Story Wooden Dwelling Houses
By Katsuhiko Kohara and Kenji Miyazawa

Following the 1995 Hyogo-ken Nanbu Earthquake, a joint research group composed of Kogakuin University, Tokyo University, and others tested six two-story woodframe houses using a shake table. In this paper, the Kohara et al. (1998, pp. 548-555) discussed the testing and results of two of the structures: Type B and Type F. From these tests, the researchers wanted to determine the damage to the structures resulting from specific ground motions used on the shake table. In addition, they wanted to quantify the dynamic behavior of the structures including the natural frequencies, displacements, and accelerations. Lastly, the researchers wanted to examine the hysteresis loops of the shear walls in the structures.

Both two-story tests structures were 11.83 m (33.8 ft) long by 7.28 m (23.9 ft) wide with story heights of 2.9 m (9.5 ft). The total floor area of each structure was 133 m² (1432 ft²). For both structures, lateral forces were resisted by varying amounts of diagonal braces and plywood sheathing during different phases of testing. The interior wall surfaces of both structures were sheathed with gypsum wallboard. The exterior wall surfaces of structure Type B were covered with mortar stucco. The exterior wall surfaces of structure Type F were sheathed with siding. The roofs of both structures were covered with roofing tiles.

The two test structures were tested with various construction configurations using several input motions. The structures were tested using a sine wave sweeping frequency motion, the Japan Meteorological Agency (JMA) 1995 Kobe Earthquake, and the 1940 El Centro Earthquake with a scale factor of 1.5. In addition, micro-tremor measurements were made for each experiment to determine the natural frequencies of the structures. The longitudinal direction of both test structures was excited with the North-South component of these ground motions. The transverse direction of both test structures was excited with the East-West component of these ground motions. Both structures were excited with the vertical component of the ground motions.

Structure Type B was tested during five different phases for a total of eleven experiments. For Phase 1, the structure was tested with diagonal braces, mortar stucco, and gypsum wallboard. The sill plate anchor bolts were fastened and one-third of the mortar stucco was removed for Phase 2. One-third of the gypsum wallboard was removed and the sill plate anchor bolts were not fastened for Phase 3. For Phase 4, the sill plate anchor bolts were fastened, one-third of the diagonal braces were removed, and an additional one-sixth of the gypsum wallboard was removed. The sill plate anchor bolts were again fastened and the majority of the diagonal braces were removed and replaced with plywood sheathing for Phase 5.
Structure Type F was tested during five different phases for a total of ten experiments. For the first phase, diagonal braces, siding, and gypsum wallboard was installed throughout the test structure. Eccentricity was introduced in the test structure by making an opening on the south side of the building and rearranging the diagonal braces to alter the stiffness of some of the shear walls for Phase 2. For Phase 3, the exterior wall siding was removed from the first floor walls and the sill plate anchor bolts were fastened. The gypsum wallboard on in the interior first floor walls was removed for Phase 4. For Phase 5, most of the diagonal braces were removed and replaced with plywood sheathing and the holdown anchors were fastened.

At the start of experimental testing on the structures, the natural frequencies were 6.49 Hz and 6.05 Hz for structure Type B and Type F, respectively. As a result of removing diagonal braces, mortar stucco, and gypsum wallboard over different phases of testing and cumulative damage effects, the natural frequencies of the structures decreased with each test. Researchers found that when the natural frequency of the structure was 4 to 5 Hz, extensive damage occurred to the mortar stucco or exterior siding board and to the gypsum wallboard. In addition, they found that damage occurred to the structure frame when the natural frequency of the structure was around 3 Hz.

From the first phase of testing of both structures with the JMA Kobe earthquake, researchers found that the input acceleration was magnified 1.3 to 1.8 times in the structure. The structures had first story displacements between 1/52 rad and 1/120 rad (0.8% to 1.9% drift) during Phase 1. By removing one-third of the mortar stucco from structure Type B, the input acceleration was magnified 2.0 to 2.3 times in the structure with first story displacements between 1/28 rad and 1/46 rad (2.2% to 3.6% drift). After removing the exterior siding from structure Type F, the input acceleration was magnified 2.2 to 2.4 times with first story displacements between 1/23 rad and 1/29 rad (3.4% to 4.3% drift). These results suggest that both the stucco mortar and exterior wall siding were both effective in limiting the response of the structures.

In structure Type B, the researchers found that the diagonal braces resisted between 7% and 17% of total base shear while the stucco mortar resisted between 21% and 47% of the base shear. In structure Type F, the diagonal braces resisted between 29% and 54% of the total base shear. From hysteresis loops, the researchers found that walls with mortar stucco from structure Type B had a much higher stiffness than those with exterior siding from structure Type F. However, the stiffness of the walls with mortar stucco decreased over the phases of testing due to cracks. The stiffness of the walls with exterior siding remained constant during testing.

### Shaking Table Tests of a Real Scale Wooden House Subjected to Kobe Earthquake

By Yoshimitsu Ohashi, Isao Sakamoto, and Masahiko Kimura

In 1996, Ohashi et al. (1998, pp. 556-563) tested three woodframe houses subjected to ground motions on a shake table. Although three woodframe houses were tested, only the single-story structure was reported in this paper. Since woodframe houses suffered extensive damage during the 1995 Hyogoken-Nanbu (Kobe) Earthquake, the researchers wanted to determine the seismic safety of these structures. In addition, they wanted to quantify the effects of nonstructural finish.
materials on the stiffness of the structure. The researchers also examined the woodframe house with configurations in which the stiffness of the house was symmetric and eccentric.

The test structure was 5.4 m (17.7 ft) long by 3.6 m (11.8 ft) wide and 2.9 m (9.5 ft) high. Although the test structure was a single story, it was equivalent to the first story of a two-story structure. The post and beam structure was constructed with 120 mm (4 3/4 in) by 120 mm (4 3/4 in) columns at the corners with 45 mm (1 3/4 in) by 90 mm (3 1/2 in) diagonal braces. The north and south walls of the house had a single door opening at the center and the east and west walls each had two symmetrically placed window openings. The second floor of the house was sheathed with plywood. In some phases of testing, the interior surface of the walls was sheathed with gypsum wallboard and the exterior surface of the walls was sheathed with cement fiberboard siding. The sill of the house was fastened to the steel base with anchor bolts. In addition, the corner columns were attached to the base with holdown anchors. Additional weight was added to the floor in order to create a seismic shear coefficient of 0.2.

The house was tested during four stages of construction. For Phase 1, the frame structure (without gypsum wallboard or cement fiberboard siding) was tested with a low supplemental dead and live weight. The frame structure was tested using the full dead and live weight in both standard and eccentric braced conditions for Phase 2. Each of the four walls was constructed with four diagonal braces for the standard braced condition. For the eccentric braced condition, the North and West walls were constructed with six braces in each wall and the South and East walls were constructed with two braces in each wall. For Phase 3, the frame structure with gypsum wallboard was tested in both standard and eccentric braced conditions. The cement fiberboard siding was installed for Phase 4 and tested in only the standard braced condition.

The structure was tested using the JR-Takatori record from the 1995 Kobe Earthquake with the North-South component, East-West component, and the vertical component. All four phases were tested using the JR-Takatori ground motion scaled to a maximum velocity of 25 cm/s, equivalent to one-fifth of the full-scale recorded motion. In addition, one phase was tested using the JR-Takatori ground motion scaled to a maximum velocity of 50 cm/s. Two phases were tested using the full-scale JR-Takatori ground motion with a maximum velocity of 135 cm/s in the North-South direction. In addition, micro-tremor measurements were made before and after each phase of testing to measure the natural frequencies of the structure. Random motion excitation tests and sine wave excitation tests were preformed to verify the natural frequencies.

Following the final phase of testing with both gypsum wallboard and exterior siding installed, no damage was found after the 50 cm/s excitation. After the first full-scale excitation, cracks formed in the gypsum wallboard around the corners of openings. After the second full-scale excitation, the cracks in the gypsum wallboard became larger and some of the exterior siding began to fall down. However, there was no damage to the structural frame and braces. Accelerations were recorded up to 1000 gal (1.02 g) at the floor level in the North-South direction representing a response magnification factor of 1.3. From the natural frequency testing of the structure phase by phase, the stiffness of the structure increased as the gypsum wallboard and exterior siding were installed. As a result, the percentage of the total base shear resisted by the diagonal braces decreased as the additional finish materials were applied. During Phase 3 with gypsum wallboard
installed, the diagonal braces resisted between 30% and 50% of the total base shear. During Phase 4 with both gypsum wallboard and exterior siding installed, the diagonal braces resisted between 10% and 20% of the total base shear. In addition, the tension forces in the holdown anchors increased as the finishing materials were applied.

The researchers developed an analytical model using a bi-linear and slip hysteresis. Newmark’s beta method was used for integration of the ground motions. The stiffness and the yield displacement for the hysteresis model were determined from the force-deformation relationships measured in the experimental testing. The researchers concluded this method could be used to predict the seismic performance of a woodframe structure since the results were close to the measured response of the structure.

**Experimental Study on the Aseismic Capacity of a Wooden House Using Shaking Table**

By Jeong-Moon Seo, In-Kil Choi, and Jong-Rim Lee

In 1998, Seo et al. (1999, pp. 1143-1162) tested two one-quarter-scale post and beam woodframe house models using a shake table. In this experiment, the researchers wanted to determine the maximum peak ground acceleration that an ancient Korean commoner’s house located in areas of low to moderate seismicity could sustain without collapse. The first model was subjected to an earthquake record from a rock site and the second to an earthquake record from a soft soil site. The ground motion was scaled such that the peak ground acceleration level increased incrementally until the test models experienced severe damage or failure. In addition, the researchers wanted to measure the natural frequency and the damping in the test models.

The two single-story one-quarter-scale identical models were 1.8 m (5.9 ft) long by 0.9 m (3.0 ft) wide by 0.7 m (2.4 ft) high. The models were fabricated using fresh pine lumber with tenons at beam-column joints and dovetails at column-cross member joints. The roof was constructed with round timbers scaled to a diameter of 22.5 mm (7/8 in) at 75 mm (3 in) on center. To simulate the weight of mud plaster and straw thatches typically for the roof of these houses, 25 mm (1 in) thick steel plates were used to produce a total roof mass of 930 kg (2050 lbs). The ancient Korean commoner’s houses were typically supported on rock foundation blocks at the columns. For the experiment, a ball bearing was used to create a hinge condition at the bottom of each column. Each model was instrumented with nine accelerometers and six Linear Variable Displacement Transformers (LVDTs) to measure the response at the top of two corner columns and the center column.

The two models were tested on a 4 m (13.1 ft) wide by 4 m (13.1 ft) long six degree-of-freedom shake table. The electro-hydraulic servo system controlled shake table was capable of producing maximum horizontal accelerations of 1.5 g and maximum vertical accelerations of 1.0 g with a maximum specimen mass of 30,000 kg (66.1 k). In this experiment, the first model was subjected to the Nahanni Earthquake recorded at a rock site in eastern Canada in 1985. The second model was subjected to the Imperial Valley Earthquake of October 1979 recorded at a soft soil site in the western United States. Two horizontal components and one vertical component of the
earthquake ground motion were used in the shake table testing of both models. For the testing of Model 1 with the rock site ground motion, the peak horizontal ground acceleration was increased from 0.1 g to 0.6 g in increments of 0.1 g. For the testing of Model 2 with the soft soil site ground motion, the peak horizontal ground acceleration was increased from 0.05 g until failure in increments of 0.05 g. The peak vertical ground acceleration was scaled to two-thirds of the peak horizontal ground acceleration for each of the tests. In addition to exciting the two models with the earthquake ground motions, random white noise was input on the shake table to measure the natural frequency and damping of the models.

From the random white noise tests, the natural frequencies of Model 1 were 3.32 Hz and 3.52 Hz in the longitudinal and transverse directions, respectively. The natural frequencies of Model 2 were 3.32 Hz and 4.29 Hz in the longitudinal and transverse directions, respectively. Since both models were one-fourth scale, a prototype structure would have natural frequencies one-half of those recorded in the models. The researchers attributed the difference in the natural frequencies between the two models in the transverse direction to the method of fabrication of the models and carpenter skill. The modal damping ratio of both models was seven percent in both directions.

The first model was tested up to a peak horizontal ground acceleration of 0.6 g with a ground motion from a rock site. The researchers measured the peak acceleration of the model significantly less than the input peak acceleration of the ground motion, especially at higher levels of the peak ground acceleration. The peak vertical acceleration of the structure was slightly more than the input peak vertical acceleration of the ground motion. Acceleration and displacement responses of the structure in the longitudinal direction were larger than the transverse direction due to differences in energy content of the ground motion. The second model was tested up to a peak horizontal ground acceleration of 0.25 g with the ground motion from a soft soil site. The trends of the acceleration and displacement responses were similar to those found in Model 1. However, Model 2 failed at a lower peak horizontal ground acceleration of 0.25 g in the longitudinal direction. The researchers noted that peak ground acceleration at failure might have been low due to cumulative damage effects during the incremental testing of the models.

The experimental results of both models were compared to non-linear dynamic analyses that used the modified Double-Target model. The researchers obtained analytical model characteristics such as the stiffness and damping ratio from the random white noise tests. The two models were idealized as single-degree-of-freedom structures with lumped masses. For Model 1, the analytical model was run using an input peak horizontal ground acceleration of 0.6 g in both horizontal directions. The analytical model predicted smaller accelerations and displacements than the experimental test model. The researchers believe this may have been a result of not using the vertical component of the ground motion in the analytical model or cumulative damage effects in the experimental model. For Model 2, the analytical model was run using a input peak horizontal ground acceleration of 0.08 g in both horizontal directions. The analytical model predicted smaller accelerations and larger peak displacements. The researchers believe this discrepancy may have been a result of the stiffness of the transverse frames used in the analytical model.
Overall, the modified Double-Target model was able to simulate the non-linear dynamic response of a post and beam house with tenon joints.

**Dynamic Performance of Wooden Bearing Walls By Shaking Table Test**
*By Nobuyoshi Yamaguchi and Chikahiro Minowa*

Following the 1995 Kobe Earthquake, Yamaguchi et al. (1998, pp. 26-33) tested timber shear walls using a shake table. The researchers tested only bearing walls because of ease in changing the walls as compared to changing a whole structure during phases of testing. They wanted to compare dynamic hysteresis loops developed in this experiment with previously developed static hysteresis loops of timber shear walls. Since most analytical computer models use static hysteresis loops for shear walls, it was important to know the relation between static and dynamic hysteresis loops. In addition, the researchers performed a collapse analysis using conservation of energy.

The test specimens were 3.64 m (12 ft) long by 2.94 m (9.6 ft) high with a 1.82 m (6 ft) wide opening at the center of the wall. The bearing walls were framed using a continuous beam across the top of the wall and a continuous sill at the base of the wall. Four 105 mm (4.1 in) by 105 mm (4.1 in) columns supported the header beam and were connected to the beam and sill using vertical bolts. The wall was sheathed with 9 mm (3/8 in) thick plywood sheets 0.91 m (3 ft) wide by 2.73 m (9 ft) tall. The sheathing was nailed to the framing using N50 nails (2.7 mm (0.106 in) diameter by 50 mm (1.97 in) long) spaced at 150 mm (5.9 in). The shear walls were placed in a moving steel frame that prevented the shear walls from taking any vertical load yet still allowed the walls to resist lateral inertia loads from weights on the moving steel frame.

A 15 m (49.2 ft) long by 14.5 m (47.6 ft) wide earthquake simulator at the National Institute of Earth Science and Disaster Prevention in Japan was used to excite the shear walls with the Japan Meteorological Agency (JMA) Kobe North-South ground motion. Three different specimens with seismic shear coefficients of 0.3, 0.4, and 0.5 were used for testing. The seismic shear coefficient is defined as the lateral force in a wall when the tilting angle of the wall is 1/120 rad divided by weight on the wall. Servo accelerometers were used to measure accelerations at the shake table and at the top of the wall. In addition, laser displacement meters were used to measure displacements at the top of the walls.

During the testing of the three specimens with the JMA Kobe ground motion, the shear walls with seismic shear coefficients of 0.3 and 0.4 collapsed during testing. Force-deformation relationships were developed for each of the three shear walls and then compared to the static monotonic force-deformation relationships developed previously by Kawai. For the first specimen with a seismic shear coefficient of 0.3, the dynamic hysteresis matched well with the static hysteresis until a tilting angle of about 1/120 rad. However, after a tilting angle of 1/120 rad, the tilting angle of the static hysteresis increased rapidly due to creep effects in the timber. The maximum strength of the shear wall during the dynamic test was 114% of the maximum strength during the static test. In addition, the displacement at the maximum strength for the dynamic test was 50% of the displacement at the maximum strength for the static test. A static
yield point was not observed on the dynamic hysteresis, but a dynamic yield point was observed just prior to the maximum strength on the dynamic hysteresis. From these results, the researchers concluded that a shear wall subjected to dynamic loads has more strength but less ductility than a shear wall subjected to static loads.

Since the shear wall with a seismic shear coefficient of 0.4 collapsed and the shear wall with a seismic shear coefficient of 0.5 did not collapsed, this type of shear wall will collapse with this ground motion with a seismic shear coefficient between 0.4 and 0.5. The researchers proposed a method for predicting the collapse of the shear walls based on the experimental data and conservation of energy. This method first involves determining the equivalent natural period of a shear wall based on the elastic stiffness. Using a response spectrum for the ground motion, the spectral displacement and the spectra acceleration could be determined using the equivalent natural period. The energy required to cause collapse could be calculated using $E_d = \frac{1}{2} \cdot m \cdot S_a \cdot S_d$. The potential energy at collapse can be determined from one dynamic hysteretic loops since potential energy is not dependent on the seismic shear coefficient. If the potential energy is more than the required energy, the shear wall is predicted to not collapse. If the potential energy is less than the required energy, the shear wall is predicted to collapse. This method was used to test three additional shear walls using the 1940 El Centro ground motion. All three of these tests accurately predicted whether the shear walls would collapse.
References


Task 1.3.1- Loading Protocol and Loading Rate Effects

Literature Review

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Introduction

A total of six publications were reviewed for the effect of loading protocols and loading rate on the cyclic response of woodframe shear walls. Before the review is presented in Section 3, descriptions of available loading protocols are presented in Section 2. Because some terms used to describe the loading of structural specimens can carry loose definitions, descriptions of common terms used are given in Table 1. This should serve to avoid confusion in the following review.

Table 1: Description of Common Terms

<table>
<thead>
<tr>
<th>Term</th>
<th>Speed</th>
<th>Displacement History</th>
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<tbody>
<tr>
<td>Monotonic</td>
<td>-</td>
<td>Ramp from zero until failure; some unloading and reloading may occur</td>
</tr>
<tr>
<td>Cyclic</td>
<td>-</td>
<td>Prescribed displacement or load pattern with full reversal</td>
</tr>
<tr>
<td>Static/Quasi-static</td>
<td>Slow enough to simulate static loads</td>
<td>-</td>
</tr>
<tr>
<td>Dynamic</td>
<td>Fast enough to be representative of a typical seismic building response</td>
<td>-</td>
</tr>
<tr>
<td>Pseudo-dynamic</td>
<td>Usually slow</td>
<td>Earthquake response</td>
</tr>
</tbody>
</table>
Overview of Protocols

Several different loading protocols exist for the cyclic testing of woodframed shear walls. The following gives brief descriptions of some of the more common protocols used to test structural specimens.

ASTM E 72 (1995)

ASTM provides a standard procedure for the evaluation of sheathing panels used in woodframed shearwalls. In the specification the specimen is loaded at a constant rate to 3.0, 7.0, and 10.5 kN with complete unloading between each load increment. After the 10.5 kN load is applied, the specimen is unloaded and then reloaded monotonically until failure.

ASTM E 564 (1995)

A standard for testing an entire woodframed shearwall as opposed to just the sheathing is given in ASTM E 564. The standard provides two loading sequences: a static test and an optional cyclic test. In the static test the specimen is preloaded to 10% of the expected ultimate load to seat the connections and then unloaded. Three increments of one-third the expected ultimate load are then applied. At each increment the specimen is unloaded before application of the next higher load increment.

In the optional cyclic sequence the 10% static preload is applied and removed. A load increment of one-third the expected ultimate load is then applied and removed, followed by an equal magnitude load in the reverse direction. The reversed load is then released to form a complete cycle. Five cycles are completed at each load increment until failure of the specimen. ASTM states, “The duration of load application at each increment shall be sufficient to permit load and deflection readings to be recorded.” This statement implies that the optional cyclic procedure is a quasi-static test.
**SPD Protocol (1987)**

Originally developed by the Technical Coordinating Committee on Masonry Research (TCCMAR), the Sequential Phased Displacement (SPD) protocol has been modified and adopted by the Structural Engineers Association of Southern California (SEAOSC). The protocol is based on what is called the First Major Event (FME), which can generally be considered as the displacement corresponding to the yield state of the specimen. Prior monotonic testing must be performed to determine the FME for the cyclic protocol. The displacement history is composed of groups of stabilization and degradation cycles that are repeated at higher amplitudes as shown in Figure 1.

![Figure 1: SPD Protocol Displacement History](image-url)

The ATC-24 protocol was developed by Krawinkler for the testing of steel components. This protocol was widely used for the cyclic testing of steel moment connections after the 1994 Northridge Earthquake. The ATC-24 protocol was soon replaced, however, by the SAC protocol for Phase II of the SAC study.

The displacement history prescribed under the ATC-24 protocol is shown in Figure 2. The loading sequence is controlled by $\delta_y$, where $\delta_y$ is the yield displacement of the test specimen. Like the FME for the SPD protocol, $\delta_y$ needs to be determined prior to testing. The value of $\delta_y$ may vary for a given specimen, depending on the analytical tools, assumptions, and who performs the analysis.

![Figure 2: ATC-24 Protocol Displacement History](image)
SAC Protocol (1997)

Realizing the shortcomings of the ATC-24 protocol, Krawinkler developed for SAC two loading protocols for the cyclic testing of steel moment connections: a standard loading protocol and a near-fault loading protocol (see Figure 3). These loading protocols are based on the inter-story drift angle, which is defined as the inter-story lateral drift divided by the story height. A major advantage of the SAC protocol is that no prior testing is required to obtain parameters ($\delta_y$, FME, etc.) necessary for characterization of the protocol.

![a) Standard Loading Protocol](image)

![b) Near-fault Loading Protocol](image)

Figure 3: SAC Protocols Displacement History

Working Group 7 of the ISO Technical Committee on Timber Structures developed the ISO protocol. Originally developed for joint testing, the procedure is considered appropriate for the testing woodframed shear walls. The load history is based on the displacement at ultimate load ($u$), where the amplitude of each cycle is a percentage of $u$. A schedule of the displacement history is given in Figure 4.

Figure 4: ISO Protocol Displacement History
CEN Protocols (1995)

Short Protocol

The CEN short protocol consists of three equal amplitude cycles followed by a constant ramp load until failure. The amplitude of the three initial cycles is found by multiplying the yield slip ($\Delta_{\text{yield}}$) by an assumed ductility (D). An initial monotonic test must be performed to determine the yield slip, which is based on the intersection of two tangent lines on the load-deformation curve. An example of a $\Delta_{\text{yield}}$ calculation and a typical displacement history for the CEN Short protocol is given in Figure 5.

![Diagram of CEN Short Protocol](image)

Figure 5: Calculation of $\Delta_{\text{yield}}$ and Typical Displacement History for CEN Short Protocol
Long Protocol

The CEN long protocol consists of three cycle groups with three equal magnitude cycles per group. The third cycle group is followed by a constant ramp load until failure. The amplitude of the first cycle group is equal to 35% of the maximum load displacement followed by 50% and 80% for the second and third cycle groups respectively. The displacement at maximum load must be found from previous monotonic testing. A plot of a typical displacement history is given in Figure 6.

![Figure 6: Typical Displacement History for CEN Long Protocol](image-url)

The FCC protocol, developed by the Forintek Canada Corporation, consists of sinusoidal cycle groups of three equal magnitude cycles. Cycle group amplitude is determined from the nominal yield slip ($\Delta_{\text{yield}}$) defined as half the ultimate load displacement and found from prior monotonic testing. The first cycle group amplitude is equal to 50% of $\Delta_{\text{yield}}$ followed by 100% of $\Delta_{\text{yield}}$ and then 50% again for the third cycle group. A similar pattern is repeated in subsequent cycle groups as shown in Figure 7, until specimen failure is reached.

![Figure 7: Displacement History for FCC Protocol](image)
Literature Review

Comparison of Static and Dynamic Response of Timber Shear Walls
David W. Dinehart and Harry W. Shenton III (1998)

Introduction

Research performed by Dinehart and Shenton investigates the resistance of woodframe shearwalls subjected to static and dynamic racking loads. Their study compared the response of identical shearwalls to the loading procedures using the same test facility. Of interest was the dynamic response of a standard shearwall, and a comparison of the differences in stiffness, ductility, ultimate load, and failure mechanism that occurred between the loading procedures. Prior to this research, no studies had been conducted comparing the differences between standard 2.4 m by 2.4 m (8 ft by 8 ft) shearwalls subjected to both static and dynamic loads. The standard size is prescribed in ASTM E 72.

Test Specimens

A total of twelve walls were tested, four static tests and eight dynamic tests. Half of the specimens were sheathed with 11.9 mm (15/32 in) thick plywood and the other half with 12.7 mm (1/2 in) thick OSB. Two 1.22 m by 2.44 m (4 ft by 8ft) sheathing panels were used per wall oriented vertically and fastened with 8d nails. All wall components and construction other than sheathing were consistent between the walls. Standard 2 by 4 studs made from No. 2 Spruce-Pine-Fur with dimensions of 38 mm by 89 mm (1.5 in by 3.5 in) were used for the framing. The studs were spaced at 406 mm (16 in) on center with double end studs, double top plates, and a single sill plate.

Test Setup

The test apparatus consisted of a self-reacting steel frame made from channels, angles, and wide flange sections welded or bolted together. Roller bearing guides were used at the top of the wall in order to prevent lateral movement of the specimen during testing. The wall was anchored by placing a 254 mm by 76 mm by 13 mm (10 in by 3 in by 1/2 in) steel plate on top of the sill plate and fastening it to the test frame with 19 mm (3/4 in) bolts. This shear anchorage was designed to prevent any movement of the base during testing. In addition, commercially available hold-downs were fastened to the ends of the wall and bolted to the test frame. The actuator used to load the wall was bolted to the test apparatus at wall height and load was applied to the specimen via a steel member fastened to the top plate.
Test Procedure

The protocol prescribed in ASTM E 564 was used for the static testing with the exception that higher test loads were used. The higher loads were used to ensure that the ultimate load was reached in the third stage, which is consistent with the procedure used by the American Plywood Association. Two walls were tested with plywood and two with OSB. The dynamic testing was performed using the SPD protocol at a frequency of 1 hertz, except for the final seven cycles, which were performed at 0.5 hertz due to equipment limitations. The FME chosen utilized the full ±76 mm (±3 in) stroke of the actuator and corresponded to wall drift of 0.75%. Three of the dynamic tests were conducted with plywood and three with OSB.

Results

Different failure mechanisms were observed between the static and dynamic tests. In the static tests the sheathing pulled away from the framing along the edges, pulling the nails with it. Sill plate splitting parallel to the grain occurred at the uplift corner of the wall. Some interior studs were observed to twist along their length and split. In the dynamic tests most of the damage occurred in the sheathing fasteners. The nails either fatigued and broke off or pulled out from the framing and sheathing. Different from the static tests, the sheathing remained secured against the frame when nail pullout occurred.

The load-deformation responses of the plywood and OSB sheathed specimens behaved similarly in the static tests. The response of the plywood was similar to that of the OSB in the dynamic tests, although the average ultimate load of the plywood walls was slightly higher than that in the OSB walls. Additionally, the ultimate load in the plywood wall occurred at the 52nd cycle as opposed to the 45th cycle in the OSB wall. Quantities obtained from the testing and typical plots containing positive values of the load histories are given in Table 2 and Figure 8.

<table>
<thead>
<tr>
<th>Loading Procedure</th>
<th>Sheathing</th>
<th>Average ultimate load (kN)</th>
<th>Avg. displacement at ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static</td>
<td>Plywood</td>
<td>33.38</td>
<td>84</td>
</tr>
<tr>
<td></td>
<td>OSB</td>
<td>32.04</td>
<td>79</td>
</tr>
<tr>
<td>Dynamic</td>
<td>Plywood</td>
<td>31.60</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>OSB</td>
<td>28.04</td>
<td>46</td>
</tr>
</tbody>
</table>

Table 2: Data from Tests by Dinehart and Shenton
Conclusions

Ultimate loads produced in the static and dynamic tests were comparable, with the static results slightly higher. In the dynamic tests however, the displacements at which ultimate loads occurred were significantly lower. Also, the stiffness and ultimate load of the specimens were shown to degrade as more loading cycles were imposed in the dynamic tests. Assuming the same yield displacement of 6 mm, a conclusion was made that the static specimens had a higher ductility. The ductility was calculated to be 34% larger in the plywood walls and 42% higher in the OSB walls. It was not clear whether this difference was due to the effects of cyclic loading or the effects of loading rate, since the dynamically tested walls were loaded at a much higher rate.
Introduction

Research performed by the Army Corps of Engineers at the Construction Engineering Research Laboratory compared plywood sheathed shear walls with a proprietary integrated wood truss design proposed by the Building Science Corporation. A monotonic and two cyclic loading protocols (SPD and Modified SAC) were used for testing the wall configurations. The review here is focused on the results from testing of the plywood-sheathed walls.

Test Specimens

There were three different configurations for the plywood-sheathed walls. All of the wall specimens were approximately 8 ft by 8 ft square with a single 4 ft by 8 ft sheet of 1/2 in STR 1 plywood fastened near the center of the wall to provide lateral resistance. Each wall was anchored by fastening the bottom plate with 7/8 in A325 bolts and 1/2 in steel plates. Key elements of the different specimens are given in Table 3.

<table>
<thead>
<tr>
<th>Wall</th>
<th>Configuration</th>
</tr>
</thead>
</table>
| 1    | • Double 2x4 top plate and single 2x4 bottom plate  
      | • 2x4 studs at 16” O.C.  
      | • 8d edge nails at 4” O.C. and 8d field nails at 8” O.C.  
      | • No holdowns |
| 2    | • Single 2x6 top and bottom plates  
      | • 2x6 studs at 24” O.C.  
      | • 8d edge nails at 4” O.C. and 8d field nails at 8” O.C.  
      | • No holdowns |
| 3    | • Single 2x6 top plate with 4x6 blocking attached and single 3x6 bottom plate  
      | • 2x6 studs at 24” O.C. with 4x6 studs at plywood edges  
      | • 10d edge nails at 3” O.C. and 10d field nails at 8” O.C.  
      | • Simpson HD8A holdowns placed at top and bottom of 4x6 edge studs |

Test Setup

The test setup consisted of a steel box test frame oriented symmetrically around the specimen and bolted to a strong floor. Stub columns with teflon sliders attached to the frame prevented out of plane motion of the specimen. Vertical actuators attached to the frame provided vertical load and ensured the top plate remained horizontal. A 40 in stroke actuator bolted to a nearby strong wall at specimen height provided the lateral load through a steel load distribution beam.
Test Procedure

Monotonic testing was performed by loading the specimen at a rate of 1/2 in per second until complete wall failure or until 8 in of displacement was reached. The cyclic tests were performed using an FME displacement amplitude of 0.4 in. A loading frequency of 0.1 hertz was used in the SPD tests, and the SAC tests used a constant loading rate of 12 in per minute. Initial monotonic testing indicated that the presence of vertical load increased shear wall capacity due to the resistance of overturning moment. Consequently, in subsequent monotonic tests, the vertical load was removed to create the most critical condition. No vertical loads were applied to the specimens tested with the SPD protocol. A vertical load of 8000 lb was applied to specimens tested under the SAC protocol.

Results

A common failure mechanism in the #1 and #2 walls was nails tearing through the sheathing at the top and bottom plate edges. This was observed in both the monotonic and cyclic testing protocols and is likely due to the lack of overturning resistance causing the nail loads to be concentrated at the top and bottom plates. Another contributing factor was the small edge distances of the nails due to the 2x top and bottom plates and edge studs. In the monotonic and SAC tests for the #3 walls, the nails tended to fail in bending and then pull through the plywood while remaining embedded in the framing. In the SPD tests, unlike the other protocols, the nails tended to fail in shear. This may be a result of the large number of cycles imposed after the specimen has reached its yield state. The difference in failure mechanisms between the wall configurations can be attributed to the use of holdowns in the #3 wall in addition to larger members at the sheathing edges providing for larger edge nail distances.

Conclusions

Since the primary objective of the research was a comparison of standard shear wall systems with a newly designed truss system, a comparison of the different sheathing configurations and loading protocols was not emphasized. However, there were some useful conclusions made by the author worth noting.

- Wall capacity and ductility is severely reduced in the absence of vertical loads and/or holdowns (no overturning resistance).
- Cyclic loading reduces wall ductility.
- The SPD protocol produces consistently lower wall capacities than the SAC protocol (27% lower in the #3 wall)
- The SPD protocol produces consistently lower ductility than the SAC protocol.
- Much higher (about 4 times) shear strengths were obtained from the #3 wall. The increased strength is attributed to the addition of holdowns and larger edge nailing distances.
Nailed Wood-Frame Shear Walls for Seismic Loads
Test Results and Design Considerations
Erol Karacabeyli and Ario Ceccotti (1998)

Introduction

Research by Karacabeyli and Ceccotti investigated the effects of different loading protocols on the performance of plywood shearwalls. They subjected shearwalls of the same construction to various loading schedules and compared the respective responses. Some quantities under consideration were the ultimate load capacity, the displacement at ultimate load, and the dissipated energy.

Test Specimens

Shear walls used in the testing were 4.88 m wide by 2.44 m high and used 2 by 4 Spruce-Pine-Fur dimension lumber for all of the framing members. Wall studs were positioned at 406mm on center and blocking was placed at mid height. Double top plates and single bottom plates were used and anchored with 12.5 mm bolts at both the top and bottom. An actuator applied the racking load through a distribution beam attached to the top plate and a mass of 16000 kg was used to apply a constant vertical load.

Test Procedure

Protocols used to specify the actuator motion were as follows:

- Monotonic loading with a displacement rate of 0.13 mm/sec
- SPD using a constant frequency of 0.5 hertz
- CEN Long Cyclic Procedure using a constant velocity of 0.42 mm/sec
- CEN Short Cyclic Procedure using a constant velocity of 0.42 mm/sec
- FCC Cyclic Procedure using a constant frequency of 0.5 hertz
- ISO Cyclic Procedure using a constant velocity of 20 mm/sec
- Pseudo-dynamic tests using a peak displacement velocity of 20 mm/sec

Results

Different failure mechanisms were observed depending on the loading protocol used. Four failure modes were prevalent.

1. Shear fatigue failure of the nails.
2. Nails pulling through the sheathing.
4. Nails tearing out of the edges of the sheathing.
The only tests where nail fatigue failure was observed were under the SPD and FCC protocols. This was attributed to much higher energy demand occurring under these protocols as compared with the others. The other protocols produced a mix of the other three failure modes.

Figure 9 gives values obtained from the testing. Figures 10, 11, and 12 are plots of the SPD, ISO, and CEN Long test results superimposed on the monotonic test curve. The SPD protocol is denoted as ASTM in the test results because SPD was being considered for adoption by ASTM.

<table>
<thead>
<tr>
<th>Protocol</th>
<th>F_{max}</th>
<th>F_{max}</th>
<th>Unit</th>
<th>Energy</th>
<th>Energy</th>
<th>Maximum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kN/m</td>
<td>kip/ft</td>
<td>mm</td>
<td>in</td>
<td>kN.m</td>
<td>ft.Ed</td>
<td>m/s</td>
</tr>
<tr>
<td>ASTM 37-01 1st cycle</td>
<td>8.55</td>
<td>0.601</td>
<td>1.00</td>
<td>47</td>
<td>1.85</td>
<td>1.60</td>
<td>21.10</td>
</tr>
<tr>
<td>ASTM 37-01 3rd cycle</td>
<td>7.25</td>
<td>0.547</td>
<td>0.81</td>
<td>45</td>
<td>1.78</td>
<td>0.96</td>
<td>-</td>
</tr>
<tr>
<td>ASTM 37-02 1st cycle</td>
<td>8.18</td>
<td>0.550</td>
<td>1.00</td>
<td>48</td>
<td>1.88</td>
<td>1.00</td>
<td>20.70</td>
</tr>
<tr>
<td>ASTM 37-02 3rd cycle</td>
<td>6.56</td>
<td>0.441</td>
<td>0.80</td>
<td>42</td>
<td>1.66</td>
<td>0.88</td>
<td>-</td>
</tr>
<tr>
<td>ASTM Average 1st cycle</td>
<td>7.56</td>
<td>0.575</td>
<td>1.00</td>
<td>47</td>
<td>1.86</td>
<td>1.00</td>
<td>21.00</td>
</tr>
<tr>
<td>ASTM Average 3rd cycle</td>
<td>6.90</td>
<td>0.464</td>
<td>0.81</td>
<td>44</td>
<td>1.72</td>
<td>0.92</td>
<td>-</td>
</tr>
<tr>
<td>FCC 37-31 1st cycle</td>
<td>7.86</td>
<td>0.528</td>
<td>0.92</td>
<td>50</td>
<td>1.95</td>
<td>1.65</td>
<td>19.40</td>
</tr>
<tr>
<td>ISO 97-07 1st cycle</td>
<td>8.67</td>
<td>0.583</td>
<td>1.01</td>
<td>52</td>
<td>2.04</td>
<td>1.09</td>
<td>19.30</td>
</tr>
<tr>
<td>CEN-LONG 42-02 1st cycle</td>
<td>8.23</td>
<td>0.520</td>
<td>0.97</td>
<td>55</td>
<td>2.18</td>
<td>1.17</td>
<td>9.50</td>
</tr>
<tr>
<td>CEN-SHORT 42-04 1st cycle</td>
<td>8.55</td>
<td>0.575</td>
<td>1.00</td>
<td>&gt;48</td>
<td>1.91</td>
<td>&gt;1.02</td>
<td>8.60</td>
</tr>
<tr>
<td>HOSTODYNAM 4.00 42-02 Wall 73-02</td>
<td>10.24</td>
<td>0.688</td>
<td>1.20</td>
<td>&gt;54</td>
<td>2.13</td>
<td>&gt;1.14</td>
<td>7.10</td>
</tr>
<tr>
<td>STATIC 31-01</td>
<td>8.40</td>
<td>0.564</td>
<td>0.98</td>
<td>76</td>
<td>2.98</td>
<td>1.60</td>
<td>1.80</td>
</tr>
</tbody>
</table>

Notes: Dimensions: 2.4 m (8 ft) in height and 4.9 m (16 ft) long
Test configuration: ASTM E564 with vertical loads of 2.27 kN/m (0.153 kip/ft)
Placing: No. 1 & 2 6.35 mm (0.250 inch) Steel plates, 600 0.418 mm (0.025 inch) Steel plates
Nails: 3 mm (0.12 inch) diameter x 65 mm (2.5 inch) power driven spaced at 15 cm (6 inch) at perimeter and 30 cm (12 inch) on intermediate supports

1 0.5 mm (3/32 inch) Canadian Softwood Plywood (CSP) panels positioned horizontally with continuous blocking
2 0.5 mm (3/32 inch) Canadian Softwood Plywood (CSP) panels positioned vertically without blocking

Figure 9: Karacabeylik and Cecotti Test Results

Figure 10: Hysteresis Curve for SPD Test
Conclusions

All of the test protocols produced roughly the same maximum load capacity (within ±10%). Tests performed under the SPD protocol produced ultimate load displacements that were smaller than those produced with the other protocols (about 40% less than the monotonic displacement). Energy demands in the SPD tests were significantly higher than in the other tests, which is a likely cause of the reduced displacement at ultimate load. Pseudo-dynamic testing produced ultimate loads and displacements that were consistently higher than those produced under various protocols including the monotonic. The authors provided this as evidence that monotonic testing can be conservatively used to determine the maximum design capacity. They also pointed out that the first envelope of the hysteresis loops as opposed to the commonly used third envelope could be conservatively used to obtain the design capacity.
Dynamic Performance of Wooden Bearing Walls by Shaking Table Test
Nobuyoshi Yamaguchi and Chikahiro Minowa (1998)

Introduction

Research by Yamaguchi and Minowa involved dynamic testing of woodframe shear walls by means of a shake table. The testing, motivated by the 1995 Kobe Earthquake, was carried out in order to determine the dynamic performance of wood shear walls and to compare the shake table test results with previously performed quasi-static tests. In addition, a mathematical model was developed to predict collapse of the shear walls under dynamic loading, which was compared with results from the dynamic testing. The discussion here involves only the test results.

Test Specimens

Each shear wall specimen consisted of essentially two plywood-sheathed shear walls separated by an 1820 mm opening and connected by the top and bottom plates. The resulting system could be considered to act as a shear wall frame or a perforated shear wall. Plywood sheathing fastened flush with the edge of the wall had dimensions of 910 mm by 2730 mm and was 9 mm thick. N50 nails (2.7 mm diameter) spaced at 150 mm were placed at the sheathing edges. Accelerometers were placed at the base and top of the specimens and laser displacement meters were used to measure the relative displacement between the top and bottom of the specimens.

Test Setup

A steel braced frame with wooden cross walls placed perpendicular to the direction of shaking was used as the test apparatus. The cross walls were pinned to allow rotation in the direction of shaking and were used to support mass required to simulate inertial loads. Although the cross wall supported all of the vertical loads, the shear wall specimens were fastened to the cross walls such that they provided the lateral load resistance. The apparatus was placed on a shake table capable of simulating Kobe Earthquake ground motions.

Procedure

Tests were performed and hysteresis loops created relating wall load to the tilting angle (inter-story drift angle) of the wall for specimens with shear coefficients of 0.3, 0.4, and 0.5. The shear coefficient was defined as the allowable lateral wall load divided by the inertial weight \(Co = P_{allow} / W\) and was adjusted by changing the inertial weight. The allowable lateral load is defined as the wall strength when the tilting angle is 1/120 radian. Results from previous quasi-static tests on walls of the same construction were plotted with the dynamic hysteresis loops as a means of comparison.
Results

Common failure mechanisms observed were the nails pulling out of the framing and in some cases, nails punching through the sheathing. In either case the sheathing tended to pull away from the framing at large deformations.

One of the objectives of the testing was to determine the shear coefficient required to resist a Kobe magnitude earthquake. Shear coefficients of 0.3 and 0.4 resulted in collapse of the specimens, collapse being defined as the point on the hysteresis loop where the wall strength degrades to half of its ultimate strength. With a shear coefficient of 0.5 this collapse condition was never met, indicating that a shear coefficient between 0.4 and 0.5 is required to resist a Kobe magnitude earthquake.

Conclusions

A comparison of the quasi-static and dynamic testing showed a trend of higher ultimate strength and lower ductility for specimens under dynamic loading. For the specimen with a shear coefficient of 0.3, the ultimate strength in the dynamic test was found to be 114% of that in the quasi-static test. The displacement at ultimate load in the dynamic test was found to be 50% of that in the quasi-static test. It was speculated that the slow rate of loading in the quasi-static test allowed the wall to creep, which would account for the higher displacement. It was also noted that although a reasonably well defined yield point existed for both the quasi-static and dynamic tests, the points did not coincide. The dynamic yield point occurred at a higher load.
Influence of Cyclic Test Protocols on Performance of Wood-Based Shearwalls
Ming He, Frank Lam, and Helmut G.L. Prion (1998)

Introduction

Researchers at the University of British Columbia investigated the effects of three commonly used protocols on the performance of woodframed shear walls. The authors were interested in how test results are affected by the use of different loading protocols, and which protocols produce results that best reflect those observed in shake table testing and in actual earthquakes. They were also interested in the effects of different panel sizes and nailing configurations on shear wall performance.

Five walls were tested using three different loading protocols. The protocols used were FCC, CEN Short, and CEN Long. Three walls of the same construction were used to evaluate the three different protocols. Two additional walls were constructed using different panel sizes and nailing configurations and tested under the FCC protocol to evaluate the effects of these parameter changes. The discussion here looks only at the three walls tested under the three different protocols.

Test Specimens

Test walls were 7.2 m long by 2.4 m high and used standard North American framing with blocking at mid-height. Wall studs were 38 mm by 89 mm and placed 400 mm on center. Double end studs and top plates were used with single bottom plates. The framing was connected with air-driven 76 mm common nails. Each wall used a single oversized 9.5 mm OSB sheet that completely covered the framing. The sheathing was fastened using 50 mm spiral nails placed at 76 mm on center at the edges, and 305 mm on center at the interior studs.

Test Setup

An actuator bolted to a steel reaction frame at wall height was used to load the specimen. The load was applied through a steel distribution beam bolted to the test wall. Steel rods attached to hydraulic jacks were spaced at 2.4 m on center and applied a vertical load to the load distribution beam. This setup created a constant vertical load of 9.12 kN/m and was used to simulate vertical dead loads. Displacement values were measured with reference to the foundation in order to eliminate any effects of flexibility in the reaction frame.
Procedure

Previous monotonic tests performed by Lam et al. on walls of the same construction as the walls here were used to obtain parameters (ultimate load, yield displacement, etc.) necessary to characterize the protocols. An initial cyclic frequency of 0.25 hertz was used in the FCC protocol until the later stages of testing where the frequency was changed to 0.125 hertz. A constant velocity of 0.4 mm/s was used in both the CEN Short and Long protocols. The wall tested under the FCC protocol was subjected to 22 cycle groups.

Results

Ultimate loads obtained using the different protocols are given in Table 4 along with the respective displacements.

<table>
<thead>
<tr>
<th>Loading Protocol</th>
<th>Ultimate Load (kN)</th>
<th>Displacement at ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FCC</td>
<td>101.54</td>
<td>21.46</td>
</tr>
<tr>
<td>CEN short</td>
<td>112.94</td>
<td>23.38</td>
</tr>
<tr>
<td>CEN long</td>
<td>115.33</td>
<td>31.95</td>
</tr>
</tbody>
</table>

The smaller ultimate loads and displacements in the FCC protocol are likely a result of nail fatigue that occurred when testing under this protocol. No nail fatigue was observed in the CEN protocols where failure was a result of nails pulling out of the framing and nails pulling through the sheathing. The ultimate load in the CEN Short protocol occurred in the first cycle of testing and was never reached again. This prompted the authors to conclude that permanent damage had occurred in the first cycle and to question whether subsequent cycles produced meaningful results. The ultimate load in the CEN Long protocol occurred during the monotonic stage.

Hysteresis curves given in Figures 13, 14, and 15 allowed the calculation of strength degradation and energy dissipation for each specimen. Results from prior monotonic tests are plotted with the curves for comparison purposes. Strength degradation was the greatest between the first and second equal magnitude cycle and was minimal in the third cycle. The authors noted that this agrees with results reported by Daneff et al. and also cited this as an indication that three equal magnitude cycles are adequate for strength stabilization within a cycle group. Energy dissipation was highest using the FCC protocol, which was a result of the large number of high amplitude cycles used in this protocol. Although energy dissipation was lower in the CEN protocols, it was still well above that observed in shake table testing of specimens under simulated earthquake loads.
Figure 13: Hysteresis Curve for FCC Test

Figure 14: Hysteresis Curve for CEN Short Test
Conclusions

A comparison of the three different protocols indicates the CEN Long protocol produces results that best represent the behavior of an actual structure under earthquake loading. The FCC protocol produced a failure mechanism inconsistent with that observed in actual earthquake loading. Additionally, the large number of cycles produced unrealistic energy dissipation. Permanent damaged that occurred in the first cycle of the CEN Short test created a lack of confidence in results obtained from subsequent cycles. The CEN Long test produced a realistic failure mechanism and had the highest ultimate load, however, the energy dissipation was still high when compared with actual earthquake loading. The authors concluded that although the CEN Long protocol produced the most realistic results, improvements are still required to emulate an actual earthquake response.
Introduction

Researchers at the University of California, Irvine performed shear wall testing on standard shear walls using three variations of the SPD protocol. The study was prompted by the lack of a standard procedure for the cyclic testing of woodframed shear walls. Researchers were interested in identifying the effects of different loading sequences on the performance of woodframed shear walls.

Test Specimens

Twenty-four standard 2.44 m by 2.44 m shear walls were constructed using Douglas fir dimension lumber. Standard 51 mm by 102 mm framing members were used with a single bottom plate and double top plates. Double studs were used at panel edges and 102 mm by 102 mm posts were used at the wall ends. Sheathing was 9.5 mm thick CDX plywood and oriented vertically with 3 mm spacing between panel edges. Bolted steel holdowns were used at the end studs and four 16 mm bolts with steel plate washers were used to anchor the bottom plates. Four different nail types were used to fasten the sheathing to the framing for a total of six walls with each nail type. The nail types were: 8d hand driven common nails, 8d hand driven galvanized box nails, 8d pneumatically driven common nails, and 8d pneumatically driven box nails.

Test Setup

The test apparatus conformed to standards outlined by ASTM E 564. A steel base plate thin enough to allow full rotation of the sheathing was bolted to the strong floor and used to secure the specimens. Racking load was applied through a steel distribution beam lag bolted to the top plate. The load distribution beam was attached to a vertical steel column pin connected to the strong floor to allow rotation in the direction of loading. A ±76 mm actuator bolted to a strong wall and placed at mid-height of the vertical beam was used to apply the load. Because the top of the vertical beam was attached to the horizontal loading beam, this setup allowed the limited stroke of the actuator to be amplified.
Test Procedure

Two walls of each nail type were tested with one of three variations of the SPD protocol. The first sequence used the loading protocol defined by the SPD standard. The second sequence followed the SPD standard except that the tests began at a displacement of 200% of the FME and continued as specified from there. This was intended to simulate the large excursions that are indicative of near-field events. The third sequence followed the SPD standard except that the three equal magnitude cycles following the decay cycles were removed. These cycles were removed in an attempt to minimize nail fatigue failure. The testing was performed at a cyclic frequency ranging from 0.25 to 0.5 hertz until a displacement of 40 mm was reached, at which point the frequency was reduced to obtain a constant maximum velocity.

Results

Typical failure mechanisms observed were nail fracture, nails pulling through the sheathing, and nail withdrawal from the framing. It was noted that common nails were more prone to pulling through the plywood, while box nails tended to fracture more frequently. No mention was given to whether a failure mode appeared more frequently in one loading sequence than another.

Results of the testing were reported in the form of hysteresis loops for each of the loading sequences and strength levels corresponding to drifts of 0.5%, 1.0%, and 1.5%. Average racking loads corresponding to the drift levels are given in Table 5.

Table 5: Data From Tests by Ficcadenti, Steiner, Pardoen, and Kazanjy

<table>
<thead>
<tr>
<th>Drift Level</th>
<th>Nail Type</th>
<th>Sequence 1</th>
<th>Sequence 2</th>
<th>Sequence 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5%</td>
<td>H/D Common</td>
<td>27.9 kN</td>
<td>27.2 kN</td>
<td>33.5 kN</td>
</tr>
<tr>
<td></td>
<td>H/D Galvanized</td>
<td>26.1 kN</td>
<td>28.3 kN</td>
<td>32.7 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Common</td>
<td>26.4 kN</td>
<td>31.6 kN</td>
<td>31.2 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Box</td>
<td>26.0 kN</td>
<td>28.6 kN</td>
<td>31.3 kN</td>
</tr>
<tr>
<td>1.0%</td>
<td>H/D Common</td>
<td>39.5 kN</td>
<td>42.1 kN</td>
<td>42.7 kN</td>
</tr>
<tr>
<td></td>
<td>H/D Galvanized</td>
<td>38.3 kN</td>
<td>42.2 kN</td>
<td>43.9 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Common</td>
<td>37.4 kN</td>
<td>45.3 kN</td>
<td>42.0 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Box</td>
<td>37.5 kN</td>
<td>42.1 kN</td>
<td>41.3 kN</td>
</tr>
<tr>
<td>1.5%</td>
<td>H/D Common</td>
<td>39.4 kN</td>
<td>47.8 kN</td>
<td>43.7 kN</td>
</tr>
<tr>
<td></td>
<td>H/D Galvanized</td>
<td>41.0 kN</td>
<td>49.4 kN</td>
<td>46.9 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Common</td>
<td>39.2 kN</td>
<td>51.4 kN</td>
<td>45.4 kN</td>
</tr>
<tr>
<td></td>
<td>P/D Box</td>
<td>40.7 kN</td>
<td>47.9 kN</td>
<td>44.9 kN</td>
</tr>
</tbody>
</table>
Conclusions

Some key observations made by the authors are given below.

- Specimens subjected to a larger number of prior inelastic cycles produced lower strength levels.
- Nail type did not significantly affect shear wall performance, however, the loading sequence did, with sequence 2 producing the highest loads followed by sequence 3.
- There was no significant influence of loading sequence on the displacement capacity of the specimens
References


Task 1.3.3 - Dynamic Characteristics of Woodframe Buildings Literature Review

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Scope of Review

The main objective of Task 1.3.3 is to determine the dynamic characteristics of wood frame buildings and to develop a period formula specific for wood structures by regressing on structural characteristics. We have reviewed some of the current formulas used to calculate the period for wood frame buildings and also for other materials and types of construction, and we propose some possible regressors to be investigated in finding a new formula for wood frame buildings. Through analysis of recorded earthquake response and by forced and ambient vibration testing, we will be developing a database of periods, damping ratios and mode shapes of wood frame buildings; in particular, this will enable us to develop a more accurate period formula for design purposes. We will be focusing on the fundamental mode behavior, which tends to dominate the dynamic characteristics. This task will also focus mainly on low-rise shear-wall buildings, as this is the predominant type of wood-frame construction in North America.

Dynamic Properties of Woodframe Structures

We are evaluating the dynamic properties of wood shear wall buildings, mainly modal parameters such as frequencies, damping and mode shapes of the structures and how these parameters change with motion amplitude. In reviewing much of the available literature, we have found that much research has been done on the dynamic and hysteretic characteristics of wood subsystems and connection panels (e.g. Cheung 1984; Falk 1986; Dolan 1989; Polensek and Schimel 1991), but that full-scale testing of wood shear-wall buildings has been sparse. We will be focusing on the behavior of the entire structure in the elastic range (smaller amplitude of vibrations), and we will compile a database of modal parameters based on the reviewed literature and also based on tests that we will perform in the course of this research.

Polensek and Schimel (1991) evaluated the degree of non-linearity and degradation of damping and stiffness properties in wood subsystems. They observed that energy was dissipated by slipping interfaces of connected materials, and that damping tends to increase with increasing
amplitude of vibration up to some limit, after which prior damage tends to reduce interface friction and therefore reduce damping and stiffness of shear wall, bending and connection panels. They also noted that the dynamic behavior of the panels was the same regardless of the lumber grade, suggesting that panel damping and stiffness depend mostly on nailed joints and less on the grade of lumber used in framing.

Seo et al. (1981) performed static and cyclic lateral load tests on wooden frames with tenon beam-column joints. The tests showed nonlinear and inelastic behavior, with estimated equivalent viscous damping ratios between 13% and 27% for these types of structures. Stiffness of frame was significantly reduced with increased amplitude of displacement.

Hirashima (1988) tested a building of post-and-beam construction, a two-story building with diagonal bracings built in post-and-beam frames. He used static loading tests to obtain spring constants to use in a mathematical model of the building and forced vibration tests to observe the dynamic behavior. He noted that the test building oscillated mainly in its fundamental mode of vibration in each direction, and that the corresponding frequency was almost constant throughout the motion at 4 Hz (transverse) and 4.5 Hz (longitudinal). The corresponding damping ratios were 2.4% (transverse) and 1.4% (longitudinal), from a free vibration test with initial peak-to-peak displacements of about 1/2 mm. An earthquake record was also obtained in the test building with a 6%g peak acceleration at the roof. A Fourier amplitude spectrum of the roof accelerations showed a fundamental frequency for each direction of about 4 Hz.

Yokel, Hsi and Somes (1973) performed full-scale tests on a two-story house of conventional woodframe construction. A series of tests were conducted to determine the dynamic response of the house to a single impulse load. The natural frequency of the structure was approximately 9 Hz and damping averaged approximately 6% of critical damping, varying from 4% to 9%. The validity of these findings is questioned by the researchers, since the resolution of the displacement time history records was marginal.

Foliente and Zacher (1994) report on dynamic tests of timber structural systems. We present a table taken from their paper (see TABLE 1), where they give a summary of periods from tests performed in several different countries. Because of the difference in construction, results from other countries may not be especially relevant, and we will be focusing on the North American tests.
## Table 1: Summary of Natural Periods and Frequency of Low-rise Wood and Wood-based Buildings

<table>
<thead>
<tr>
<th>Building Type</th>
<th>Natural Period $T_n$ (sec)</th>
<th>Natural Frequency $1/T_n$ (Hz)</th>
<th>Reference(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>One- and two-story New Zealand residential</td>
<td>0.1 to 0.6</td>
<td>1.7 to 10.0</td>
<td>Dowrick (1987)</td>
</tr>
<tr>
<td>One-story truss-frame residential</td>
<td>0.14 to 0.26</td>
<td>3.8 to 7.2</td>
<td>Gavrilović and Gramatikov (1991)</td>
</tr>
<tr>
<td>Two- and three-story N. American residential</td>
<td>0.14 to 0.32</td>
<td>3.0 to 7.0</td>
<td>Sugiyama (1984)</td>
</tr>
<tr>
<td>Two-story residential (Greece)</td>
<td>0.18 to 0.22</td>
<td>4.5 to 5.6</td>
<td>Toulia et al. (1991)</td>
</tr>
<tr>
<td>Two-story base-isolated residential</td>
<td>0.48 to 1.25</td>
<td>0.8 to 2.1</td>
<td>Sakamoto et al. (1990)</td>
</tr>
<tr>
<td>One-, one and a half-, and two-story N. American residential and school buildings</td>
<td>0.06 to 0.25</td>
<td>4.0 to 18.0</td>
<td>Soltis et al. (1981)</td>
</tr>
<tr>
<td>One-, two- and three-story Japanese residential</td>
<td>0.11 to 0.33</td>
<td>3.0 to 9.0</td>
<td>Arima et al. (1990)</td>
</tr>
<tr>
<td>Three-story Japanese residential</td>
<td>0.16 to 0.20</td>
<td>4.7 to 6.2</td>
<td>Nakajima et al. (1993)</td>
</tr>
<tr>
<td>One-, and two-story comm'l/industrial (plywood roof diaphragm &amp; conc/masonry walls)</td>
<td>0.20 to 0.80</td>
<td>1.2 to 5.1</td>
<td>Bouwkamp et al. (1994)</td>
</tr>
<tr>
<td>All (Range of Values)</td>
<td>0.06 to 0.80</td>
<td>0.8 to 18.0</td>
<td></td>
</tr>
</tbody>
</table>
Current Code Period Formulas

Current building codes require a design earthquake load based on the building’s system characteristics, site location, occupancy, etc. The code specifies simplified formulas to approximate the building’s dynamic behavior. One important factor in determining how the building will behave during an earthquake is its fundamental period. This is used, for example, to help determine the appropriate seismic base shear coefficient for the design of a structure.

Recent research has shown that the current 1997 Uniform Building Code period formulas substantially underestimates the building periods for concrete and steel moment-resisting frame buildings, as well as that for concrete shear-wall buildings (Goel and Chopra, 1997, 1998). An important objective of this task is to evaluate and improve the current code period formulas for wood structures.


The 1997 UBC prescribes the following period formula (called Method A) for buildings:

\[ T = C_t \cdot h_n^{3/4} \]  

(1)

The values of \( C_t \) vary according to material and framing characteristics:

- \( C_t = 0.035 \) (Steel Moment Resisting Frames)
- \( C_t = 0.030 \) (Reinf. Concrete Moment Resisting Frames and Eccentric Braced Frames)
- \( C_t = 0.020 \) (all other buildings)

or, for Concrete or Masonry Shear Wall Buildings, the following may be used:

\[ C_t = \frac{0.1}{A_c^{1/2}} \]  

where \( A_c = \sum A_e \left[ 0.2 + \left( \frac{D_e}{h_n} \right)^2 \right] \)  

(2)

and \( \frac{D_e}{h_n} \leq 0.9 \)

\( A_c = \) minimum cross-sectional area, in sq. feet, of the shear walls in the first story of structure.

\( D_e = \) length, in feet, of a shear wall in the first story in the direction parallel to the applied forces.

\( h_n = \) height, in feet, above the base to the uppermost level in the main portion of the structure.
The 97 UBC also allows for the period to be calculated from structural analysis, but it imposes limits on the maximum periods obtained that way to 1.3 of Method A period for Zone 4 buildings or 1.4 of Method A period for Zones 1, 2 and 3.

Other Building Codes

Many other building codes, such as the 1990 NBCC (National Building Code of Canada) and the Australian Standard, have based their period formulas on older versions of the Uniform Building Codes (see IAEE, 1992). These formulas do not differentiate between building materials or structural systems. These codes prescribe as the natural period of buildings:

$$T = 0.09 \frac{h_n}{\sqrt{L}} \quad \text{or}$$  \hspace{1cm} (3)

$$T = 0.1N$$ \hspace{1cm} (4)

where:

- $L$ = the overall building dimension in the direction of the earthquake forces or the length of the wall or braced frame parallel to the lateral forces, depending on the country’s interpretation of the UBC formula.
- $N$ = number of stories/levels.

Other codes, such as the New Zealand Standard, do not prescribe a simplified period formula. They prescribe a formula based on Rayleigh’s method, similar to the “Method B” formula in the UBC:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{\sum_{i=1}^{n} f_i \delta_i}}$$  \hspace{1cm} (5)

where:

- $w_i$ = that portion of the total seismic dead load located at or assigned to level $i$.
- $\delta_i$ = horizontal displacement at level $i$ relative to the base due to applied lateral forces, $f_i$.
- $f_i$ = lateral force at level $i$.
- $g$ = acceleration due to gravity.
- $n$ = uppermost level in the main portion of the structure.
Recent Developments

Goel and Chopra (1997, 1998) have presented alternative period formulas for moment-resisting frame buildings and for concrete shear-wall buildings. They obtained information about the fundamental modes of vibration of a number of buildings by analyzing their recorded motion from various California earthquakes. These structures were shaken strongly but not so strongly as to enter the inelastic range. They were divided into two categories depending on the strength of the earthquake shaking they experienced, i.e. whether or not the peak ground acceleration was less than 0.15 g. After determining that the current code formulas substantially underestimated the natural vibration periods for these structures, they re-evaluated the theory upon which the code formulas were based and derived new formulas by regression analysis. These period formulas lead to a best fit, in the least-squares sense, to the measured period data. The final recommended period formulas were derived looking at the trend obtained from only the buildings experiencing peak ground accelerations of 0.15 g or greater. At smaller acceleration levels, the periods tend to be smaller because the non-structural components contribute significantly to the lateral stiffness.

Goel and Chopra concluded that Rayleigh’s method was sufficient to give a good approximation of the dynamic behavior of moment-resisting frame buildings. Based on this method, the period formula should be of the form $T = C H^\gamma$, where $C$ and $\gamma$ are to be determined from regression analysis in the form $\ln T = \ln C + \gamma \ln H$. The least-squares estimates of $\ln C$ and $\gamma$ then give the median estimate of the period, that is, there is a 50% probability that the actual period of the building is greater that the period estimated by the regression formula. Since for determination of design base shear the formula should provide lower values of the period (to be conservative), Goel and Chopra chose a lower bound of a standard deviation from the best fit line. Also, they provided an upper limit for periods found using rational analysis rather than the code formula. The resulting lower bound period formulas were the following, with standard error estimates of $s_e = 0.209$ for RC MRF and $s_e = 0.233$ for steel MRF:

**Reinforced Concrete Moment Frames:**
$$T = 0.016 H^{0.90} \quad (6)$$
no larger than 1.4 T if using rational analysis

**Steel Moment Resisting Frames:**
$$T = 0.028 H^{0.80} \quad (7)$$
no larger than 1.6 T if using rational analysis

where

$H = h_n$, the total building height from base of structure, ft

Rayleigh’s method was not sufficient to give a good estimate for the dynamic characteristics of shear-wall buildings, so Goel and Chopra chose to use various other well established analytical procedures, such as Dunkerley’s method, which combines both flexural and shear deformations of a cantilever. Based on this method, the period formula should be of the form:
\[ T = C \frac{H}{\sqrt{A_e}} \]

where

\[ A_e = 100 \frac{A_e}{A_B} \]  \hspace{1cm} (8)

and

\[ A_e = \frac{A}{1 + 0.83 \left( \frac{H}{D} \right)^2} \]

\( H = h_n \), the total building height from base of structure, ft

\( C \) = constant to be defined by regression

\( A_B \) = plan area of building, ft\(^2\)

\( A \) = total area of shear walls, ft\(^2\)

\( D \) = building dimension parallel to direction being considered, ft

Regression analysis yielded the following formula, with an error of estimate \( s_e = 0.143 \) while the UBC-97 error estimate is \( s_e = 0.546 \):

**Concrete Shear Wall:**

\[ T_L = 0.0019 \frac{H}{\sqrt{A_e}} \]  \hspace{1cm} (9)

no larger than 1.4 \( T_L \) if using rational analysis

---

**Expansion of Period Database**

We will be developing a period database by analyzing various structural vibration records, such as available seismic records and dynamic tests to be performed throughout the course of this research (including the UCSD shake table tests). These vibration records will be analyzed using the program MODE-ID, which uses a well-established system identification procedure to estimate the modal parameters of the dominant contributing modes for the building being evaluated.

**Available seismic records**

We have obtained several earthquake records from wood-frame buildings instrumented by the California Strong Motion Instrumentation Program (see Appendix A). Attempts will also be made to collect and analyze any earthquake records from wood-frame buildings that are available from other strong motion databases, such as the USGS National Strong Motion Program. After analyzing these records, we will select additional structures to perform field vibration tests (both ambient and forced vibrations).

**Dynamic tests to be performed**

The purpose of the field vibration tests is to fill any gaps in the data obtained from the analysis of the earthquake records, providing us with a more reliable regression analysis result. The testing
will be performed by measuring naturally occurring vibrations or by forced vibrations induced with the Caltech shaker.

**Method for system identification**

Modal identification is an important application of system identification in structural dynamics where modal parameters based on a model with linear dynamics are estimated using dynamic data from a structure. Modal identification can be performed in the time domain without the need to develop a structural model involving mass, stiffness and damping matrices (Beck, 1978). The method was initially applied to the measured seismic response from tall buildings where only a single input (the recorded base acceleration) was used (Beck and Jennings, 1980). The method was then extended to handle multiple inputs in order to find the modal parameters from seismic motions recorded on a bridge (Werner *et al.*, 1987). The computer program called MODE-ID that implements this approach has been extensively applied to earthquake and other dynamic data. MODE-ID is based on a least-squares output-error method, which utilizes a class of models defined as follows.

Structural motion at the $N_o$ observed degrees of freedom is modeled as a superposition of $N_m$ dominant modes:

$$x_i(t) = \sum_{r=1}^{N_m} x_{ir}(t), \quad i = 1, \ldots, N_o$$

where $x_{ir}$ is the contribution of the $r^{th}$ mode to the response at the $i^{th}$ degree of freedom.

The response for the $(N_m - 1)$ dynamic modes of vibration is calculated numerically using a very accurate discrete-time recursive approximation (Beck and Dowling, 1988) of the well-known equation of motion:

$$\ddot{x}_{ir} + 2\zeta\omega_r\dot{x}_{ir} + \omega_r^2 x_{ir} = \phi_{ir} \sum_{k=1}^{N_I} p_{ik} f_k(t)$$

$$x_{ir}(0) = \phi_{ir} c_r; \quad \dot{x}_{ir}(0) = \phi_{ir} d_r; \quad \sum_{i=1}^{N_o} \phi_{ir}^2 = 1$$

where the $f_k, k = 1, \ldots, N_I$ are the measured accelerations at the $N_I$ structural supports (e.g. defining the motion at the base of the structure).

A pseudostatic “mode” is also necessary:

$$\ddot{x}_{ir} = \sum_{k=1}^{N_I} r_{ik} f_k(t)$$
This accounts for the quasi-static contributions to the structural motions induced by the support motions during the earthquake, ignoring inertial and damping effects since these are accounted for in the dynamic response contributions (Werner et al., 1987). The simplest pseudostatic mode is rigid-body motion such as the direct contributions from rocking and translation of the base of a building.

The model parameters \( a \) to be estimated are the modal parameters for each of the identified \((N_m - 1)\) dynamic modes, that is, the natural frequencies and damping ratios, \( \omega_r \) and \( \zeta_r \), the initial modal displacement and velocity, \( c_r \) and \( d_r \), the modeshape components at the observed degrees of freedom \( \{ \phi_{ir}, i=1,\ldots,N_o \} \), and the input participation factors \( \{ p_{rik}, k = 1,\ldots, N_I \} \); together with the pseudo static influence coefficients \( \{ r_{ik}, i = 1,\ldots,N_o, k = 1,\ldots, N_I \} \). The latter parameters may be fixed on a theoretical basis in some situations (e.g. for the pseudostatic response due to rocking and translation of the base of a building). Only the modeshape components at the observed degree of freedom can be identified since the “missing” modeshape components at the unobserved degrees of freedom cannot be identified directly without introducing a structural model as a basis for the “interpolation.”

The model parameters \( a \) are estimated by minimizing the mean square of the prediction errors at all the observed degrees of freedom, that is:

\[
J(a) = \frac{1}{N_o N} \sum_{i=1}^{N_o} \sum_{n=1}^{N} (\hat{y}_i(n) - x_i(n; a))^2
\]

Typically, the discrete system output \( \{ \hat{y}_i(n): n = 1,\ldots, N; i = 1,\ldots, N_o \} \) in the above equation consists of measured acceleration time histories at \( N_o = 10 \) to 20 degrees of freedom for some sampling interval \( \Delta t \). The model output \( x_i(n; a) \) in the above equation is a nonlinear function of the parameters and so the minimization of \( J(a) \) must be done numerically by an iterative optimization algorithm. The optimization algorithm used in the MODE-ID program is a robust one exploiting the linearity of the model dynamics (Beck, 1978; Werner et al., 1987).
Period Formulas by Regression on Data

Method

We will use a Bayesian statistical estimation method similar to the Maximum Likelihood estimation method, that can be viewed as fitting a lognormal distribution to a database of periods which depends on selected regressors to be investigated. For example, a period formula similar to equation (1) can be derived from a statistical model of the form:

\[ \ln T = \ln C + \gamma \ln x + s_e^2 \epsilon \]

where \( \ln C \) and \( \gamma \) are parameters to be estimated, the regressor \( x \) is a structural characteristic, \( \epsilon \) is a unit Normal random variable (i.e. zero mean and unit variance) and \( s_e^2 \) is the variance in the predicted value of \( \ln T \).

The estimated relationship \( \ln \hat{T} = \ln \hat{C} + \hat{\gamma} \ln x \) can be viewed as a best fit curve for the data set, although technically it gives the median period for the given regressor value. The standard error for this period estimate is calculated from:

\[ s_e = \sqrt{\frac{\sum_{i=1}^{N} [\ln T_i - (\ln \hat{C} + \hat{\gamma} \ln x_i)]^2}{(N - 2)}} \]

where:

\( \ln \hat{C} \) and \( \hat{\gamma} \) are the estimates that minimize \( s_e \) as a function of \( \ln C \) and \( \gamma \), and \( N = \) total number of data points \( (T_i, x_i) \) in the period database.
Regressors To Be Investigated

Here are some parameters whose individual or combined effects on the dynamic behavior of wood frame structures will be evaluated:

- Height of building
- Width of building
- Length of building
- Effective shear area, $A_e$
- Nailing pattern, plywood thickness, etc. (perhaps by using UBC allowable shear force as a proxy).
References


May 26, 1999

Robert K. Reitherman
Executive Director
CUREe
1301 S. 46th Street
Richmond, CA 94804-4698

Dear Bob:

As you requested, enclosed please find a table listing the seven wood-frame buildings that are currently instrumented by the California Strong Motion Instrumentation Program (CSMIP). The table includes number of stories, lateral force resisting system, year of construction, instrumentation date, and number of sensors.

Low-level shaking and response records have been obtained at five of the seven wood-frame buildings since they were instrumented. Maximum acceleration values recorded on the ground and the structure of these five buildings are listed in the second table.

CSMIP is planning to instrument more wood structures, especially single-family dwellings, as recommended by the Strong Motion Instrumentation Advisory Committee (SMIAC) of the Seismic Safety Commission. We are looking forward to working cooperatively with the CUREe-Caltech Woodframe Project.

Sincerely,

[Signature]
Anthony F. Shakal
Program Manager
California Strong Motion Instrumentation Program

Enclosures
List of CDMG/CSMIP Instrumented Wood-Frame Buildings as of 5/99

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Station Number</th>
<th>No. of Stories</th>
<th>Floor Dimension</th>
<th>Lateral Force Resisting System</th>
<th>Year of Construction</th>
<th>Date Instrumented</th>
<th>No. of Sensors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parkfield – Elementary School</td>
<td>36531</td>
<td>1</td>
<td>48' x 30'</td>
<td>plywood shear walls in longitudinal direction</td>
<td>1949</td>
<td>6/87</td>
<td>6+FF</td>
</tr>
<tr>
<td>Bishop - Fire Station</td>
<td>54545</td>
<td>1</td>
<td>62' x 50'</td>
<td>Perimeter plywood shear walls</td>
<td>1983</td>
<td>9/88</td>
<td>6</td>
</tr>
<tr>
<td>Eureka – Office Building</td>
<td>89687</td>
<td>2</td>
<td>80' x 48'</td>
<td>perimeter plywood shear walls</td>
<td>1992</td>
<td>2/95</td>
<td>11</td>
</tr>
<tr>
<td>Templeton – Hospital *</td>
<td>36695</td>
<td>1</td>
<td>87' x 51'</td>
<td>distributed plywood shear walls</td>
<td>1977</td>
<td>6/94</td>
<td>9+FF</td>
</tr>
<tr>
<td>San Bernardino – Motel</td>
<td>23701</td>
<td>3</td>
<td>181' x 48'</td>
<td>plywood shear walls in 1st story</td>
<td>1986</td>
<td>9/94</td>
<td>15</td>
</tr>
<tr>
<td>Fremont – Motel</td>
<td>57720</td>
<td>2</td>
<td>145' x 38'</td>
<td>plywood shear walls in long. direction in 1st story</td>
<td>1989</td>
<td>7/95</td>
<td>11</td>
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<tr>
<td>Indio - Hospital *</td>
<td>12759</td>
<td>1</td>
<td>298' x 244'</td>
<td>distributed plywood shear walls</td>
<td>1981</td>
<td>6/97</td>
<td>8+FF</td>
</tr>
</tbody>
</table>

* Instrumented under OSHPD/CSMIP Hospital Instrumentation Project.
List of Light-Shaking Records from CSMIP-Instrumented Wood Frame Buildings

<table>
<thead>
<tr>
<th>Date of Earthquake</th>
<th>Time (UTC)</th>
<th>Magnitude (Ml)</th>
<th>Epicentral Distance (km)</th>
<th>Maximum Acceleration (g)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>hr:min:sec</td>
<td></td>
<td></td>
<td>Ground</td>
<td>Structure</td>
</tr>
<tr>
<td><strong>Parkfield – Elementary School</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>04/04/93</td>
<td>05:21:25.3</td>
<td>4.2</td>
<td>7</td>
<td>7.5% H</td>
<td>12.3% H</td>
</tr>
<tr>
<td>12/20/94</td>
<td>10:27:47.2</td>
<td>4.7</td>
<td>4</td>
<td>8.9% H</td>
<td>20.1% H</td>
</tr>
<tr>
<td><strong>Bishop – Fire Station</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>05/17/93</td>
<td>23:20:48.8</td>
<td>6.0</td>
<td>61</td>
<td>1.8% H</td>
<td>4.4% H</td>
</tr>
<tr>
<td><strong>Eureka – 2-story Office Building</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>02/08/95</td>
<td>09:36:51.1</td>
<td>3.9</td>
<td>13</td>
<td>3.7% H, 0.8% V</td>
<td>6.2% H</td>
</tr>
<tr>
<td><strong>San Bernardino – 3-story Motel</strong></td>
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<td></td>
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<tr>
<td>06/28/97</td>
<td>21:45:25.1</td>
<td>4.2</td>
<td>1</td>
<td>6.4% H, 3.5% V</td>
<td>9.2% H</td>
</tr>
<tr>
<td>07/26/97</td>
<td>10:24:16.9</td>
<td>3.7</td>
<td>0</td>
<td>3.8% H, 2.7% V</td>
<td>7.6% H</td>
</tr>
<tr>
<td>03/11/98</td>
<td>12:18:51.8</td>
<td>4.5</td>
<td>18</td>
<td>2.3% H, 1.1% V</td>
<td>7.1% H</td>
</tr>
<tr>
<td><strong>Indio – 1-story Hospital</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>07/26/97</td>
<td>03:14:56.0</td>
<td>4.9</td>
<td>33</td>
<td>2.4% H</td>
<td>8.3% H</td>
</tr>
</tbody>
</table>
Task No. 1.4.1.1 - Anchorage of Woodframe Buildings
Literature Review

James A. Mahaney
Wiss, Janney, Elstner Associates, Inc.
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Introduction

This report was prepared as part of the CUREE-Caltech Woodframe Project, which is aimed at developing reliable and economical ways of improving woodframe building performance in earthquakes. The Woodframe Project is divided into five interrelated elements: Testing and Analysis; Field Investigations; Building Codes and Standards; Economic Aspects; and Education and Outreach. This report presents the results from the literature review for Task 1.4.1.1: Anchorage of Woodframe Buildings, which is within the Testing and Analysis element.

The main objective of Task 1.4.1.1 is to improve the understanding of the seismic performance of sill-plate-to-foundation anchorage connections and thereby increase the reliability of these critical connections under earthquake loading. The impetus of this research results from brittle failures (longitudinal splitting) of sill plates that have been observed after earthquakes and during laboratory testing. Another objective of Task 1.4.1.1 is to understand the sill plate failure mechanism and the factors that affect its performance so that ductile sill-plate-to-foundation connections can be developed.

Task 1.4.1.1 is divided into two parts: a literature review and laboratory testing. The primary purpose of the literature review is to identify variables from previous testing and post-earthquake observations that should be considered during the laboratory testing part of this task. Using findings from the literature review, laboratory tests will be conducted on a wide range of different sill-plate-to-foundation anchorage configurations that will include variables such as sill plate thickness and width, anchor bolt diameter, anchor bolt washer sizes, over-sized bolt holes, etc. This report addresses the literature review part of Task 1.4.1.1.

The approach taken for the published literature review was to concentrate the research in three subject areas: 1. Sill plate-to-foundation tests; 2. full-scale shear wall tests; 3. documented post-earthquake sill plate damage. Resources for the literature review included university libraries, university researchers, organizations associated with wood products, organizations associated with earthquake engineering, and published documents located on the internet. Based on the literature reviewed to date, valuable information was obtained on the behavior of a variety
of sill plate anchorage configurations. This information was used to further develop and direct the testing component of the task.

**Task Methodology**

The approach that was taken to accomplish this task was to first identify potential test variables to be evaluated. Initially the test variables included the following: wood species, bolt diameter, power-driven pins, sill plate thickness and width, washer size and type, dead loads, bolt location in sill plate, oversized bolt holes in sill plate, retrofitted sill plate connections, and foundation material. A literature review on the subject of sill plate-to-foundation connections was then performed. The purpose of the review was to avoid repeating earlier research and to extract relevant information. Information from the literature identified several test variables that appear to influence the performance of sill plate-to-foundation connections. Based on our review of the literature, a testing program has been developed that focuses the research and testing on selected test variables while holding other test variables constant. The testing program will evaluate the performance of several different sill plate-to-foundation assemblies.

**Literature Review**

**Sill plate-to-foundation tests:** Published literature that evaluated the performance and behavior of bolted sill plate-to-foundation connections was reviewed. The review focused on wood sill plates with bolted connections, which are loaded in single shear. Power-drive pin type fasteners through wood sill plates into concrete floor/foundations were also reviewed through selected Evaluation Reports by the International Conference of Building Officials (ICBO) and other organizations, which evaluated proprietary products. These evaluation reports are prepared based on test reports submitted by the manufacturers.

**Full-scale shear wall tests:** Due to the eccentricities between the sheathing and anchor bolts in a typical shear wall assembly as well as concentrated loads at the ends of shear walls due to overturning, failures of sill plates are probably not solely caused by in-plane shear, but are likely caused by a combination of shear and overturning forces. Previous shear wall tests were reviewed for pertinent information including, test setup, loading, and results.

**Documented post-earthquake sill plate damage:** Published documentation on the behavior and performance of sill plate-to-foundation connections in woodframed buildings was reviewed.

Of the potential test variables identified above, several test variables appeared to influence the performance of sill plate-to-foundation connections. These test variables will be evaluated in the testing component of this task. In addition, sill plate-to-foundation configurations that appeared to have performed well were incorporated into the test program.
Tests

Through a series of tests, WJE will evaluate the performance of sill plate-to-foundation connections using traditional anchor bolts and washers, anchor bolts with oversized washers, anchor bolts with square plate washes, and power-driven pins. A variety of sill plate widths and thicknesses will be evaluated through testing. WJE will develop and test alternative sill plate-to-foundation connections that minimize sill plate cross-grain bending. Tentative alternative connections incorporate double sill plates nailed together, and rods extending above the sill plate without nuts and washers. Anchor bolts with oversized holes drilled in the sill plate and retrofitted sill plate-to-foundation connections will also be evaluated through testing.

Some sill plate connections will be tested in direct shear, but the majority of the sill plates tests will incorporate direct shear coupled with the overturning component of the shear wall. The proposed test setup uses a four-foot long section of shear wall, which is bolted to a simulated concrete foundation. The test setup is intended to model the inherent eccentricity between the sheathing and the anchor bolts, or drive pins, which creates cross grain bending in the sill plate as the sheathing lifts the sill plate off the foundation. Loads will be applied to the test specimens using the loading protocols developed under a separate task.

Sill Plate Anchorage Mechanics

This section of the report describes the construction of a typical shear wall, and presents plausible sill plate failure mechanisms and feasible assemblies that improve the performance of sill plate-to-foundation connections. Woodframed shear walls are framed using 2x studs and plates, sheathed on one or both sides, and connected to the foundation with anchor bolts or power-driven pins. Holdowns may or may not be located at each end of the wall. Historically, anchor bolts and power-driven pins have used conventional washers that are 2 to 3 times the bolt or pin diameter. More recently, large square plate washers have been used with anchor bolts, and 3x sill plates have been used in lieu of 2x plates.

When shear walls are loaded, horizontal shear forces are transferred into the sheathing at the top of the wall, transferred out of the sheathing and into the sill plate, and transferred from the sill plate into the foundation through anchor bolts or power-driven pins. Shearwall overturning moments generate tension and compression forces in the sheathing and/or boundary members (studs or holdown posts) at the ends of the wall. If the overturning moment on the wall exceeds the dead load restoring moment and the strength of the holdown, the wall tends to lift off the foundation.

If holdowns are not present or installed such that they allow the wall to uplift, the overturning forces (tension or compression) are predominately in the sheathing and concentrated in the area of the first anchor bolt from the end of the wall. The tension force in the sheathing is transferred to the anchor bolt through the sill plate. The tension force times the distance between the sheathing and the edge of anchor bolt washer generates a bending moment across the grain of the sill plate (cross-grain bending). If conventional washers have been used, the cross-grain bending moment may cause the sill plate to split. If large plate washers have been used, the tendency for
the sill plate to split may be reduced because more of the eccentricity is taken by bending of the plate washer, than by cross-grain bending in the sill plate. If 3x sill plates are used, the cross-grain bending strength of the plate may also prevent sill plate failures.

If special holdowns which have been designed to prevent the sill plate from lifting off the foundation are located at the ends of the wall, the overturning forces will be predominately in the holdown post. The tension in the holdown post is transferred to the foundation through the special holdown. Very little tension is in the sheathing, and the bending moment across the grain of the sill plate is very small. As a result, sill plate failures do not occur.

However, some commercially available holdown assemblies may not be stiff enough to prevent the sill plate from lifting off the foundation. As a result, overturning forces are distributed between the holdown, and the sheathing-to-sill plate-to-anchor bolt assemblies based on relative stiffness of the two assemblies. Depending on the type of anchor bolt washer used in the assembly, cross-grain bending moments may or may not cause sill plate failures.

**Literature Review**

The literature review was intended to be as broad as possible and included publications available through universities; federal, state, and local governments; and the private sector. On numerous occasions, the authors of publications were contacted in an effort to obtain additional information. Valuable information and additional literature on the subject of sill plate-to-foundation connections was also obtained through discussions with colleagues associated with wood technology and woodframe construction. Relevant information from several hundred tests was evaluated.

The literature review concentrated on three subject areas: 1. sill plate-to-foundation tests; 2. full-scale shear wall tests; 3. documented post-earthquake sill plate damage. A test results summary for each subject area is presented below. More detailed information regarding the tests is contained in Appendices A through D.

**Sill Plate-to-Foundation Tests**

Published test results on sill plate-to-foundation anchorages were obtained through the Earthquake Engineering Research Center (EERC) and the ASCE Journal of Structural Engineering. For the tests, 2x4, 2x6 and 3x4, Southern Pine sill plates were bolted to a concrete foundation or to double 2x main members with a single bolt and loaded in single shear. The loading utilized displacement controlled reverse-cyclic protocols. Some of the tests incorporated a restraining strap in the area of the bolt that was on the top and sides of the sill plate. The specimens with the strap failed at a higher ultimate load than the specimens without the strap.

The test results suggest that 2x4 sill plates are more susceptible to splitting failures than thicker or wider sill plates. In addition, the confining affects of the straps improved the performance of the sill plates. Test specimens with restraining straps tended to fail at higher loads because
failures were due to a combination of bolt yielding or fracture and sill plate damage instead of only splitting failure of the sill plate.

Power-driven pins are frequently used as a substitute for anchorage of interior walls to concrete, and where cast in place anchors have been omitted or walls have been relocated. Typically, these applications are for non-load bearing walls. Steel pins are mechanically driven through the sill plate and embedded into the concrete below. The pins and the equipment used to drive the pins into the concrete are manufactured and distributed by proprietary manufacturers. In order for building codes to allow these systems to be used, the manufacturers conduct testing of their materials and submit the results to the various code approval agencies.

Information on power-driven pins from two companies was reviewed. This information included the evaluation reports from the International Conference of Building Officials (ICBO) and the City of Los Angeles. One of the manufacturers also supplied a copy of the test report that was submitted to ICBO. Both manufacturers use a similar method to test their products; they drive their pins through a wood sill plate, remove the wood while leaving the pins in place, and then the pins are tested in shear and tension. ASTM standard E 1190-95 *Standard Test Methods for Strength of Power-Actuated Fasteners Installed in Structural Members* is used for conducting the test on the pins. In this standard, the test load is applied through a steel plate to the pin rather than applying load to a wood sill plate. Both product manufacturers list allowable values for their product based on the strength of the anchorage in concrete and state that the allowable strength of their product in wood should be determined by calculation based on national standards.

**Full-scale Shear wall Tests**

Published results on to full-scale shear wall tests were obtained through the University of California, Irvine/City of Los Angeles shear wall testing program, EERC, the ASCE Journal of Structural Engineering, the American Plywood Association (APA), and American Forest & Paper Association (AF&PA). For these tests, plywood was applied to one or two faces of 8-foot high, woodframed stud walls of different lengths. The sill plate was attached to concrete, steel, or timber supports. Loads were typically applied at the top of the wall, but varied in loading pattern and loading rate. Test variables that were identified from the literature included sill plate thickness and width; wood species; size, spacing and location of anchor bolts; anchor bolt washer size; holdown details; and applied vertical loading. Information was also collected on the ultimate horizontal load and the failure mode of the test specimen.

The literature review identified 12 resources that were related to full-scale shearwall tests. Collectively, the resources reported results from 201 tests. Due to the large number of test variables, definitive conclusions regarding the influence of test variables on sill plate anchorage performance could not be reached. However, after evaluating the test data, it was apparent that improved sill plate anchorage performance was achieved when one or more of the following items was incorporated into the shear wall assembly.
• Holdowns: When special holdowns were designed to minimize vertical displacement at the end of the shear wall, and, therefore minimize the uplift of the sill plate, greater shear wall capacity was achieved and sill plate splitting did not occur during the tests.

• Anchor bolt washers: Oversized anchor bolt washers reduced the tendency for the sill plates to split and in some cases eliminated it.

• Sill plate thickness: 3x sill plates performed better than 2x sill plates, and 3x sill plates did not exhibit splitting failures.

Documented Post-Earthquake Sill Plate Damage

While post-earthquake damage has been reported, there was very limited detailed information on the performance and behavior of sill plate-to-foundation connections. Useful information was obtained from EERC and Earthquake Damage Analysis Corporation. The reports documented damaged plywood, split sill plates, anchor bolt deformations, crushed wood under anchor bolt washers, split holdown posts, cracked concrete stem walls, and cracked concrete slabs. The anchor bolt washers were typically round and appeared (from the photographs) to be standard size washers. As-built deficiencies noted in the reports included missing and wrong size nuts and washers, and anchor bolts located too close to the edge of the foundation and/or sill plate. Discussions with APA would also suggest that oversized bolt holes in the sill plates, and cutting and notching of sill plates adversely affected the performance of the sill plate-to-foundation connections.

In contrast with the level of sill plate damage reported in post-earthquake documentation, full-scale shearwall tests performed in the laboratory rarely report sill plate related failures. The differences between test configurations and actual construction practice may account for the differences in performance.

Discussion

Test Variables

Test variables, which were contained in the literature and related to sill plate-to-foundation anchorage, were reviewed and qualitatively evaluated. Summarized below are the test variables that may influence the performance of sill plate-to-foundation connections. A brief discussion regarding the research findings is included with each test variable.
Washer Size and Type

Of the 201 shearwall tests, 187 tests from different resources reported using plate washers of at least 2 1/2 inches x 2 1/2 inches x 3/16 inch (64 mm x 64 mm x 4.8 mm). Two tests reported using round washers with 3x sill plates. Specific information as to the size and shape of the washers was not obtained from 12 tests. Out of the 187 tests with plate washers, only 8 specimens exhibited some degree of sill plate failure. Sill plate failures did not occur in the two tests with round washers and 3x sill plates. The effect of sill plate thickness is discussed below.

One of the post-earthquake documents indicated that sill plate splitting was related to the use of round washers.

The research clearly indicates that washer size and possibly washer shape influence the behavior and performance of sill plates. Because of insufficient test data, conclusions could not be reached regarding the influence that round washers have on the behavior and performance of sill plates. However, the research does indicate that oversize square plate washers, as noted above, reduce or eliminate sill plate splitting failures. As a result, sill plates with a variety of washer size and shape are proposed in the testing program. The proposed tests may demonstrate that oversize washers reduce cross-grain bending moment in the sill plate to an acceptable level were the sill plate does not split.

Holdown Type

The majority of full-scale shearwall tests reviewed had holdown devices at the ends of the shearwall to resist overturning. Several of the tests used commercially available holdown hardware. However, the majority of the tests used specially designed and fabricated holdowns (special holdowns) that prevented the ends of the shearwall from lifting up. Very few of the specimens that used the special holdowns suffered sill plate splitting type damage. In contrast, the failure mode of the specimens that used commercially available holdowns varied depending on whether or not other test variables were included in a specific test. Of the tests with commercially available holdowns, some specimens exhibited sill plate splitting failure, while other specimens exhibited other types of failures.

The test results demonstrate that special holdowns tend to prevent the sill plate from lifting as the boundary element and holdown assembly is loaded. As a result, the sill plate is not subject to cross grain bending, which may cause splitting failures. It appears from the literature that these tests were intended to evaluate sheathing material. The special holdowns were used to ensure that failure occurred in the sheathing material and not in the sill plate. However, special holdowns are not commercially available, and therefore, not representative of as-built conditions. It is likely that the cost to field install special holdowns with the accuracy and level of quality control as was used in these tests would be extremely high. At this point in time, alternative means and methods are proposed to improve the performance of sill plate-to-foundation connections. The current testing program does not intend to use specially designed and fabricated holdowns.
**Sill Plate Thickness and Width**

Test specimens were fabricated using 2x4, 2x6, or 3x4 sill plates. All shearwall and single shear test specimens that were constructed with 3x4 sill plates did not exhibit sill plate splitting failures. The single shear test specimens that were constructed with 2x6 sill plates did not perform as well as 3x4 members; 2x6 specimens failed due to bolt yielding, necking and fracture, and sill plate crushing and splitting. Because the shearwall tests used only 2x4 or 3x4 framing, the performance of 2x6 framed shearwalls was not evaluated. Due in part to the variability in the test results for the 2x4 sill plates, it was not possible to draw conclusions regarding 2x4 sill plate splitting failures. However, it should be noted that 94 out of 102 specimens that were fabricated with 2x4 sill plates and oversize square plate washers did not exhibit sill plate splitting failures. Because sill plate thickness and width may be an important factor, sill plates of varying thickness and width in combination with different types of washers are proposed in the testing program. The proposed tests may demonstrate that when certain types of washers are used the cross-grain bending strength of a particular size sill plate will be sufficient to prevent sill plate splitting failures.

**Wood Species**

Most test specimens were constructed using Douglas Fir. Other wood species used in the tests included Southern Pine and Spruce Pine Fir. Hem Fir was not reported to be used in any tests. Common practice on the west coast is to use pressure-treated Hem Fir for sill plates. Hem Fir is a less dense material, and is easier to pressure-treat than Douglas Fir. Southern Pine, which is commonly used for sill plates in the eastern U.S. is similar in density to Douglas Fir. It was not possible to quantitatively determine how different wood species influenced the performance of the sill plates and the results of the tests.

For the proposed tests, it is assumed that the cross-grain bending strength will not vary significantly between the species of wood typically used in the construction of sill plates attached to foundations. With respect to wood species, tests are proposed with pressure-treated Douglas Fir, Southern Pine and Hem Fir sill plates. Since Hem Fir has a lower specific gravity and lower allowable bolt values than Douglas Fir and Southern Pine, the majority of the tests will use Hem Fir sill plates. Stud grade or better Douglas Fir is proposed for the wall framing.

**Foundation Material**

Most test setups used reinforced concrete foundations to anchor the shearwall sill plates and holdowns, while a few test setups used steel or timber (which was, in turn, attached to a rigid base). There were no indications that the type of foundation material affected the performance of the sill plate anchorage or shearwall test. The post-earthquake documents support this conclusion even in the case of a substandard concrete foundation. There were only a few reports of spalled or cracked concrete footings. The post-earthquake documents indicate that the weak link in the system is some other component of the lateral force resisting system.
The proposed tests will use a short reinforced concrete stem wall to simulate a typical foundation. The stem wall will be secured to the test bay floor.

**Bolt Diameter**

Most test specimens used 5/8 inch (16 mm) diameter bolts, although 1/2 inch (13 mm), 3/4 inch (19mm), and 7/8 inch (22 mm) bolts were also used. It was not possible to quantitatively determine how different anchor bolt sizes influenced the performance of the sill plates and the results of the tests.

Based on the authors' experience, anchor bolts used in residential construction are 1/2 or 5/8 inch diameter with the exception that heavily loaded shearwalls may be designed using 3/4 inch diameter anchor bolts. A series of tests have been proposed that are intended to identify quantitatively the relative performance of 2x4 and 2x6 stud walls with either 1/2 or 5/8 inch diameter anchor bolts. Larger diameter anchor bolts are currently not in the test program.

**Dead Load**

Most tests were performed with no vertical loads except for the self-weight of the specimen and test setup. As a result, conclusions solely from the tests could not be reached regarding the influence of dead load on the performance of sill plates.

While the tests did not superimpose vertical load onto the shearwall assemblies, woodframed construction will have a combination of load bearing and non-bearing walls. Bearing walls with superimposed vertical load will act similar to the special holdowns discussed above, in that the vertical load will tend to prevent the sill plate from lifting off the foundation, and will reduce cross-grain bending moments in the sill plate. It is anticipated that superimposed vertical load will improve the performance of the sill plate-to-foundation connection. A series of tests have been proposed that are intended to replicate “typical residential” bearing and non-bearing shearwalls without holdowns.

**Bolt Location and Oversize Bolt Holes**

Most test specimens used 1/16 inch (1.6 mm) oversize bolt holes. From the literature it was not possible to determine the location of the bolts relative to the centerline axis of the sill plate or the influence that bolt hole size had on the test results and performance of the sill plates.

Based on the authors experience, anchor bolts used in residential construction are not predrilled and bolted to sill plates, but are normally “wet set” in the concrete. As a result, the location of field-installed anchor bolts with respect to the centerline axis of the sill plate can vary by several inches. Prior to erecting the framed wall onto the foundation, anchor bolt holes are drilled in the sill plate. To expedite the wall erection, some contractors elect to drill holes in the sill plate that are in some cases 1/4 inch larger than the size of the anchor bolt. Considering the above practice and typical construction tolerances, field-installed anchor bolts will be located in oversized holes.
with some bolts in partial contact with the sill plate while other bolts “float” within the bolt hole. As the shearwall is loaded, one bolt may take the majority of the load until the bolt and/or sill begins to fail. The sill plate slides until it contacts other anchor bolts. Theoretically, a progressive sill plate failure could occur. Even though shear friction forces are neglected when designing sill plate anchorages, shear friction forces are developed between the sill plate and foundation, and will resist lateral load in proportion to the coefficient of friction and the vertical load on the wall. This action may improve the performance of sill plates. However, while shear friction forces reduce the shear forces acting on the anchor bolts, the uplift forces and load path associated with overturning moments are not changed.

To evaluate offset anchor bolts and oversized bolt holes, a series of tests are proposed that quantitatively identify the relative performance of 2x4 and 2x6 sill plates with eccentric bolts and 2x4 sill plates with either 1/8 or 1/4 inch oversized bolt holes. To evaluate the influence that shear friction and vertical load has on lateral loads resisted by anchor bolts, additional in-plane shear tests are proposed on walls with vertical loads applied that are intended to represent both non-bearing and bearing wall conditions.

**Epoxy for Oversized Bolt Holes and Slant Bolts**

Information on oversized bolt holes filled with epoxy or slanted bolts was not found during our research. Most test specimens used 1/16 inch (1.6 mm) oversize bolt holes, which conform with building codes, and would not be required to be fill with epoxy. As a result, the literature research did not yield useful information.

However, as discussed in the *Bolt Location and Oversize Bolt Holes* section above, field-installed anchor bolts may be located too close to the edge of the sill plate or may be located within oversized holes. Several retrofit techniques have been used in the field to address these issues. Bolts located too close to the edge of the sill plate are “abandoned” and new anchor bolts installed, either vertically or at an angle. Oversized bolt holes are filled with epoxy. A series of tests are proposed with slanted bolts, and with 1/8 and 1/4 inches oversized holes filled with epoxy.

**Power-Driven Pins**

Information related to power-driven pins was limited to the results of testing conducted by manufacturers of the power-driven fasteners. Even though the testing included driving the fasteners through a wood sill plate, the manufacturers tests were conducted to determine the strength of the fasteners in the concrete substrate. The test loads were applied to the pins through a steel plate and not through the wood sill. The manufacturers state that the allowable strength of their pins based on wood sill capacity should be determined by calculation based on national standards for wood rather than being based on their test results.

The two manufacturers whose literature was reviewed indicated that the typical sizes of power-driven pins that are used for sill attachments varies from 0.14 inches (3.5 mm) diameter to 0.205 inches (5.2 mm) diameter. The depth of embedment of the fasteners in the concrete varies from
about 5/8 inch (16 mm) to 1 7/8 inch (48 mm). Several tests are proposed with power-driven pins of about 0.14 inches (3.5 mm) diameter and about 0.17 inches (4.3 mm) diameter driven through 2 inch nominal sill plates. The loading will be applied to the pins through the sill plate. Standard plate washers, supplied by the manufacturers, will be used with all pins. Minimal dead loads will be used in the tests with drive pins since these applications are generally for non-load bearing applications.

**Proposed Test Program**

This section of the report briefly describes the testing apparatus, loading protocols, and tests that are tentatively planned for this task.

**Testing Apparatus**

The test set-up has been designed to test sill plate-to-foundation connections associated with shearwalls. A critical area of the connection is located toward the ends of the shearwall where not only horizontal shear forces must be transferred, but also overturning forces. Test specimens are intended to represent the lower corner of typical 8-foot high by at least 8-foot long shearwalls. Test specimens, which are 4-feet long and 6-feet tall, are built with 2x and 3x framing, sheathed in plywood, and connected to a simulated stem wall concrete foundation with either anchor bolts or power-driven pins. Some tests will include holdowns at the end of the wall. The stem wall foundation is secured to the floor of the test bay. Forces are applied to the test specimens through a plate and channel bolted to the plywood, and are ultimately transferred to the test bay floor. See Appendix E for further details.

**Loading Protocols**

As a logical transition from the single shear tests identified in the literature to tests on the proposed test specimens, an initial series of test are proposed with loads applied horizontally at the base of the specimen. The horizontally applied forces are intended to load the sill plate predominately in shear while minimizing overturning forces.

The balance of the specimens are tested with loads applied at a angle of one horizontal to two vertical. The angle at which the load is applied was established by assuming that the shearwall is 8-feet tall, and the wall is loaded in pure shear, i.e., the unit shear per foot is the same on all four sides of the wall. With a four-foot long segment of wall represented by the test specimen, the vertical force at the end of the wall will be twice as large as the horizontal force in the wall.

The loading protocols for this task are being established under Task 1.3.1 *Rate of Loading and Loading Protocol Effects.* It is envisioned that loads will be applied in a cyclic pattern and will vary in magnitude.


Tests

The tests are intended to evaluate the test variables that were identified in the literature review and which appeared to significantly influence the behavior and performance of sill-plate-to-foundation connections. A series of tests are planned that hold some test variables constant while modifying materials, components, or the configuration of selected test variables. See Appendix F for additional details.

The initial series of tests that apply horizontal forces to test specimens will evaluate selected test variables including wood species, sill plate thickness and width, anchor bolt size, power-driven pin size, and superimposed vertical load. For the remainder of tests, loads will be applied at a two-to-one incline. A series of tests will evaluate different sizes and shapes of anchor bolt washers with 2x4 and 2x6 sill plates; smooth and threaded rods without washers; 3x4 and 3x6 sill plates with standard washers or plate washers; and anchor bolts with standard or plate washers but with holdowns. Additional tests are proposed that evaluate oversized bolt holes in sill plates, oversized bolt holes filled with epoxy, retrofitted bolts installed at an angle, and bolts that are not located along the centerline axis of the sill plate. The additional tests incorporate different types of washers. A series of tests is also scheduled that use a double sill concept with and without holdowns. In this concept, the plywood is fastened to the upper sill, the lower sill is bolted to the foundation, and the sills are internailed.

Conclusions

The subject of this literature review was the anchorage of wood sill plates to foundations. To review the performance of these connections, test reports of sill plate-to-concrete anchors, tests of woodframed shearwalls, and post-earthquake reconnaissance reports have been reviewed. Only a limited number of tests were found that dealt specifically with wood sill plate-to-foundation connections. However, a larger number of woodframed shearwall tests have been conducted in which sill plate anchorage information was obtained. Reports of wood sill plate performance in previous earthquakes were also reviewed.

Information from the literature identified several test variables that appear to influence the performance of the sill plate-to-foundation connections. The most significant test variables are noted below:

- Many of the full-scale shearwall tests employed the use of special, very stiff holdown devices at the ends of the wall to prevent the sill plate from lifting off the foundation, which induces cross-grain bending moments in the sill plates. In these tests, sill plate failures have not been reported, whereas sill plate splitting has been reported in tests without special holdowns. To evaluate the performance of typical sill plate-to-foundation connections, standard commercially available holdowns are proposed with selected test specimens.

- Common construction practice has been to use standard cut round washers and nuts with sill plate anchors. Post-earthquake reports indicate that sill plate failures
generally involve longitudinal splitting of the sill plate. In contrast, full-scale shearwall tests indicate that oversized anchor bolt washers reduce the tendency of the sill plates to split and in some cases eliminate sill plate splitting. Tests are proposed to evaluate the performance of anchors with different size and shape washers.

- Common construction practice also has been to use 2x thick sill plates bolted to the foundation. Sill plate-to-foundation tests and full-scale shearwall tests indicate that 3x sill plates perform better than 2x sill plates, and 3x sill plates do not exhibit splitting failures. Tests are proposed to evaluate the performance of 2x4, 3x4, 2x6, and 3x6 sill plates with different size and shape anchor bolt washers.

- Occasionally, anchor bolts and pins are not located along the centerline axis of the sill plate, and bolt holes in sill plates are oversized to accommodate construction tolerances, and contractor means and methods. Tests are proposed to evaluate the performance of sill plates with anchor bolts located in oversized holes, or located away from the sill plate center line. To address typical field conditions, feasible retrofit solutions will be tested that include bolts in oversized holes and filled with epoxy; and supplemental bolts installed at an angle.

**Acknowledgement**

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In addition, the authors would like to thank Tonatiuh Rodriguez-Nikl of Wiss, Janney, Elstner Associates, Inc., who conducted a large part of the literature survey, collected and analyzed data, and assisted in the writing and organization of this report. Kent Sasaki, SE of Wiss, Janney, Elstner Associates, Inc. provided valuable review of the report.
References


APPENDIX A

Single Shear Test Data
# CUREe-Caltech Woodframe Project
## Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in single shear tests

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<th>Publication</th>
<th>ASCE Journal of Structural Engineering</th>
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<tr>
<td><strong>Title</strong></td>
<td>Seismic Performance of Confined Sill Plate Connectors</td>
</tr>
<tr>
<td><strong>Author</strong></td>
<td>Joseph M. Bracci, Rebecca F. Stromatt, David G. Pollock</td>
</tr>
<tr>
<td><strong>Date</strong></td>
<td>Nov-96</td>
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### Loading Information

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<td><strong>Load Pattern</strong></td>
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<td><strong>Method of Loading</strong></td>
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<td><strong>Method of Attachment</strong></td>
<td>end caps on side member connect to actuator, main member attaches to reaction frame</td>
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<td><strong>Eccentricities</strong></td>
<td>controlled by laterally supporting rollers</td>
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### Single shear tests

| **Sill Thickness & Width (side member)** | 2 × 6 |
| **Foundation (main member)** | double 2 × 6. Split in some tests! |
| **Wood Species** | Southern pine, specific gravity = (0.68 - 0.80 for foundation, 0.39 to 0.45 for side member) |
| **Bolt Size** | 12.7 mm, 1/2" |
| **Bolt Spacing** | single bolt |
| **Bolt Edge Distance** |        |
| **Bolt End Distance** | very long |
| **Ultimate Load** | 13.3 kN, 2.99 kips, 14.8 kN, 3.33 kips, 13.8 kN, 3.10 kips, 14.5 kN, 3.26 kips, 17.8 kN, 4.00 kips |
| **Failure Mode** | bolt yielded, plate split, bolt yielded, plate split, bolt yielded, plate split, bolt yielded and necked, plate split, bolt fractured, plate split |

**Comments:** Designed to yield in NDS mode III
CUREe-Caltech Woodframe Project

Task 1.4.1 - Anchorage of Woodframe Buildings

Objective: Evaluate performance of sill plate-to-foundation type connections for use in shear walls in single shear tests

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<td>Method of Loading</td>
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<td>Method of Attachment</td>
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</tr>
<tr>
<td>Eccentricities</td>
<td>controlled by laterally supporting rollers</td>
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| Sill Thickness & Width (side member) | 2 x 6 |
| Foundation (main member) | double 2 x 6. Split in some tests! |
| Wood Species | Southern pine, specific gravity = (0.66 - 0.80 for foundation, 0.39 to 0.45 for side member) |
| Bolt Size | 12.7 mm, 1/2" |
| Bolt Spacing | 22.2 mm, 7/8" |
| Bolt Edge Distance | single bolt |
| Bolt End Distance | very long |
| Ultimate Load | 17.8 kN, 4.00 kips |
| Failure Mode | bolt and washer fractured, plate split |
| Comments | Designed to yield in NDS mode III |
| Designed to yield in NDS mode IV |
| Designed to yield in NDS mode III or IV |
| Designed to yield in NDS mode I or II |
**CUREe-Caltech Woodframe Project**

**Task 1.4.1 - Anchorage of Woodframe Buildings**

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in *single shear tests*.

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**Monotonic / Cyclic**: Cyclic

**Load Pattern**: sets of 2 cycles of amplitudes 1/8, 1/4, 3/8, 1/2, 3/4, 1, 1-1/2, 2, 2-1/2

**Rate**

**Method of Loading**: single shear with eccentricity

**Method of Attachment**: end caps on side member connect to actuator, main member attaches to reaction frame

**Eccentricities**: controlled by laterally supporting rollers

**Sill Thickness & Width (side member)**: 2 x 6

**Foundation (main member)**: double 2 x 6. Split in some tests!

**Wood Species**: Southern pine, specific gravity = (0.68 - 0.80 for foundation, 0.39 to 0.45 for side member)

**Bolt Size**: 22.2 mm, 7/8"

**Bolt Spacing**: single bolt

**Bolt Edge Distance**

**Bolt End Distance**: very long

**Ultimate Load**: 31.6 kN, 7.10 kips 26.2 kN, 5.89 kips 30.3 kN, 6.81 kips 27.6 kN, 6.21 kips 34.2 kN, 7.68 kips

**Failure Mode**: plate crushed and split shear wedge formed on one side plate crushed and split shear wedge formed on one side plate crushed and split

**Comments**: Designed to yield in NDS mode I or II
### CUREe-Caltech Woodframe Project

#### Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in single shear tests

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<td></td>
</tr>
<tr>
<td><strong>Test ID</strong></td>
<td>1 2 3 4 5 6 7 8 9</td>
<td></td>
</tr>
<tr>
<td><strong>Brief Summary</strong></td>
<td>Tests with and without steel confining device, various sill plate sizes</td>
<td></td>
</tr>
</tbody>
</table>

**Loading Information**

- **Monotonic / Cyclic:** Cyclic
- **Load Pattern:** Incremental displacement control. Ramp Loading. Two cycles at each displacement level from .125" to 3"
- **Rate:** quasi-static
- **Method of Loading:** Lateral shear force
- **Method of Attachment:**
- **Eccentricities:**

**Single-shear tests**

<table>
<thead>
<tr>
<th>Single-shear tests</th>
<th>Thickness &amp; Width (side member)</th>
<th>2 x 4</th>
<th>2 x 4</th>
<th>2 x 4</th>
<th>2 x 4</th>
<th>3 x 4</th>
<th>3 x 4</th>
<th>2 x 6</th>
<th>2 x 6</th>
<th>2 x 6</th>
</tr>
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<tbody>
<tr>
<td>Foundation (main member)</td>
<td>R / C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood Species</td>
<td>Southern Pine</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td>single bolt</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Bolt End Distance</td>
<td>very long</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>3.24 kips, 4.77 kips, 5.80 kips, 6.37 kips, 6.61 kips, 7.56 kips, 5.11 kips, 4.76 kips, 5.95 kips</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>split, bolt fracture, split, bolt fracture, bolt fracture, bolt fracture, bolt fracture, bolt fracture, bolt fracture</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Comments</td>
<td>no clamp, clamp, no clamp, clamp, no clamp, clamp, no clamp, clamp, no clamp</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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APPENDIX B

Full Scale Shearwall Test Data Summary
## Full-Scale Shear Wall Test Data Summary

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Loading</th>
<th>Anchorage Details</th>
<th>Frame Geometry</th>
<th>Failure Mode</th>
<th>Forces</th>
</tr>
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<tbody>
<tr>
<td><strong>Sheet No.</strong></td>
<td><strong>Column</strong></td>
<td><strong>No. Specimens</strong></td>
<td><strong>Reversed Cycle (R), Load/Unload (L),Monotonic (M)</strong></td>
<td><strong>Plate Washer (P) / Other</strong></td>
<td><strong>Commercial H.D. (C) / Special H.D. (S) / None (N)</strong></td>
</tr>
<tr>
<td>1 1 75</td>
<td>R D</td>
<td>P C</td>
<td>5/8</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>2 1 2</td>
<td>L S</td>
<td>P S</td>
<td>7/6</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>2 3 2</td>
<td>L S</td>
<td>P S</td>
<td>7/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>2 4 2</td>
<td>L S</td>
<td>P N</td>
<td>7/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>3 1 4</td>
<td>L S</td>
<td>P C</td>
<td>3/4</td>
<td>12</td>
<td>1</td>
</tr>
<tr>
<td>3 2 8</td>
<td>R D</td>
<td>P C</td>
<td>3/6</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>4 1 1</td>
<td>R S</td>
<td>C 1/2</td>
<td>2</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>4 2 1</td>
<td>R S</td>
<td>C 1/2</td>
<td>2</td>
<td>1</td>
<td>8</td>
</tr>
<tr>
<td>5 1 2</td>
<td>M S</td>
<td>S 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>5 2 3</td>
<td>R D</td>
<td>S 5/8</td>
<td>4</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>6 1 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>6 2 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>6 3 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>6 4 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>7 1 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>7 4 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>8 1 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>8 2 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
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<tr>
<td>8 3 1</td>
<td>R D</td>
<td>P S</td>
<td>5/8</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>9 1 1</td>
<td>M S</td>
<td>C 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>9 2 1</td>
<td>M S</td>
<td>C 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>9 3 1</td>
<td>R D</td>
<td>C 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>9 4 1</td>
<td>R D</td>
<td>C 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>9 5 1</td>
<td>R D</td>
<td>C 5/8</td>
<td>2</td>
<td>3.5</td>
<td>2.3</td>
</tr>
<tr>
<td>10 1 1</td>
<td>R D</td>
<td>O C 5/8</td>
<td>2</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>10 2 1</td>
<td>R D</td>
<td>O C 5/8</td>
<td>2</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>10 3 1</td>
<td>R D</td>
<td>P C 5/8</td>
<td>2</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>11 1 31</td>
<td>L S</td>
<td>P S</td>
<td>1</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td>11 2 12</td>
<td>L S</td>
<td>P S</td>
<td>1</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td>12 1 39</td>
<td>L S</td>
<td>P S</td>
<td>1</td>
<td>8</td>
<td>2</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>201</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*See Appendix C for data on individual tests*
APPENDIX C

Full Scale Shearwall Test Data
## CUREe-Caltech Woodframe Project

### Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in **full-scale shear wall tests**

<table>
<thead>
<tr>
<th>Publication</th>
<th>CoLA - UCI Light Framed Shearwall Research Project</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>UCI / CoLA</td>
</tr>
<tr>
<td>Author</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td>Oct 98 through Oct 99, CD with all CoLA due out in Jan</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>1</td>
</tr>
<tr>
<td>Test ID</td>
<td>Groups 1 through 27</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>75 Tests</td>
</tr>
</tbody>
</table>

### Monotonic / Cyclic
- Reversed Cyclic

### Load Pattern
- Sets of sine waves. Each set has amplitudes D, 0.75"D, 0.5"D, 0.25"D. D, D, D. Increase D in each subsequent set.
- .5 Hz from 2' to 1.6' then reducing linearly to .25 Hz @ 3.2''

### Method of Loading
- Servo-Hydraulic actuator

### Method of Attachment
- Load beam (pusher bar) attached to double top plate with multiple 3/8" lags (8 to 16 depending on expected sample strength)
- None by design

### Full scale shear wall tests

#### Sill Thickness & Width (side member)
- Mostly 3x sill, some 2x (See Group Summary on CD)

#### Foundation (main member)
- Bed of test setup is a 3.5" wide x 2" high steel "stem wall". This is attached to a 8" double channel which is in turn attached to lab strong wall (24" thick RC, #10's @ 10" o.c.)
- Studs & plates: 2x Std & btr DF, air dried. Posts: 4x4 DF KD Std & btr. 3x members where needed were cut from 4x4s such that knots were cut away (wood grade was maintained or improved).

#### Wood Species
- Heavy Plate washers (2.5 x 2.5 x 3/16) used with anchor bolts

#### Bolt Size
- 4 each - 5/8"

#### Bolt Spacing
- 2 ea at 12" from each end, one at 36" from wall end, and the last 40" from the other end

#### Washer Size
- 3/4" bolts on post centerline

#### Bolt End Distance
- Shear plate centerline 5.5" from end of post with 3x sill: 6.5" with 2x sill

#### Holdown Type
- Heavy Welded Ben Schmid HD 3071, attached to post using two 2 5/8" shear plates per post, cross bolts are 3/4" & heavy (2.75 x 2.75 x 1/4") plate washers used on outside of posts.

#### Sheathing Material (1 or 2 faces)
- 3/8 3 ply Str I, 15/32 4 ply Str I, 7/16 OSB (See Group Summary file on CD)

#### Dead Load
- See Group Summary on CD

#### Ultimate Load
- None

#### Failure Mode
- Nail fracture due to cycling

### Comments

---

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### Anchorage of Woodframe Buildings

**Task 1.4.1 - Anchorage of Woodframe Buildings**

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

<table>
<thead>
<tr>
<th>Publication</th>
<th>Confidential Research in Progress. Raw Data.</th>
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<tbody>
<tr>
<td>Title</td>
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</tr>
<tr>
<td>Author</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Sheet Number</td>
<td>2</td>
</tr>
<tr>
<td>Test ID</td>
<td>4A1.2, 4B1.2, 8A1.2, 8B1.3</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>4 x 8 wall with hold-down, 4 x 8 wall without hold-down, 8 x 8 wall with hold-down, 8 x 8 wall without hold-down</td>
</tr>
</tbody>
</table>

**Loading Information**

- **Monotonic / Cyclic:** Loading / Unloading Cycles
- **Load Pattern:** Design, Torsion Design, Failure
- **Rate:** Quasi-static
- **Method of Loading:**
- **Method of Attachment:**
- **Eccentricity:**
- **Comments:** Modified ES64 Test Method

**Full-scale shear wall test details**

- **Sill Thickness & Width (side member):** 2 x 4
- **Foundation (main member):**
- **Wood Species:**
- **Bolt Size:** 7/8" (1/32" oversized holes) anchor bolts for shear.
- **Bolt Spacing:** 3-3/4" minimum o/c spacing (non-standard, confidential detail)
- **Washer Size:** 2-1/2" square x 1/4"
- **Bolt Edge Distance:** approx. 1-5/16" (Calculated)
- **Bolt End Distance:** approx. 4" (drawing unclear)
- **Hold-down Type:** Simpson PHD8 with tension load bolt and slotted hole in hold-down, bolts pretensioned to 1 kip
- **Shear Wall Aspect Ratio (L/H):** 1:2 (8'-1-1/8" x 4'), 1:1 (8'-1-1/8" x 8'-1-1/8")
- **Sheathing Material (1 or 2 faces):** 15/32" OSB. One Face. 8" common at 6" o/c. edge, 12" o/c. field.
- **Dead Load:** "Some influence of dead load" Dead load data not reported
- **Ultimate Load:** 3.071 & 2.772 kips, 2.9 kips avg, 1.253 & 1.376 kips, 1.3 kips avg, 5.663 & 5.854 kips, 5.8 kips avg. [3.406, 4.071 & 3.700 kips, 3.9 kips avg. of two]
- **Failure Mode:** Sheathing Nails, Some minor sill plate splitting, Sheathing Nails, Some minor sill plate splitting
- **Comments:** Uplift at outermost bolt: 4.81 kips avg., Uplift: 3.1 kips avg., Uplift: 3.8 kips avg., Uplift: 2.5 kips avg. of two
# Curee-Caltech Woodframe Project
## Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

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<tr>
<td><strong>Publication</strong></td>
<td>ASCE Journal of Structural Engineering</td>
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<tr>
<td><strong>Title</strong></td>
<td>Comparisons of Static and Dynamic Response of Timber Shear Walls</td>
</tr>
<tr>
<td><strong>Author</strong></td>
<td>David W Dinneart &amp; Harry W. Shenton III</td>
</tr>
<tr>
<td><strong>Date</strong></td>
<td>Jun-96</td>
</tr>
<tr>
<td><strong>Sheet Number</strong></td>
<td>3</td>
</tr>
<tr>
<td><strong>Test ID</strong></td>
<td>static</td>
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<tr>
<td><strong>Brief Summary</strong></td>
<td>4 tests</td>
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<th>Loading Information</th>
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<tbody>
<tr>
<td><strong>Monotonic / Cyclic</strong></td>
<td>load / unload cycles</td>
</tr>
<tr>
<td><strong>Load Pattern</strong></td>
<td>Similar to ASTM E-564</td>
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<tr>
<td><strong>SEAOCS Sequential Phased Displacement</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Rate</strong></td>
<td>20 lb/sec</td>
</tr>
<tr>
<td><strong>Method of Loading</strong></td>
<td>Horizontal actuator on I beam with web attached to top plate</td>
</tr>
<tr>
<td><strong>Method of Attachment</strong></td>
<td>Guide I beam bolted to top of specimen</td>
</tr>
<tr>
<td><strong>Eccentricities</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Comments</strong></td>
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<table>
<thead>
<tr>
<th>Full scale shear wall tests</th>
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</thead>
<tbody>
<tr>
<td><strong>Sill Thickness &amp; Width (side member)</strong></td>
<td>2 x 4</td>
</tr>
<tr>
<td><strong>Foundation (main member)</strong></td>
<td>steel channel</td>
</tr>
<tr>
<td><strong>Wood Species</strong></td>
<td>#2 Spruce Pine Fir</td>
</tr>
<tr>
<td><strong>Bolt Size</strong></td>
<td>3/4&quot; A307</td>
</tr>
<tr>
<td><strong>Bolt Spacing</strong></td>
<td>12 bolts evenly spaced</td>
</tr>
<tr>
<td><strong>Washer Size</strong></td>
<td>10&quot; x 3&quot; x 1/2&quot; steel plate with two 7/8&quot; holes</td>
</tr>
<tr>
<td><strong>Bolt Edge Distance</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Bolt End Distance</strong></td>
<td></td>
</tr>
<tr>
<td><strong>Holdown Type</strong></td>
<td>commercially available. Two 3/4&quot; bolts in double 2x4 post</td>
</tr>
<tr>
<td><strong>Shear Wall Aspect Ratio (L/H)</strong></td>
<td>1:1 (8' x 8')</td>
</tr>
<tr>
<td><strong>Sheathing Material (1 or 2 faces)</strong></td>
<td>two plywood, two OSB (one face)</td>
</tr>
<tr>
<td><strong>Dead Load</strong></td>
<td>none</td>
</tr>
<tr>
<td><strong>Ultimate Load</strong></td>
<td>33.3 kN (7.5 kips), OSB avg. = 31.9 kN (7.2 kips)</td>
</tr>
<tr>
<td><strong>Failure Mode</strong></td>
<td>sheathing nails pull out, bottom plate split parallel to grain at uplift corner</td>
</tr>
<tr>
<td><strong>Comments</strong></td>
<td>excessive number of bolts to prevent any displacement at the base. Post peak envelope is very different for quasi-static and dynamic tests (significantly less ductility and maximum displacement in dynamic tests)</td>
</tr>
</tbody>
</table>
## Anchorage of Woodframe Buildings

### Objective:
Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests.

### Report Information

<table>
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<tr>
<th>Publication</th>
<th>Structural Engineering Worldwide 1998, Pub # T207-2, EERC CF11</th>
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<tbody>
<tr>
<td>Title</td>
<td>Performance of Bolted Wood-to-Concrete connections and bolted connections in plywood shear walls</td>
</tr>
<tr>
<td>Author</td>
<td>Joseph M. Bracci and Anthony Jones</td>
</tr>
<tr>
<td>Date</td>
<td>1998</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>4</td>
</tr>
<tr>
<td>Test ID</td>
<td>Confined sill plate</td>
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<tr>
<td>Brief Summary</td>
<td>1 test</td>
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### Loading Information

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<th>Reversed cyclic</th>
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<tr>
<td>Load Pattern</td>
<td>Incremental displacement control. Ramp loading. Two cycles at each displacement level from .125 to 3&quot;</td>
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<tr>
<td>Rate</td>
<td>Quasistatic</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Horizontal load at top</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>Steel I beam attached to top plate to distribute load</td>
</tr>
<tr>
<td>Eccentricities</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

### Full-scale shear wall tests

| Sill Thickness & Width (side member) | 2 x 4 | 2 x 4 with clamp reinforcement |
| Foundation (main member)            | R/C   |                                |
| Wood Species                        |       |                                |
| Bolt Size                           | 1/2"  |                                |
| Bolt Spacing                        |       |                                |
| Washer Size                         |       |                                |
| Bolt Edge Distance                  |       |                                |
| Bolt End Distance                   |       |                                |
| Holdown Type                        |       |                                |
| Shear Wall Aspect Ratio(U/H)        | 1:1   |                                |
| Sheathing Material (1 or 2 faces)  | 15/32" one face |                                |
| Dead Load                           | None  |                                |
| Ultimate Load                       | 8.14 kips | 9.19 kips                      |
| Failure Mode                        | Split in sill plate from end through hold-down bolt | Nail failure |
| Comments                             | Similar to the APA tests carried out at UCI |
### CUREe-Caltech Woodframe Project

Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

<table>
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<tr>
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<th>6th US Conference on Earthquake Engineering Proceedings, EERC CF10</th>
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<tbody>
<tr>
<td>Title</td>
<td>Lateral Load Resistance of Narrow Plywood Shear Walls</td>
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<tr>
<td>Author</td>
<td>Robin Shepherd and Bryan Alfred</td>
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<tr>
<td>Date</td>
<td>1998</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>5</td>
</tr>
<tr>
<td>Test ID</td>
<td>Static</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>2 tests</td>
</tr>
<tr>
<td>Monotonic / Cyclic</td>
<td>Monotonic</td>
</tr>
<tr>
<td>Load Pattern</td>
<td>constantly increasing displacement</td>
</tr>
<tr>
<td>Rate</td>
<td>2 inches per minute</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Horizontal load at top of frame</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>32&quot; x 5.5&quot; x 0.25&quot; plate bolted to overhang of double 2 x 4 top plate</td>
</tr>
</tbody>
</table>

#### Full scale shear wall tests

<table>
<thead>
<tr>
<th>Sill Thickness &amp; Width (side member)</th>
<th>2 x 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation (main member)</td>
<td>R / C</td>
</tr>
<tr>
<td>Wood Species</td>
<td>Douglas Fir</td>
</tr>
<tr>
<td>Bolt Size</td>
<td>two 5/8&quot; Hilli expansion anchor bolts</td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td>Symmetric. hold-down: 19&quot; center to center, anchor bolts: 9&quot; center to center</td>
</tr>
<tr>
<td>Washer Size</td>
<td></td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td></td>
</tr>
<tr>
<td>Bolt End Distance</td>
<td></td>
</tr>
<tr>
<td>Holdown Type</td>
<td>9&quot;, 5600 lb capacity, typical L-shaped. 3/4&quot; rod. two 3/4&quot; bolts (3/4&quot; holes) in post. Bolts prestressed to 1 kip. One end: same as at left. Other end: 27&quot; long-strap type. Bolts prestressed to 1 kip</td>
</tr>
<tr>
<td>Shear Wall Aspect Ratio(L/H)</td>
<td>1 : 3.5</td>
</tr>
<tr>
<td>Sheathing Material (1 or 2 faces)</td>
<td>3/4&quot; Struct. 1 plywood (one side). 10d common @ 4&quot; o.c. on edge, 12&quot; o.c. field</td>
</tr>
<tr>
<td>Dead Load</td>
<td>none</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>2250 lb, average of two tests</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Initial cupping of hold-down. plywood pulls away, sill plate split into 2 pieces, vertical member destroyed.</td>
</tr>
<tr>
<td>Comments</td>
<td>2 x 4 frame, double boundary elements and top plate. Negligible horizontal slip.</td>
</tr>
</tbody>
</table>

100 | Woodframe Project: Testing and Analysis Literature Reviews
CUREe-Caltech Woodframe Project
Task 1.4.1 - Anchorage of Woodframe Buildings

Objective: Evaluate performance of sill plate-to-foundation type connections for use in shear walls using full-scale shear wall tests

| Publication | APA, Technical Services Division. Research Report #158 |
| Title | Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading |
| Author | Rose, John D. |
| Date | Mar-98 |
| Test ID | 1A, 2B, 3A, 4A |

**Brief Summary**

- **Monotonic / Cyclic:** TCCMAR Sequential Phased Displacement
- **Reversed Cyclic:** TCCMAR Sequential Phased Displacement
- **Rate:** 0.5 Hz
- **Method of Loading:** Horizontal shear load applied at top
- **Method of Attachment:** Steel H Beam at top of wall with 3/4" thick x 3-1/2" wide spacers between steel and wall to allow panel rotation
- **Eccentricities:**
- **Comments:**

| Sill Thickness & Width (side member) | 3 x 4 |
| Foundation (main member) | 3" wide steel channel - edges inset to allow panel rotation |
| Wood Species | Green Douglas fir, 12-18% moisture |
| Bolt Size | 5/8", 1/16" max hole oversize |
| Bolt Spacing | 4 bolts, symmetrically placed, two at 12" from edge, two at 40° from holdown |
| Washer Size | Steel square plate washers 1/4" thick x 2-1/2" square on sill |
| Bolt Edge Distance | |
| Bolt End Distance | |
| Holdown Type | Specially designed for test. Welded: 4x4 posts. Two 3/4" bolts (25/32" holes) on post. 7/8" bolt (15/16" hole) to foundation. |
| Shear Wall Aspect Ratio(L/H) | 1:1 |
| Sheathing Material (1 or 2 faces) | One 15/32" struc 1 plywood, 10d common @ 4" o.c. | One 15/32" struc 1 plywood, 10d common @ 4" o.c. | One side 15/32", struc 1. Other side 1/2" GWB | One 15/32", struc 1 OSB #1, 10d common @ 4" o.c. |
| Dead Load | 10,150 lb (average), LF = 2.5, approx. disp. at ultimate = 2" |
| Ultimate Load | 10,000 lb (average), LF = 2.7, approx. disp. at ultimate = 2" |
| Failure Mode | Fastener fatigue starting near corners of panel |
| Comments | Uplift force at 1.44' lat. Disp. = 8300 lb | Uplift force at 1.44' lat. Disp. = 9300 lb | Uplift force at 1.44' lat. Disp. = 9100 lb | Uplift force at 1.44' lat. Disp. = 6850 lb |
### CUREe-Caltech Woodframe Project

**Task 1.4.1 - Anchorage of Woodframe Buildings**

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls using full-scale shear wall tests

<table>
<thead>
<tr>
<th>Publication</th>
<th>APA, Technical Services Division. Research Report #158</th>
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<tbody>
<tr>
<td>Title</td>
<td>Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading</td>
</tr>
<tr>
<td>Author</td>
<td>Rose, John D.</td>
</tr>
<tr>
<td>Date</td>
<td>Mar-68</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>7</td>
</tr>
<tr>
<td>Test ID</td>
<td>6A, 6A, 7A, 8A</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>Reversed Cyclic</td>
</tr>
<tr>
<td>Loading Information</td>
<td>TCCMAR Sequential Phased Displacement</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Horizontal shear load applied at top</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>Steel H Beam at top of wall with 3/4&quot; thick x 3-1/2&quot; wide spacers between steel and wall to allow panel rotation</td>
</tr>
<tr>
<td>Eccentricities</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>

**Shear Wall Aspect Ratio(L/H)**

| Sheathing Material (1 or 2 faces) | 3:1 |
| Shear Wall Aspect Ratio (L/H) | 1:1 |
| Wood Species | Green Douglas fir, 12-18 % moisture |
| Foundation (main member) | 3" wide steel channel - edges inset to allow panel rotation |
| Bolt Size | 5/8", 1/16" max hole oversize |
| Bolt Spacing | 4 bolts, symmetrically placed, two at 12" from edge, two at 40" from edge |
| Washer Size | Steel square plate washers 1/4" thick x 2-1/2" square on sill |
| Bolt Edge Distance | |
| Bolt End Distance | Specially designed for test. Welded. 4x4 posts. Two 3/4" bolts (25/32" holes) on post. 7/8" bolt (15/16" hole) to foundation. |
| Holdown Type | |
| Shear Wall Shear Wall Tests | |
| Shear Wall Test Info | |
| Dead Load | 8,750 lb (avg.), LF = 2.1, approx. disp. at ultimate = 2.1" |
| Ultimate Load | 10,300 lb (avg.), LF = 2.3, approx. disp. at ultimate = 2.0" |
| Failure Mode | Fastener fatigue starting near corner of panel |
| Comments | Uplift Force at 1.44" lat. Displ. = 7450 lb |
### CUREe-Caltech Woodframe Project

#### Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls using full-scale shear wall tests

<table>
<thead>
<tr>
<th>Publication</th>
<th>APA, Technical Services Division. Research Report #158 Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading</th>
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<tbody>
<tr>
<td>Title</td>
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</tr>
<tr>
<td>Author</td>
<td>Rose, John D.</td>
</tr>
<tr>
<td>Date</td>
<td>Mar-98</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>8</td>
</tr>
<tr>
<td>Test ID</td>
<td>BLS.1A, BLS.1B, BLS.1C</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>Tests from Appendix B – For Ben L. Schmid</td>
</tr>
<tr>
<td>Loading Information</td>
<td>-</td>
</tr>
<tr>
<td>Monotonic / Cyclic</td>
<td>Reversed Cyclic</td>
</tr>
<tr>
<td>Load Pattern</td>
<td>TCCMAR Sequential Phased Displacement</td>
</tr>
<tr>
<td>Rate</td>
<td>-</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Horizontal shear load applied at top</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>Steel H Beam at top of wall with 3/4&quot; thick x 3-1/2&quot; wide spacers between steel and wall to allow panel rotation</td>
</tr>
<tr>
<td>Eccentricities</td>
<td>-</td>
</tr>
<tr>
<td>Comments</td>
<td>-</td>
</tr>
<tr>
<td>Full-scale shear wall tests</td>
<td>-</td>
</tr>
<tr>
<td>Sill Thickness &amp; Width (side member)</td>
<td>3 x 4</td>
</tr>
<tr>
<td>Foundation (main member)</td>
<td>3&quot; wide steel channel - edges inset to allow panel rotation</td>
</tr>
<tr>
<td>Wood Species</td>
<td>Green Douglas fir, 12-18% moisture</td>
</tr>
<tr>
<td>Bolt Size</td>
<td>5/8&quot;, 1/16&quot; max hole oversize</td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td>4 bolts, symmetrically placed, two at 12&quot; from edge, two at 40&quot; from edge</td>
</tr>
<tr>
<td>Washer Size</td>
<td>Steel square plate washers 1/4&quot; thick x 2-1/2&quot; square on sill.</td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td>-</td>
</tr>
<tr>
<td>Bolt End Distance</td>
<td>-</td>
</tr>
<tr>
<td>Holdown Type</td>
<td>Specially designed for test. Welded. 4x4 posts. Two 3/4&quot; bolts (25/32&quot; holes) on post. 7/8&quot; bolt (15/16&quot; hole) to foundation.</td>
</tr>
<tr>
<td>Shear Wall Aspect Ratio (L/H)</td>
<td>1:1</td>
</tr>
<tr>
<td>Sheathing Material (1 or 2 faces)</td>
<td>One face, 15/32&quot; struc 1 plywood, 16d common @ 4&quot; o/c., studs at 16&quot;</td>
</tr>
<tr>
<td>Dead Load</td>
<td>None</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>11,100 lb (average), LF = 2.7</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Fastener fatigue starting near corners of panel, some bearing of sheathing on test rig</td>
</tr>
<tr>
<td>Comments</td>
<td>H.D. Load @ 1.44&quot; disp = 8,150 lb</td>
</tr>
</tbody>
</table>

---

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### CUREe-Caltech Woodframe Project

**Task 1.4.1 - Anchorage of Woodframe Buildings**

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

| Publication | Applied Technology Council Report ATC-R-1, EERC 500 / A66 / R-1 |
| Title       | Cyclic Testing of Narrow Plywood Shear Walls |
| Author      | Robin Shepherd and Bryan Allred |
| Date        | 1995 |
| Sheet Number| 9 |
| Test ID     | Static 1 | Static 2 | Dynamic 1 | Dynamic 2 | Dynamic 3 |
|             | 1 test | 1 test | 1 test | 1 test | 1 test |
| Monotonic / Cyclic | Monotonic | Reversed Cyclic |
| Load Pattern | constantly increasing displacement | Cycles 3 sine waves. Amplitude start at 50% code-rated capacity, increments of 50% CRC, 25% when load levels off. |
| Rate        | 2 inches per minute | 2 Hz. |
| Method of Loading | Horizontal load at top of frame |
| Method of Attachment | 32" x 5.5" x 0.26" plate bolted to overhang of double 2 x 4 top plate |
| Eccentricities | |

| Width (side member) | 2 x 4 |
| Foundation (main member) | R/C |
| Wood Species | Douglas Fir |
| Bolt Size | two 5/8" Hill expansion anchor bolts |
| Bolt Spacing | Symmetric, hold-down: 15" center to center, anchor bolts: 9" center to center |
| Washer Size | |
| Bolt End Distance | |
| Holdown Type | 9", 5000 lb capacity, typical L-shaped, 3/4" bolt, two 3/4" bolts (3/4" hole) in each post. |
| Shear Wall Aspect Ratio (L/H) | 1:3.5 |
| Sheathing Material (1 or 2 faces) | 3/8" Struct. 1 plywood (one side). 6d common @ 4" o.c. on edge, 12" o.c. field |
| Dead Load | |
| Ultimate Load | 2243 lb @ 8.75", 2.17" uplift, 2249 lb @ 8" horizontal displacement, 2" uplift |
| Failure Mode | Sill plate splitting at corner edge nailing and hold-down bolt. Boundary element fracture at bolt line. |
| Comments | 2 x 4 frame, double boundary elements and top plate. Negligible horizontal slip. |
# Anchorage of Woodframe Buildings

## Objective:
Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

<table>
<thead>
<tr>
<th>Publication</th>
<th>Earthquake Spectra v.19, no.3. EERC 400/E2395/v.10(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Narrow Plywood Shear Panels</td>
</tr>
<tr>
<td>Author</td>
<td>Schmidt, Ben. Richard Nielsen &amp; Robert Linderman</td>
</tr>
<tr>
<td>Date</td>
<td>Aug-94</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>10</td>
</tr>
<tr>
<td>Test ID</td>
<td>1</td>
</tr>
<tr>
<td>Brief Summary</td>
<td>CA school construction standards. Round Washer, Dead Load</td>
</tr>
<tr>
<td></td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Field construction tolerances. Round washer, Dead Load</td>
</tr>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>CA school construction standards. Square washers no Dead Load</td>
</tr>
</tbody>
</table>

### Loading Information

- **Monotonic / Cyclic:** reversed cyclic
- **Load Pattern:** 3 Stages. 0.2" increments to 1". 5 cycles at 1". 0.2" increments to 3" and final increment @ 4.3" (jack limit)
- **Rate:** quasi-static
- **Method of Loading:** Horizontal shear load attached at top. Hydraulic activator.
- **Method of Attachment:** connected to overhang of double 2x4 top plate. Lateral support frame
- **Eccentricities:**
- **Comments:**

### Full-scale shear wall tests

- **Sill Thickness & Width (side member):** 3 x 4 (2-1/2" x 3-1/2" net)
- **Foundation (main member):** 17" x 14" R/C beam (f_o = 2670, #4 & #5 bars). Dry packed with silka groud 212
- **Wood Species:** Doug Fir. 2x4: no. 1, sill: pressure treated
- **Bolt Size:** two 5/8" anchor bolts (11/16" hole)
- **Bolt Spacing:** Stacked 2-1/2" diam. x 3/16" and 1-3/4" diam. x 3/32"
- **Washer Size:** 3/4" diam. x 3/32"
- **Bolt Edge Distance:** 2-1/2" square x 1/4"
- **Bolt End Distance:**
- **Holdown Type:** Commercially available, L-shaped. Base of hold-down 9" from foundation. Two 3/4" bolts (13/16" hole) in post. 3/4" L bolt (7/8" hole) to foundation.
- **Shear Wall Aspect Ratio (L/W):** 1:2
- **Sheathing Material (1 or 2 faces):** 1/2" plywood: "APA Rated Sheathing" (one face)
- **Dead Load:**
  - 500 psf
  - 500 psf
- **Ultimate Load:**
  - not tested. Test reached 4.5 kips
  - not tested. Test reached 5.5 kips
  - not tested. Test reached 4.5 kips
- **Failure Mode:**
  - Uplift leading to nail pullout in area of uplift. Permanent deformation in hold-down anchor
  - Stiffer behaviour due to proper tightening of bolts. Higher load due to dead load.
  - More flexible due to looser bolts. Stiffer due to proper torque on bolts. Lower capacity due to lack of dead load
- **Comments:**

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**CUREe-Caltech Woodframe Project**

**Task 1.4.1 - Anchorage of Woodframe Buildings**

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests

<table>
<thead>
<tr>
<th>Publication</th>
<th>APA Report 154, Technical Services Division</th>
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<tbody>
<tr>
<td>Title</td>
<td>Structural Panel Shear Walls</td>
</tr>
<tr>
<td>Author</td>
<td>John R. Tissel</td>
</tr>
<tr>
<td>Date</td>
<td>Jul-90</td>
</tr>
<tr>
<td>Sheet Number</td>
<td>11</td>
</tr>
<tr>
<td>Test ID</td>
<td></td>
</tr>
<tr>
<td>Brief Summary</td>
<td>31 tests. Stapled and Panel over Gypsum 12 Tests. Double sheathing</td>
</tr>
</tbody>
</table>

**Loading Information**

<table>
<thead>
<tr>
<th>Monotonic / Cyclic</th>
<th>Load / Unload</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Pattern</td>
<td>Load to design shear, twice design shear, failure. Unload each time.</td>
</tr>
<tr>
<td>Rate</td>
<td>quasi-static. Wait five minutes between loadings</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Horizontal load applied via timber member attached to top plate</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>Timber member bolted to top plate</td>
</tr>
<tr>
<td>Eccentricities</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td>The test data subtracts uplift component of deflection readings</td>
</tr>
</tbody>
</table>

**Full-scale shear wall tests**

<table>
<thead>
<tr>
<th>Sill Thickness &amp; Width (side member)</th>
<th>mostly 2x4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation (main member)</td>
<td>timber as per ASTM E72</td>
</tr>
<tr>
<td>Wood Species</td>
<td>Kiln Dried Douglas Fir</td>
</tr>
<tr>
<td>Bolt Size</td>
<td>as per ASTM E72</td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td>as per ASTM E72</td>
</tr>
<tr>
<td>Washer Size</td>
<td>1/4&quot; x 3&quot; x 3&quot;</td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td></td>
</tr>
<tr>
<td>Bolt End Distance</td>
<td>very long</td>
</tr>
<tr>
<td>Holdown Type</td>
<td>Bolts extend from R/C base and attach to frame via a plate that bears in compression on the top of the top plate. There are no holes in the sill plate due to the hold-down.</td>
</tr>
<tr>
<td>Shear Wall Aspect Ratio(L/H)</td>
<td>1:1</td>
</tr>
<tr>
<td>Sheathing Material (1 or 2 faces)</td>
<td>one, various materials two, various materials</td>
</tr>
<tr>
<td>Dead Load</td>
<td>none</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>1.1 kips average 3.1 kip average</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>fastener withdrawal crushing in bearing area between boundary element and sill plate</td>
</tr>
<tr>
<td>Comments</td>
<td>staples</td>
</tr>
</tbody>
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CUREe-Caltech Woodframe Project  
Task 1.4.1 - Anchorage of Woodframe Buildings  

Objective: Evaluate performance of sill plate-to-foundation type connections for use in shear walls in full-scale shear wall tests  

<table>
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<tr>
<td>Publication</td>
<td>APA Technical Services Division, Report 105</td>
</tr>
<tr>
<td>Title</td>
<td>Plywood Shear Walls</td>
</tr>
<tr>
<td>Author</td>
<td>Noel R. Adams</td>
</tr>
<tr>
<td>Date</td>
<td>Jan-87</td>
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<tr>
<td>Sheet Number</td>
<td>12</td>
</tr>
<tr>
<td>Test ID</td>
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<tr>
<td>Brief Summary</td>
<td>Summary of 39 tests with different sheathing materials, fasteners and configurations</td>
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<table>
<thead>
<tr>
<th>Loading Information</th>
<th></th>
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</thead>
<tbody>
<tr>
<td>Monotonic / Cyclic</td>
<td>load / unload</td>
</tr>
<tr>
<td>Load Pattern</td>
<td>3 cycles of load / unload at 1200 lb, 2400 lb and failure. 400 lb increments</td>
</tr>
<tr>
<td>Rate</td>
<td>400 lb / min</td>
</tr>
<tr>
<td>Method of Loading</td>
<td>Top plate attached to 2 x 6, attached in turn to horizontal actuator</td>
</tr>
<tr>
<td>Method of Attachment</td>
<td>2 x 6 bolted (3 bolts) to specimen</td>
</tr>
<tr>
<td>Eccentricities</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Full-scale shear wall tests</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Sill Thickness &amp; Width (side member)</td>
<td>2 x 4</td>
</tr>
<tr>
<td>Foundation (main member)</td>
<td>Timber: 6 x 6 bolted to floor</td>
</tr>
<tr>
<td>Wood Species</td>
<td>Frame is Douglas Fir 11-15% moisture</td>
</tr>
<tr>
<td>Bolt Size</td>
<td>as per ASTM E72-61</td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td>as per ASTM E72-61</td>
</tr>
<tr>
<td>Washer Size</td>
<td>1/4&quot; x 3&quot; x 3&quot;</td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td></td>
</tr>
<tr>
<td>Bolt End Distance</td>
<td>long</td>
</tr>
<tr>
<td>Holdown Type</td>
<td>Bolts extend from R/C base and attach to frame via plate that bears in compression on top plate. There are no holes in sill plate due to the hold-down.</td>
</tr>
<tr>
<td>Shear Wall Aspect Ratio(L/H)</td>
<td>1:1</td>
</tr>
<tr>
<td>Sheathing Material (1 or 2 faces)</td>
<td>various types, one face</td>
</tr>
<tr>
<td>Dead Load</td>
<td>none</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>8.4 kips average</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Mostly: nails pulling through plywood. In a few cases (when sheathing bore on test rig) there was bearing failure.</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX D

Post-Earthquake Data
### CUREe-Caltech Woodframe Project

#### Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls in **post-earthquake documentation**

<table>
<thead>
<tr>
<th>Publication</th>
<th>Wood Buildings (Chapter 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Hall, John F. (primary contributor)</td>
</tr>
<tr>
<td>Date</td>
<td>Jan-96</td>
</tr>
<tr>
<td>Building ID</td>
<td>Construction Deficiencies, Northridge Earthquake.</td>
</tr>
<tr>
<td>Earthquake Record</td>
<td>Northridge Earthquake</td>
</tr>
<tr>
<td>Response Spectrum</td>
<td>Santa Clarita</td>
</tr>
<tr>
<td>Sill Thickness &amp; Width (side member)</td>
<td>165 two-story duplex condo units, built 1991, plywood shearwall</td>
</tr>
<tr>
<td>Foundation (main member)</td>
<td>Reinforced Concrete, some with little reinforcement</td>
</tr>
<tr>
<td>Wood Species</td>
<td></td>
</tr>
<tr>
<td>Bolt Size</td>
<td></td>
</tr>
<tr>
<td>Bolt Spacing</td>
<td></td>
</tr>
<tr>
<td>Washer Size</td>
<td></td>
</tr>
<tr>
<td>Bolt Edge Distance</td>
<td></td>
</tr>
<tr>
<td>Bolt End Distance</td>
<td></td>
</tr>
<tr>
<td>As-Built Deficiencies</td>
<td>Missing and wrong size nuts and washers. Anchors located too close to edge of foundation.</td>
</tr>
<tr>
<td>Holdown Type</td>
<td></td>
</tr>
<tr>
<td>Shear Wall Aspect Ratio(L/H)</td>
<td>1:3 typically</td>
</tr>
<tr>
<td>Sheathing Material (1 or 2 faces)</td>
<td>One Face</td>
</tr>
<tr>
<td>Dead Load</td>
<td></td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Cracked slab, failed plywood wall, cracked stem wall, split posts and sill plate at hold-downs.</td>
</tr>
<tr>
<td>Comments</td>
<td></td>
</tr>
<tr>
<td>Report Information</td>
<td>Publication</td>
</tr>
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</tr>
<tr>
<td></td>
<td>Title</td>
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<tr>
<td></td>
<td>Author</td>
</tr>
<tr>
<td></td>
<td>Date</td>
</tr>
<tr>
<td></td>
<td>Building ID</td>
</tr>
<tr>
<td>Brief Summary</td>
<td></td>
</tr>
</tbody>
</table>

| Earthquake Record   |             |                     |                     |                          |
|                     | Response Spectrum |             |                     |                          |
|                     | Sa           |                     |                     |                          |
|                     | Comments     |                     |                     |                          |

| Earthquake          | San Fernando, February 9, 1971 | San Fernando, February 9, 1971 | Loma Prieta, October 17, 1989 |
| Building Location   | San Fernando Valley            | San Fernando Valley            | Los Gatos                   |
| Building Type       | Dwelling                        | Dwelling                        | House                       |
| Sill Thickness & Width (side member) | 2 x 4 (from image) |                     |                          |
| Foundation (main member) | R / C (from image) | R / C                           |                          |
| Wood Species        |                             |                                 |                             |
| Bolt Size           | 1/2" approx (from image)     |                                 |                             |
| Bolt Spacing        |                             |                                 |                             |
| Washer Size         | Round Washer approx. 1" (from image) | Round Washer | |
| Bolt Edge Distance  | approx 1/2" (from image)     |                                 |                             |
| Bolt End Distance   |                             |                                 |                             |
| As-Built Deficiencies | bolt clearly off center – probably cause for bottom plate split | | |
| Holdown Type        |                             |                                 |                             |
| Shear Wall Aspect Ratio ([H] | | | |
| Sheathing Material (1 or 2 faces) | 1 face | | |
| Dead Load           | crack in bottom plate through bolt line, framing sliding off bottom plate. | Uplift of wall and bottom plate. Anchor bolts pulled through bottom plate. | Cripple wall toppled laterally causing house collapse |
| Failure Mode        |                             |                                 |                             |
| Comments            |                             |                                 |                             |
# CUREe-Caltech Woodframe Project

## Task 1.4.1 - Anchorage of Woodframe Buildings

**Objective:** Evaluate performance of sill plate-to-foundation type connections for use in shear walls using post-earthquake documentation

<table>
<thead>
<tr>
<th>Publication</th>
<th>Earthquake Damage Analysis Corporation. Reference # 66 10 06.1 SME/SHE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Title</td>
<td>Report on Smerling Office Building</td>
</tr>
<tr>
<td>Author</td>
<td>Shepherd, Rob &amp; Samuel Solkin</td>
</tr>
<tr>
<td>Date</td>
<td>24-Feb-97</td>
</tr>
<tr>
<td>Building ID</td>
<td></td>
</tr>
<tr>
<td>Brief Summary</td>
<td>Damage to timber and steel components. Authors made no calculations; report is on observations only.</td>
</tr>
</tbody>
</table>

### Earthquake Information
- **Earthquake:** Northridge
- **Building Location:** 14101 Valley Heart Drive, Sherman Oaks, CA
- **Building Type:** 2 story office. Steel braced frame and MRF. Perimeter plywood shear wall
- **Foundation (main member):** R/C
- **Wood Species:**
- **Bolt Size:**
- **Bolt Spacing:**
- **Wafer Size:** Washers: round about 3 times diameter bolt, 2 times diameter nut (estimated from picture)
- **Bolt Edge Distance:**
- **Bolt End Distance:**
- **As-Built Deficiencies:** None Observed
- **Holdown Type:**
- **Shear Wall Aspect Ratio (L/W):**
- **Sheathing Material (1 or 2 faces):** Plywood (one side)
- **Dead Load:** Typical of two story office building
- **Failure Mode:** Splitting of sill plate along bolt line. Movement of sill plate. Bolt deformation. Crushing under washer (picture)

### Commentary

---

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APPENDIX E

Test Apparatus
TYPICAL TEST SET-UP
APPENDIX F

Test Matrix
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test Variable/Wood Species</th>
<th>Nominal Sill Plate Size</th>
<th>Bolt Diameter</th>
<th>Nut/Washer</th>
<th>Vertical Load</th>
<th>Lateral Loading Notes</th>
<th>Notes</th>
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<tr>
<td>1 &amp; 6</td>
<td>Hem Fir</td>
<td>2x4</td>
<td>1/2</td>
<td>Standard nut &amp; washer</td>
<td>100 lb-ft</td>
<td></td>
<td></td>
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<td>1/2</td>
<td>Standard nut &amp; washer</td>
<td>100 lb-ft</td>
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<td>3</td>
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<td>Standard nut &amp; washer</td>
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<td>Standard nut &amp; washer</td>
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<td>1/2</td>
<td>Standard nut &amp; washer</td>
<td>100 lb-ft</td>
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<td>7</td>
<td>Douglas Fir</td>
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<td>1/2</td>
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<td>Standard nut &amp; washer</td>
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<td>14</td>
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<td></td>
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<td>Smooth rod &amp; washer</td>
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<td>Alternate: Holdown w/ 100 lb-ft vertical load</td>
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<td>Alternate: Holdown w/ 100 lb-ft vertical load</td>
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<td>Alternate: Holdown w/ 100 lb-ft vertical load</td>
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<td>1/8 inch oversize holes</td>
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<td>Full plate-width washer &amp; nut</td>
<td>100 lb-ft</td>
<td>1/8 inch oversize holes w/ epoxy</td>
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<td>Slanted bolt</td>
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<td>Full plate-width washer &amp; nut</td>
<td>100 lb-ft</td>
<td>1/4 inch oversize holes</td>
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<td>30</td>
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<td>Full plate-width washer &amp; nut</td>
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<td>1/4 inch oversize holes w/ epoxy</td>
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<td>31</td>
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<td>5/8</td>
<td>Standard nut &amp; beveled plate washer</td>
<td>100 lb-ft</td>
<td>Slanted bolt</td>
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<td>32 &amp; 33</td>
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<td>5/8</td>
<td>Slotted Plate washer &amp; nut</td>
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<td>Bolt 1/2 inch off center</td>
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<td>100 lb-ft</td>
<td>Bolt 1 inch off center</td>
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<td>Slotted Plate washer &amp; nut</td>
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<td>Bolt 1-1/2 inch off center</td>
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<td>37 &amp; 38</td>
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<td>5/8</td>
<td>Standard nut &amp; washer</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - HTT22</td>
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<td>39 &amp; 40</td>
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<td>Full plate-width washer &amp; nut</td>
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<td>41</td>
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<td>5/8</td>
<td>Full plate-width washer &amp; nut</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - HTT22</td>
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</tr>
<tr>
<td>42</td>
<td>Hem Fir</td>
<td>3x4</td>
<td>5/8</td>
<td>Oversize round or plate washer &amp; nut</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - PHD5</td>
<td></td>
</tr>
<tr>
<td>43</td>
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<td>5/8</td>
<td>Full plate-width washer &amp; nut</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - PHD5</td>
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<tr>
<td>44</td>
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<td>3x4</td>
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<td>Oversize round or plate washer &amp; nut</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - PHD5</td>
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<tr>
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<td>Full plate-width washer &amp; nut</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - HTT22</td>
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</tr>
<tr>
<td>46 &amp; 47</td>
<td>Hem Fir</td>
<td>2x6</td>
<td>5/8</td>
<td>Standard nut &amp; washer</td>
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<tr>
<td>48</td>
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<td>100 lb-ft</td>
<td>Holdown at end stud - HTT22</td>
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<td>5/8</td>
<td>Standard nut &amp; washer</td>
<td>100 lb-ft</td>
<td>Holdown at end stud - HTT22</td>
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</tr>
</tbody>
</table>

Note: Standard cut washer = 1.5 inch diameter, Oversized plate washer = 2.0 inch square, Full plate-width washer = 2.5 inch square, Slotted plate = Simpson LBPS 5/8
Task No. 1.4.1.2 - Experimental Program to Study Anchorage of Hillside Woodframe Buildings

Literature Review

Yan Xiao
University of Southern California
Los Angeles, California

Li Xie
University of Southern California
Los Angeles, California

Abstract

This report presents the state of the art review on retrofit of hillside buildings for earthquake resistance, in which the structural characteristics that make hillside buildings hazardous, and the main regulations and important concepts in the Voluntary Existing Hillside Buildings Retrofit Ordinance (1996) of Los Angeles City are discussed. An actual retrofit project on a hillside house in Los Angeles is also reviewed. Finally, a testing program for study on anchorage of wood-frame hillside buildings is introduced.

Acknowledgements

The authors would like to thank Mr. R.P. Sonntag; Mr. N. Roselund; and Mr. S. Perlof for their valuable inputs to this project.
Introduction

Hillside dwellings are popular in the USA. There are many reasons why people prefer hillside homes. The followings are a few [1]:

1. More privacy than in the crowded cities or the usual suburb.
2. Attractive country atmosphere.
4. Abundant wildlife nearby, etc.

Although the cost of construction in the hills is generally higher than that on level ground, high profits can be achieved if the land is carefully selected and the design and development are intelligently done.

However, there are many evidences that indicate a need for retrofit of existing hillside buildings in high seismic zone as in California, especially those designed under guidance of older codes. In recent years, an effort has been devoted to establish standards for retrofit of hillside buildings. However, there is still a lack of sufficient experimental data to verify the performance and reliability of hillside buildings after retrofit. Further experimental investigations on seismic performance of wood hillside buildings, especially on the connections between the base-level diaphragm and the uphill foundation, are needed. This report presents the state of the art review on retrofit of hillside buildings for earthquake resistance. Emphasis is on discussion of the structural characteristics that make hillside buildings vulnerable to earthquake damage, and the main regulations and important concepts in the Voluntary Existing Hillside Buildings Retrofit Ordinance (1996) of Los Angeles City [2]. An actual retrofit project of a hillside house in Los Angeles is reviewed [3]. Finally, a testing program for study on anchorage of wood-frame hillside buildings is introduced.

Characteristics of Hillside Buildings

In the 1994 Northridge earthquake more than 300 hillside dwellings suffered structural damage, including destructive collapse of 14 hillside buildings with the loss of five lives [4]. It is estimated that there are approximately 10,000 existing hillside buildings in the City of Los Angeles. A significant portion of them may be hazardous and susceptible to life-threatening collapse or severe structural damage in event of a major earthquake.

Fig. 2.1 to Fig. 2.5 are simple illustrations of main types of configurations of hillside buildings. The first type is the simplest one with one level along the street. The second type is one level at the street and another one story up. The third type is one level at the street and another one story down. The fourth type is one level along a sloped street with garage at the same level. The fifth type is one level along a sloped street with garage at the basement.

One of the common characteristics of structural damage unique to hillside buildings is related to the connections between the grade beams at the uphill side and the base-level diaphragms. The
differences between hillside buildings and other buildings that are on a level ground can be illustrated by a comparison of sketches of a simple box type building with flexible shear walls below the lowest diaphragm (Fig. 2.6) and a similar building on a hillside foundation (Fig. 2.7). The characteristics of the hillside building are discussed below.

**Figure 2.1** One-story hillside building

**Figure 2.2** Two-story hillside building

**Figure 2.3** Hillside building with one-story down
Figure 2.4  Hillside building with garage at the same level

Figure 2.5  Hillside building with garage at the basement
Two types of earthquake forces are considered critical to the deformations of a hillside building [4]:

**Earthquake Forces Acting in the Downhill Direction**

Compared to the flexible wood shear walls and diaphragms, concrete grade beams can be assumed rigid. The horizontal deflection of the flexible base-level diaphragm in the downhill direction under earthquake forces, as shown in Fig. 2.8, is not compatible with the deflection that the rigid uphill grade beam or the connections of the base-level diaphragm to the grade beam can allow. This leads to internal forces in the connection elements. When the internal forces exceed the capacities of the elements, damage will occur. Several types of damage to the connection elements are shown in Fig. 2.10 to Fig. 2.11. Damage to these elements may lead to loss of vertical support for the framing at the uphill side. This weakness can be compounded by damage
to the connection elements due to deterioration of the wood and the metal connectors: rotted wood and corroded anchor bolts and nails caused by water from many years of poor yard drainage or heavy garden irrigation.

The horizontal deflections of the base-level diaphragm have the following two sources:

1. **Flexibility of the diaphragm**
   As the diaphragm deflects between the vertical shear resisting elements below it, the mid-span of the uphill edge of the diaphragm may deflect away from its support at the uphill side.

2. **Flexibility in the shear walls below the diaphragm**
   The entire diaphragm may deflect away from its support at the uphill side due to drift or damage in the vertical bracing elements below the diaphragm. Typical vertical bracing elements that produce damaging drift include the following three types:
   (a) Tension-only rod bracing that yields or fails;
   (b) Flexible wood-framed shear walls that allow excessive drift;
   (c) Weak shear walls that fail in shear resulting in excessive drift.

Shear walls on stepped footing belongs to those weak shear walls of type three. There has been no testing to establish design criteria for this kind of shear walls. The damage found in earthquake-shaken hillside buildings has shown that these shear walls are subject to non-uniform distribution of internal forces. Highest internal forces are produced in the nails and anchor bolts at the uphill step. If failure occurs in the most highly loaded uphill connectors, damage quickly progresses down-slope as forces are redistributed within the shear wall assembly.

**Earthquake Forces Acting in an Across-the-Slope Direction**

Due to flexibility in or damage to the vertical bracing system at the downhill side under the base-level diaphragm, earthquake forces acting in an across-the-slope direction leads to rotation of the base-level diaphragm, as shown in Fig. 2.9. Rotation of the diaphragm also leads to differential deformations that can result in damage to the connection elements along the grade beam at the uphill side, the same as those discussed above.
Figure 2.8  Deflection under downhill earthquake forces

Figure 2.9  Rotation under across-the-slope earthquake forces
Figure 2.10

(a) Sill bolted to a grade beam
(b) Offset joists
(c) Offset joists & broken sill

Figure 2.11

(a) Ledger support for floor joists
(b) Offset joists
Retrofit Ordinance of Los Angeles City

The Voluntary Existing Hillside Buildings Retrofit Ordinance (1996) of Los Angeles City was proposed after the 1994 Northridge earthquake. The goal of retrofit is to identify structural hazards and to mitigate the hazards at reasonable cost. The seismic response of an existing hillside building can be greatly improved by strengthening the connections between the base-level diaphragm and the uphill foundation and by reducing the flexibility and improving the strength of the vertical bracing system under the base-level diaphragm.

Scope

The retrofit Ordinance is applicable to all existing buildings on or into slopes steeper than 3 horizontal to 1 vertical, constructed on either uphill or downhill sites. Such buildings have been recognized as life hazardous as a result of partial or complete collapse that occurred during the Northridge earthquake. Seismic strengthening constructed prior to the effective date of the Ordinance shall also be evaluated and modified in accordance with the provisions of this Ordinance.

Definitions

Certain terms are defined in the Ordinance as follows:

**Base-Level Diaphragm:** The floor at or closest to the top of the highest level of the foundation.

**Diaphragm Anchors:** Assemblies that connect a diaphragm to the adjacent foundation at the uphill diaphragm edge.

**Primary Anchors:** Diaphragm anchors from a diaphragm strut or collector, providing a direct connection between the base-level diaphragm and the foundation at the uphill side. The diaphragm strut or collector must engage the full width of the diaphragm, thus forming the edge of the diaphragm or dividing the diaphragm into panels.

**Secondary Anchors:** Other diaphragm anchors that provide a redundant interconnection between the base-level diaphragm and the foundation at the uphill side.
Degree of Hazard

Existing hillside buildings are categorized in the Ordinance according to degree of hazard as follows:

**Category I:** the most hazardous category that includes rod-braced buildings; buildings with no shear walls below the base-level diaphragm and no primary anchors connecting the diaphragm to the foundations; and buildings with no apparent lateral force resistance.

**Category II:** those downhill buildings including buildings with sloped sill plates; buildings without foundation anchor bolts; non-shear wall buildings with connections at the foundation critical to the lateral force resistance such as braced frames or moment frames.

**Category III:** all other downhill buildings not listed in category one or two, such as downhill buildings with stepped sill plates.

**Category IV:** the least hazardous category that covers all other uphill buildings not listed in category one, including non-shear wall buildings with connections at the foundation critical to the lateral force resistance such as braced frames or moment frames.

Analysis and Design

*Base for Seismic Design*

The base for seismic design is defined as follows:

A. **Downhill Direction:** For seismic forces acting in the downhill direction, the base of the building is defined as the floor at or closest to the top of the highest level of the foundation.

B. **Normal to Downhill Direction:** For seismic forces acting normal to the downhill direction, the base of the building is defined as the lowest level of the foundation. The distribution of seismic forces over the height of the building needs to be determined. Retrofitting, however, shall only be required at the base level and below.

*Base Shear*

The allowable stress design base level lateral force shall be that required at the time of the original building permit, but not less than \( V = 0.133W \) (in LA City), where, \( W \) is the weight of the entire building.
**Downhill Base Shear Resistance**

Downhill base shear, including forces from the base-level diaphragm, shall be resisted by primary anchors between the diaphragm and the uphill foundation. Primary anchors are intended to provide the principal connections of the building to its foundation and are thus in the primary load path of seismic forces from the foundation into the superstructure. Therefore, primary anchors are designed to resist 125% of the tributary seismic forces in the downhill direction and to be spaced at a maximum 30 ft on center. The foundation must be shown to be adequate to resist the concentrated loads from the primary anchors.

Secondary anchors are intended to reduce the flexibility of the diaphragm. They shall be uniformly distributed along the uphill diaphragm edge and shall be spaced a maximum of 4 ft on center. Secondary anchors at the base level are designed for a uniformly distributed minimum force equal to the total primary anchorage design force at that level divided by the length of the uphill diaphragm edge, but not less than 300 pounds per lineal foot. The load path for secondary anchors need not be developed beyond the connection to the foundation. Secondary anchors are not required in the following special cases:

A. Foundations in the downhill direction spaced no more than 30 ft extend up to and are directly connected to the base-level diaphragm for at least 70 percent of the diaphragm depth.

B. The diaphragm is separated from the sill plate at the uphill foundation by a cripple wall which has anchor bolts and is braced in the plane of the wall and constructed with studs that are not less than 12 inches in height. Primary anchors are spaced a maximum of 20 ft on center.

C. Deflection of the plywood floor diaphragm between adjacent primary anchors is calculated to be less than 1/4 inch.

Wood diaphragm struts, collectors and other wood members connected to primary anchors shall not be less than 3X members or doubled 2X members fastened together. Secondary anchors may be developed through 2X framing members. All primary and secondary anchorage including struts, splices and collectors must be designed for 125 percent of the tributary force.

The seismic load factor shall be 1.7 for steel, wood and concrete anchorage when the strength design method is used. When the allowable stress design method is used, the one-third allowable stress increase is not permitted.

**Systems Normal to the Downhill Direction**

Vertical bracing system normal to the downhill direction below the base-level diaphragm at the downhill side is required to limit diaphragm rotation under the seismic forces normal to the downhill direction. The inter-story drift below the base-level diaphragm shall not exceed 0.005 times the story height. The total drift from the base level to the foundation shall not exceed 3/4
inch. The story height shall be measured from the average height of the top of the foundation to the story level.

Alternate Lateral Force Resisting Systems

As an alternative to providing primary anchor connections from the diaphragm to the foundation in the downhill direction and as options for resisting lateral forces normal to the downhill direction, the following systems may be used:

A. Wood Structural Panel Shear Walls:

Wood structural panel shear walls may be used provided:

a. The minimum length of shear wall shall be 8 ft.

b. The minimum level length between steps in the shear wall sill shall be 8 ft and the maximum step height between adjacent sills shall be 2'-8".

c. Sill plates do not slope and they bear on a level surface.

d. The design lateral forces shall be distributed to lateral force-resisting elements of varying heights in accordance with the stiffness of each individual element. The stiffness of a stepped wood shear wall may be determined by dividing the wall into adjacent rectangular elements, subject to the same top deflection. Sheathing and fastening requirements for the stiffest section shall be used for the entire wall. Each section of wall shall be anchored for shear and uplift at each step as an independent shear wall.

e. The drift is limited to a maximum of 0.005 times the story height.

B. Braced Frames:

Structural steel braced frames with concentric connections may be used as part of the lateral force-resisting system. All members in braced frames shall be designed to resist tension and compression forces. Seismic forces shall not induce flexural stresses in any member of the frame, in diaphragm struts, or in the connectors.

C. Rod-Braced Frames:

Tension-only bracing performed very poorly during the Northridge earthquake. Existing tension-only bracing may be included in the lateral load resisting system provided its components are exposed and tested to confirm their capacity to resist 5 times the design force and that the connections have the capacity to resist the yield strength of the bracing.
D. Steel Moment Frames:

Lateral force-resisting systems normal to the downhill direction may include steel moment frames in addition to those mentioned above provided the drift does not exceed 0.005 times the story height.

**Diaphragms**

Diaphragms at the base level and below may be of straight 1x6 or 2x6 sheathing provided downhill vertical lateral force-resisting elements or primary anchors are spaced no more than 20 ft apart and the diaphragm shear forces do not exceed 100 pounds per lineal foot. Existing plywood and diagonally sheathed diaphragms need not be investigated. Existing cantilevered wood diaphragms are acceptable provided they do not cantilever more than one half of the diaphragm back span (anchor span).

**Foundations**

All intersecting foundations shall be positively anchored where separation has occurred. Damaged foundations shall be evaluated by the engineer or architect and, if found to reduce the capacity of the vertical and lateral force-resisting system, be repaired or replaced. Metal connectors shall not be in contact with, or below earth unless the connectors are hot dipped galvanized and further protected from earth with 4 inches of concrete.

**Existing Materials**

Existing materials may be used as part of the lateral load-resisting system provided that the stresses in these materials do not exceed the values shown below:

- **Plain or reinforced concrete footings:**
  \[ f_{c'} = 2000 \text{ psi unless otherwise shown by tests.} \]

- **Douglas fir wood:**
  Allowable stress same as No. 2 D. F.

- **Reinforcing steel:**
  \[ f_t = 16,000 \text{ psi maximum} \]

- **Structural steel:**
  \[ f_t = 22,000 \text{ psi maximum} \]

- **Anchor bolts:** Current code values.

- **Wood structural panels:** Current code values.
An Example of Retrofit Projects

An actual retrofit project is reviewed in this section, referring to the retrofit Ordinance (1996). This retrofit project was completed in 1990, prior to the effective date of the retrofit Ordinance. It is located in the City of Los Angeles. Plans and elevations of the existing house are shown in Fig. 4.1 to Fig. 4.8.

The degree of hazard of the existing building falls in category three. The downhill slope is approximately 1:2. Some cracks were observed in walls and foundations around the interconnection between the original house and the addition. The existing footings were replaced by reinforced concrete grade beams with six piles below them. The new foundation plan is shown in Fig. 4.9. A section of the house after retrofit is shown in Fig. 4.10. The existing shear walls below the base-level diaphragm were all replaced by new wood shear walls.

The maximum distance between lateral force-resisting elements (shear walls) in the downhill direction before retrofit is 37 ft, larger than 30 ft (the maximum distance permitted by the Ordinance). After retrofit, the maximum distance is 25 ft, satisfying the requirement of the Ordinance.

There were no secondary anchors in the existing building. New secondary anchors were added and spaced 32 inches apart, less than 48 inches (the requirement of the Ordinance). Fig. 4.11 is an illustration of the secondary anchor at the uphill diaphragm edge.

For a preliminary check of this retrofit project, uniformly distributed dead loads for the roof, the floor, and the exterior and inner walls are assumed to take adequate values in common use as shown below:

- **Roof dead load:**
  - 15 psf on a horizontal plane

- **Floor dead load:**
  - 20 psf

- **Exterior wall dead load:**
  - 15 psf

- **Inner wall dead load:**
  - 10 psf

- **Weight of the roof:**
  - 24 x 62 x 15 = 22320 pounds
Weight of the floor:
\[ 21 \times 60 \times 20 = 25200 \text{ pounds} \]

Weight of exterior walls in half a story above the base level:
\[ 162 \times 8.5 / 2 \times 15 = 10328 \text{ pounds} \]

Weight of exterior walls in half a story below the base level:
\[ 81 \times 12.75 / 2 \times 15 = 7746 \text{ pounds} \]

Weight of inner walls in half a story above the base level:
\[ 124 \times 8.5 / 2 \times 10 = 5270 \text{ pounds} \]

Weight of inner walls in half a story below the base level:
\[ 56 \times 12.75 / 2 \times 10 = 3570 \text{ pounds} \]

Lumped weight in the roof level:
\[ W_2 = 22320 + 10328 + 5270 = 37918 \text{ pounds} \]

Lumped weight in the floor level:
\[ W_1 = 25200 + 10328 + 7746 + 5270 + 3570 = 52114 \text{ pounds} \]

Total weight:
\[ W = 37918 + 52114 = 90032 \text{ pounds} \]

(1). Base shear in downhill direction:

\[ h_n = 8.5 \text{ ft} \]

\[ T = 0.02h_n^{3/4} = 0.1 \text{ sec} \]

\[ C_a = 0.44 \]

\[ C_v = 0.64 \]

\[ T_s = C_v/(2.5C_a) = 0.58 \text{ sec} > T = 0.1 \text{ sec} \]
\[ R = 4.5 \]
\[ I = 1.0 \]
\[ V = \frac{(2.5C_a I/R)W}{1.4} = 0.174W > 0.133W \]
\[ \therefore V = 0.174 \times 90032 = 15700 \text{ pounds} \]

When the allowable stress design method is used, the total design base shear for the secondary anchors or the wood shear walls below the base-level diaphragm in the downhill direction shall be

\[ V_d = 1.25 \times 15700 = 19625 \text{ pounds} \]

(2). Base shear normal to the downhill direction:

\[ h_n = 21.3 \text{ ft} \]
\[ T = 0.02(h_n)^{3/4} = 0.2 \text{ sec} \]
\[ T_s = 0.58 \text{ sec} > T = 0.2 \text{ sec} \]
\[ \therefore V = 0.174 \times 90032 = 15700 \text{ pounds} \]

The total design base shear is the same as that in the downhill direction.

The capacity of each secondary anchor may be estimated as 900 pounds and there should be at least 25 secondary anchors. So, the total capacity of secondary anchors is 22500 pounds > 19625 pounds. The wood shear walls below the base-level diaphragm can also be shown to have sufficient capacity to resist the design base shears. Based on the above discussion, this retrofit project can be concluded as successful, though it was not checked in more details.
Figure 4.1  Plot Plan

Figure 4.2  Ground Floor Plan
Figure 4.3  Foundation Plan

Figure 4.4  Roof Plan
Figure 4.5  South Elevation

Figure 4.6  North Elevation
Figure 4.7  East Elevation

Figure 4.8  West Elevation
Figure 4.9  New Foundation Plan (After retrofit)

Figure 4.10  Section A (After retrofit)
Figure 4.11  Secondary Anchors
Testing Program

Based on extensive discussions with practicing engineers, who were involved in developing the Los Angeles City Retrofit Ordinance, as well as the discussions during the January 22 workshop, an experimental program has been defined. The objective of the program is to provide experimental background for improving the retrofit design. Three to four large-scale tests of base-level diaphragm to uphill foundation models are planned. The test setup is shown in Fig. 5.1. Analysis is currently underway to provide the pre-test prediction. The testing objectives and details for each specimen are discussed below (Fig. 5.2).

Unit-1: Retrofitted diaphragm with primary and secondary anchors under equal downhill forces.
- An equal monotonic load is applied at each edge of the diaphragm simulating the downhill earthquake force along the main loading path.
- This is to investigate the efficiency, load resisting mechanisms and failure mode of primary and secondary anchors as recommended in the LA City Ordinance.

Unit-2: Retrofitted diaphragm with only secondary anchors under equal downhill forces.
- An equal monotonic load is applied at each edge of the diaphragm simulating the downhill earthquake force along the main loading path.
- To investigate resisting mechanisms of secondary anchors for the cases where primary anchors are not used;
- To understand the level of redundancy provided by secondary anchor;
- To investigate the failure mode of secondary anchors.

Unit-3: Retrofitted diaphragm with primary and secondary anchors under unequal downhill forces.
- Unequal monotonic loads are applied at the edges of the diaphragm simulating the downhill earthquake force along the main loading path. Such downhill loading is more realistic and may be more critical.
- Besides, study on this loading condition sheds lights on the resistance of the anchoring system to the rotation generated by the earthquake force perpendicular to the downhill direction. Although the Ordinance requires other lateral resisting systems below the base-level diaphragm to eliminate the rotation mode by the earthquake force perpendicular the downhill direction, the anchoring system does provide resistance, at least for redundancy.
- Under such loading condition, unequal forces are generated in the two primary anchors with one becoming more critical than the other.
- This is to investigate the efficiency, load resisting mechanisms and failure mode of primary and secondary anchors as recommended in the LA City Ordinance.
Unit-4: Retrofitted diaphragm with only secondary anchors under unequal downhill forces.

- The loading condition is similar to Unit-3, except no primary anchor.
- To investigate resisting mechanisms of secondary anchors for the cases where primary anchors are not used;
- To understand the level of redundancy provided by secondary anchor;
- To investigate the failure mode of secondary anchors.

Figure 5.1
Figure 5.1 (continued)
Figure 5.2
Figure 5.2 (continued)
Concluding Remarks

The state of the art on retrofit of hillside buildings for earthquake resistance is reviewed in this report. The structural characteristics that make hillside buildings hazardous, and the main regulations and important concepts in the Voluntary Existing Hillside Buildings Retrofit Ordinance (1995) are discussed. A real retrofit project on a hillside house in the City of LA is also reviewed and concluded as successful. Finally, a testing program for investigation on anchorage of wood-frame hillside buildings is introduced.

References


Introduction

The wood engineering community is constantly striving to enhance the designability, performance and economy of wood structures. In particular, the behavior of floor and roof diaphragms has become an important issue with respect to lateral stiffness and deflection. It has been noted that there is seldom a problem with the strength of diaphragms, because failures are predominantly associated with the connections between a diaphragm and supporting walls. Though the occurrence of actual failures is rare, diaphragms have sometimes been a controlling factor in the overall failure of structures during seismic events. Poor understanding of diaphragm behavior has spurred the interest of researchers to formulate more accurate methods of analysis and design similar to methods already employed in the design of cold-formed steel diaphragms. The objective of this report is to review major studies in this field performed on both a theoretical and experimental basis. Sporadic since the 1950’s, most of the testing of wood diaphragms has occurred at the facilities of the Douglas Fir Plywood Association (DFPA), American Plywood Association (APA), Oregon State University, Oregon Forest Products Laboratory, Washington State University, and West Virginia University.

Early Testing

The DFPA sponsored some early tests in diaphragm behavior. Countryman (1952) describes lateral tests on plywood-sheathed diaphragms. Four specimens, 12 x 40 ft. and 20 x 40 ft., and six one-quarter scale models, 5 x 10 ft., were tested by monotonic loading at fifth-points. The specimens had varying parameters such as blocking, openings, staggered panels, gluing, plywood thickness, nail size, and boundary nailing patterns. Stiffness of the diaphragms was calculated from measured lateral deflection in the middle of the lower chord and applied load. Shear deformation, and not flexural deformation was determined to be the predominant form of deflection. Load versus deflection plots show that the actual deflection was consistently higher than calculated values using existing equations. It was found that diaphragms behave like a
horizontal girder with a shear-resistant web. Chord members resist the flexural tension and compression stresses, while the web resists the shear. Strength and stiffness of the specimens was found to be primarily dependent on the strength of the nailed plywood-to-frame connections.

Due to over conservative design codes, the DFPA pursued further studies in diaphragm action. Countryman and Colbenson (1954) report on tests of fifteen full-scale diaphragms, conducted to better understand the strength effects from:

1. Omission of blocking
2. Panel arrangement
3. Nailing schedules
4. Span-thickness combinations
5. Length-width ratio
6. Seasoning of frame lumber
7. Use of three inch lumber
8. Cut-in blocking for chords
9. Load application perpendicular to joists
10. Screwed cleats in lieu of blocking

All 24 x 24 ft. specimens were monotonically loaded with four equal lateral forces at fifth points of the span and deflections were measured from the middle of the unloaded chord. Plywood thickness and nailing schedule, along with blocking to a lesser degree, were found to be the predominant factors in determining strength and stiffness. Ultimate applied shears ranged from 733 to 2530 plf, while ultimate deflections occurred from 0.52 to 3.2 in. For blocked diaphragms, the measured deflections are consistent with a formula produced as a result of the DFPA Report No. 55 (Countryman 1952) with an average error of 15%.

In conjunction with the research described above, the DFPA also sponsored tests at the Oregon Forest Products Laboratory. The two 20 x 60 ft. roof diaphragms tested, were constructed with the lightest framing and plywood thickness permissible at that time for a roof of this size (Stillinger and Countryman 1953). The 2 x 10 joists were framed at 24 in. o.c. and sheathed with 3/8-in. thick plywood. One of diaphragms was blocked along the panel edges. The diaphragms were loaded monotonically by hydraulic jacks at the fifth points. The 3/8-in. thick plywood was found to be adequate, though not as strong as specimens with thicker plywood. The lightweight framing system performed adequately enough to be considered in construction methods. Lastly, it was found that for unblocked diaphragms, no special boundary nailing detail was required regardless of the reduced strength.

The APA became interested in lateral shear testing of diaphragms not composed of Douglas Fir plywood. Tissell (1966) validated the DFPA tests from 1955 as well as going on to test diaphragms of other various species of wood that were becoming popular in construction.
Nineteen full-scale 16 x 48 ft. diaphragms were tested. Plywood characteristics, sheathing-to-framing connections, nail types, and framing member types were varied in the tests. Monotonic loading from 16 hydraulic jacks at 3 ft. on center was used to approximate a uniform lateral load. Lateral deflections were measured with dial gages at mid span of the tension chord. The design shear values were found to be very conservative, with the average ultimate load being 1545 plf and the average allowable design load being only 420 plf (includes factors of safety). Sheathing of different species of wood was found to have a small but accountable change in shear strength and stiffness. However, effects from plywood grade and quality were found to be negligible. Tissell concluded that shear strength equivalent to that of blocked diaphragms is possible by stapling tongue-and-groove 2-4-1 plywood. Further, shorter ring-shank nails are permissible as long as a minimum penetration is attained. Open-web steel joist-framed diaphragms were slightly stronger than the lumber framed diaphragms. The DFPA design values determined from the tests previously discussed (Countryman and Colbenson 1954) were found to adequately conservative.

**Dynamic Testing**

GangaRao and Luttrell (1980) explain the efforts at West Virginia University to quantify shear response of diaphragms with the ultimate goal in preparing accurate analysis models for future design purposes. Since diaphragms had been mainly studied under static loading conditions, they propose that stiffness characteristics are an equally critical issue in a correct estimation of behavior under real-life dynamic loading. Preliminary dynamic results from tests at West Virginia University were used to derive joint slip and shear deformation response equations based on dynamic loading. They predicted that damping characteristics with respect to joint slip are the critical factors needed in order to appropriately describe diaphragm behavior under dynamic loads.

At the time, Polensek (1979) was the only researcher making attempts at quantifying damping characteristics for horizontal dynamic loading. His tests of plywood sheathed diaphragms with six or ten inch joists yielded average damping ratios between 0.07 and 0.11. However, he considered that the data accumulated had been too varied for an accurate estimation of the damping ratio. It was apparent, however, that an increase in floor span is directly proportional to the damping effect.

At West Virginia University, Jewell (1981) performed experimental tests on partial (cantilever) diaphragms in order to analyze a range of different parameters such as nail spacing, boundary conditions, connection details, load type, and damping capacity. Three 16 x 24 ft. diaphragms and six 16 x 16 ft. diaphragms were tested under monotonic, cyclic, and impact loads in the directions perpendicular and parallel to the joists. Replica diaphragms were also modeled in the same configurations as flexible composite members in a finite element analysis to determine any inaccuracy in this theoretical approach. Based on a comparison of the theoretical and experimental test results, Jewell was able to analyze relationships of plywood behavior, nail slip, effect of loading, effect of joist hangers, and damping to the stiffness of diaphragms. In most cases, the finite element approach yielded slightly more conservative results for stiffness (i.e., predicted deflections were lower than actual), based on the parameters listed above.
Corda (1982) and Roberts (1983) performed additional cantilever diaphragms tests at West Virginia University in another codependent study involving laboratory testing and finite element modeling. Corda tested six 16 x 24 ft. specimens cyclically and statically to failure in order to study local and global in-plane shear stiffness response to variations of blocking, openings, plywood thickness, corner stiffeners, and framing nail sizes. It is noteworthy that nail softening after loads up to ±9 kips on some specimens caused a large decrease in stiffness. Increased plywood thickness (without using longer nails) and corner openings reduced strength but had little effect on stiffness. Roberts’ theoretical analysis of equivalent models of the diaphragms tested by Corda, showed some evident discrepancies. Problems with the finite element analysis program included the limitation to monotonic loading, inaccurate predictions of panel slip, and the iterative processes of calculation of diaphragm deflection with respect to nail slip, a bilinear relationship, demanding the modification of results to a nonlinear solution using the tangent stiffness method. Based on the problems encountered, Roberts suggested that the limitations imposed on the program user in modeling plywood diaphragms need to be eliminated by further experimental research into stiffness characteristics of panel slip, plywood layout and connection, diaphragm openings, and nail slip under cyclic loading.

A recent APA report by Tissell and Elliott (1997) describes diaphragm testing for high load conditions equivalent to earthquakes accelerations. The primary intent was to formulate design and construction approaches for these “high-load” diaphragms, which may incorporate use of two layers of plywood, thicker plywood, or stronger fastener conditions. Ten of the diaphragms tested were 16 x 48 ft., and the dimensions of an eleventh specimen were changed to 10 x 50 ft. Hydraulic jacks at a spacing of 24 in. o.c. were used to apply a cyclic uniform load along the long side of the diaphragms. Results show that it is possible to increase shear strength by increasing the number of fasteners or adding another layer of sheathing in areas of high shear. This report also notes that plywood panel shear capacity must be checked for “high-load” diaphragms. Staples were found to be adequate fasteners in lieu of nailed sheathing-to-framing connections. Along the same lines, field glued joints and a reduced number of nails are adequate for these diaphragms.

**Similar Diaphragms**

The abundant studies of floors comprised of materials other than wood are important in order to understand general behavior of diaphragms. It is possible that wood diaphragm design methods may be simplified and accurately rationalized in terms of methods already in use for other materials. A theoretical study of the behavior of composite steel beam and concrete deck diaphragms was made by Widjaja (1993) at Virginia Polytechnic Institute and State University. Similar to many efforts currently in progress for wood diaphragms, the purpose of this study was to develop an accurate finite element analysis model that predicts diaphragm behavior, incorporates possible variations of design parameters, and derives design strength equations. Similarly, experimental cantilever tests on cold-formed steel diaphragms are important in the design of many steel-frame building roofs and composite floors (during construction, before concrete). Post-frame diaphragm testing (wood frame and metal sheathing) is also significant in terms of the more agricultural or shed type buildings.
With sponsorship from NUCOR Research and Development, Hankins (1992) performed eighteen cantilever diaphragm tests at Virginia Polytechnic Institute and State University to determine strength and stiffness of cold-formed steel sheathing, 20 and 22 gage thickness (Vulcraft 1.5B1 deck), welded or bolted to a steel frame. The 16 x 16 ft. specimens were subjected to monotonic loads. Thirteen diaphragms utilized an 8 ft. span, requiring only one filler beam. The other five specimens had a filler beam spacing of 4 ft., requiring three filler beams. Bolt and puddle weld arrangement, used to secure the sheathing, was varied to determine its effects on diaphragm behavior. Results from the tests indicate that specimens with thicker gage sheathing have more strength and stiffness. However, even though specimens with smaller filler beam spacing (three filler beams as opposed to one) had more strength, the diaphragm stiffness was less in some cases.

Hausmann and Esmay (1977) report the results of tests on twenty-six full size post-frame, metal-clad diaphragm panels. All specimens were 8 x 16 ft. with rafters at 4 ft. o.c. along the 16 ft. side and purlins at 2 ft. o.c. along the 8 ft. side. Loaded monotonically at the ends of the three interior rafters, the panels were analyzed for strength and stiffness based on varying parameters such as framing arrangement, type, number, and metal of fasteners, aluminum or steel sheathing, and with or without insulation. It was determined that purlins laid flat to the rafters was the more suitable method of framing. Screw fasteners in the panel valleys increased diaphragm stiffness and strength, especially for the steel clad specimens. Aluminum panels were more suitable with nailing, due to a larger cover width for each sheet. Placing insulation between wood-framing and metal cladding not only reduces diaphragm strength, but also seriously affects stiffness. Fastener configurations have important and measurable effects on diaphragm behavior.

Anderson and Bundy (1990) performed additional post-frame diaphragm tests to outline the effects of openings in the sheathing. Fifteen cantilever specimens, 7-2/3 x 12 ft. with two interior rafters and seven purlins were tested monotonically with varying amounts of sheathing missing. Diaphragms were constructed with SPF lumber, screw fasteners, and steel sheathing.

Fastener configurations were found to be extremely important for diaphragm stiffness. Openings in the sheathing at normal intervals caused the specimens to be ineffective as diaphragms. It was also found that spacing of purlins has little impact on strength or stiffness of the diaphragms.

In addition to the physical testing, there has been a great deal of computer modeling of post-frame diaphragms for scientific purposes in order to aid designers and validate experimental results. For example, Wright and Manbeck (1993), among many others, conducted finite element analyses of post-framed diaphragm panels. Following procedures provided by Woeste and Townsend (1991), they modeled full size 8 x 12 ft. diaphragms with 2x4 in. purlins at 2 ft. o.c., 2x6 in. rafters at 3 ft. o.c., and steel cladding secured with 16d nails. The finite element model was compared to three identical experimental diaphragm tests. The finite element model closely predicted diaphragm shear strength, but under-estimated shear stiffness by 28%. Results show that discrepancies arise due to difficulties in modeling nonlinear behavior of fasteners and intricate load paths between the wood frame and steel sheathing.
Summary

Horizontal wood diaphragm behavior is a critical element in the design of safe and economical wood structures in areas of seismic activity and high wind loading. This study of research performed in this field is important in order to know what characteristics have already been established and what questions of diaphragm behavior are still unanswered. Horizontal diaphragm stiffness with respect to adequate connection to shear walls is still a difficult relationship in wood design. In accordance with Task 1.4.2 of the CUREe-Caltech Woodframe Project, attempts are being made to devise finite element analysis models and design procedures equivalent to those for cold-formed steel diaphragms that resolve this problem for wood diaphragms. Critical to this project are diaphragm stiffness characteristics, a subject that this review shows a lack of completeness. As a result Task 1.4.2 is aimed at numerical modeling and full-scale diaphragm tests to provide stiffness guidelines for design under seismic conditions.
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Introduction

Residential homes built in the 1960’s or prior in California, were frequently constructed of an elevated first floor supported by short walls consisting of wood stud framing with non-structural exterior finishes such as stucco or horizontal wood siding. These walls, known as cripple walls, range from one to four feet in height, and provide an under-floor crawl space for ventilation, access to utilities, and a level surface for construction on a sloping grade. Cripple walls typically extend around the perimeter of the structure and carry much of the first floor joist load to the foundation. In addition, cripple walls must resist the inertial force developed in the upper stories during a strong ground shaking. Older residential homes supported on cripple walls however performed rather poorly in past earthquakes. The common mode of failure involved a large lateral drift of the upper story over the soft cripple wall, causing the building to drop vertically resulting in fractures of sewer, water or gas lines. Repair of building after cripple wall failure is usually difficult and expensive.

This preliminary report provides a brief summary of the damage observed in residential homes during recent California Earthquakes, in particular those damage that is related to cripple wall failure for both level ground and hillside constructions. Details of earthquake damage to other woodframe buildings are available [EERI 1990; EERI 1996; APA 1989]. The report also summarizes the relevant experimental research conducted to-date to characterize the seismic response of cripple walls.

Past Earthquake Damage

The 1989 Loma Prieta Earthquake and 1994 Northridge Earthquake were two of the many recent earthquakes in California that caused substantial damage to wood-frame buildings. In the 1989 Loma Prieta Earthquake, approximately 2,000 residential units were lost in Watsonville, which was very close to the epicenter of the earthquake. In the same earthquake, 5100 housing units in San Francisco and more than 1000 units were seriously damaged in Oakland [EERI 1990]. In-
plane failure of cripple walls, with consequent collapse of the walls, porches and utility lines, was the main cause of these losses. Failure mostly occurred in older buildings, which were built before the introduction of more recent codes. Many of the buildings were even built before plywood was developed. In this case, only an exterior cladding was used to brace the cripple walls, which normally consisted of stucco or 1 in by 6 in (25 mm by 152 mm) wood members laid horizontally. Consequently, very small lateral resistance was provided to the cripple wall. Also insufficient or even non-existent bolting of the mudsill plate to the foundation was found in these old buildings. Another important cause of the cripple wall failure was the deterioration of the materials over the years. The high moisture present in the crawl space underneath the first floor often resulted in rusting of the bolts and severe deterioration of the wood elements.

Most of the losses during the 1994 Northridge Earthquake were due to damage or collapse of residential single-family homes, multi-family apartments or condominiums. About 96% of the residential buildings in the Los Angeles County were constructed of light wood-frame structures, and about 60,000 units were seriously damaged by the earthquake. It was estimated that more than half of the $40 billion property losses was due to failure of woodframe constructions [Kircher et al. 1997; Reitherman 1998]. Figure 1 shows the number of residential units that were red or yellow-tagged by building inspectors after the 1994 Northridge Earthquake. The total number of damaged units was 56119, with 16,269 units or 29% of total damaged units red-tagged.

**Figure 1** Red and Yellow-Tagged Residential Buildings in the Los Angeles County after the 1994 Northridge Earthquake (EERI 1996)
(a) Damage To Level Cripple Walls

Figure 2 shows a typical failure of an ‘as-built’ cripple wall for a residential building during the 1994 Northridge Earthquake. The cripple wall was finished with an exterior stucco cladding. Even though the building was repaired and strengthened after the 1971 San Fernando Earthquake, no structural bracing was added to the cripple wall. Repair of such damaged building is difficult and expensive. Such hazard however may be mitigated by structural bracing of the cripple walls, which is rather inexpensive and has been shown to be effective against cripple wall collapse [EERI 1996].

![Figure 2](image)

**Figure 2**  Failure of an Unbraced Cripple Wall during the 1994 Northridge Earthquake
(Image reproduced from EERI Reconnaissance Report, 1996)

Figures 3(a) and (b) show a similar level cripple wall failure during the 1989 Loma Prieta Earthquake. Damage to the two-story building was mainly due to a vertical drop of the building after failure of the cripple wall. The floor joists were resting on grade after the vertical drop as can be seen in Figure 3(b). Except for the possible fracture of ventilation ducts, utility lines etc., the building appeared to be relatively undamaged by the vertical drop.
Figure 3  Damage to a Two-Story Residential Building after Cripple Wall Failure
(Images from 1989 Loma Prieta Collection, Earthquake Engineering Research Center, University of California, Berkeley)
The seismic hazard associated with an unbraced cripple wall is generally recognized [Sonntag 1999], and guidelines for identifying such seismic hazard and methods for retrofit are available [e.g. CSSC 1994; OES 1995; Vukazich 1998]. A common retrofit method involves the attachment of structural wood panels to the unbraced cripple wall frame using closely spaced nails. The effectiveness of bracing was well documented in the recent 1994 Northridge Earthquake [EERI 1996]. Figure 4(a) shows a single-family building that was retrofitted just prior to the 1994 Northridge Earthquake. The retrofit scheme involved installation of plywood panels over one third length of the cripple wall, anchoring of sill-plate and strengthening of interior girder-post connections. The retrofitted building performed very well during the 1994 Northridge Earthquake, despite being fairly close to the epicenter (about 16 km). Only minor cracking of the exterior plaster and damage to the brick chimney were noted. In contrast, five similar houses in the same area, constructed during the same period and using the same design for cripple walls, suffered severe damage due to cripple wall failure. Two of the damaged houses had to be demolished due to the severity of the damage. An example of a damaged building with an unbraced cripple wall is shown in Figure 4(b).

(b) Damage to Stepped Cripple Walls in Hillside Construction

Extensive damage to hillside constructions had also been reported in recent California earthquakes [City of LA 1996; OES 1995]. Figure 5 shows the failure of a stepped cripple wall for a hillside building during the 1984 Morgan Hill Earthquake. The stepped cripple wall was essentially unbraced with the exterior cladding provided by fiberboard panels. Large across-slope displacement occurred between the first floor and concrete stepped foundation, as can be seen in Figure 5.

Two main causes were identified for the damage observed in hillside constructions [Roselund, 1996]. Figure 6 (a) shows an inertial force from the superstructure acting in the downhill direction. Large forces are developed in the connections between the horizontal diaphragm and the uphill foundation wall. Depending on the types of bracing, lateral stiffness and strength of the substructure (shearwall or beam-and-column support), damage to the sill plate connecting the horizontal diaphragm to the uphill foundation wall may occur. Also the ledger supporting the floor on the uphill foundation may be damaged. In cases where the substructure shearwalls are poorly braced, or if the substructure beam-and-column supports are braced with tension-ties only, excessive drift may occur in the substructure resulting in unseating of the entire horizontal diaphragm and hence collapse of the structure. It is also worth noting that, because of the flexibility in the horizontal diaphragm, a non-uniform distribution of forces would result in the connection between the horizontal diaphragm and foundation wall, with the largest connection force occurring at mid-span of the horizontal diaphragm.
(a) Minor Damage to a Retrofitted Single-Story Residential Building

(b) Severe Damage to a Residential Building with Unbraced Cripple Wall

**Figure 4** Comparison of Damage Between Braced and Unbraced Cripple Walls
(Images reproduced from EERI Reconnaissance Report, 1996)
Figure 5  Failure of an Unbraced Stepped Cripple Wall for a Hillside Building
(Image from 1984 Morgan Hill Collection, Earthquake Engineering Research Center, University of California, Berkeley)
Conventional construction of wood-framed buildings on a hillside often accentuates the asymmetric response of the building when compared to a similar wood-framed building constructed on level ground [Roselund 1996]. Figure 6(b) shows a torsional response of a hillside building when subjected to an across-slope inertial force. Due to unequal heights in the uphill and downhill shearwalls or beam-and-column supports, the inertial force acts at the center of mass, which is located at some eccentricity from the center of rigidity. High local stresses are therefore developed in the short panel of the stepped cripple walls due to twisting of the building.

Figure 6  Two Possible Causes for Failures of Hillside Constructions
Previous Research

Although extensive damage to residential homes was observed in past earthquakes due to the poor performance of cripple walls, relatively little research was conducted on cripple walls in order to characterize their seismic response. Most of the experimental studies were focused on shearwalls with height-to-length aspect ratio in the range of 1 to 2 [e.g. Ficcadenti et al. 1998, Schmid et al. 1994 etc]. The small aspect ratio of the cripple wall, however, is expected to result in a lateral response that is different from that of full height shearwall since a significant shear deformation may occur due to the low aspect ratio.

Two experimental research programs on level cripple walls were conducted at the University of California, Irvine after the 1989 Loma Prieta Earthquake. The number of specimens in the two test programs was however small, with a total of twelve specimens tested. The height-to-length ratios of the cripple walls were limited to 0.125 and 0.5. For stepped cripple walls, there was no experimental testing to-date, even though damage to hillside homes due to stepped cripple wall failures have been extensive [City of LA 1996; Roselund, 1996]. In the next two sections, test results on level cripple walls from the University of California, Irvine are summarized.

(a) Tests by Shepherd and Delos-Santos [1991]

Shepherd and Delos-Santos [1991] tested seven 2 ft (610 mm) tall by 16 ft (4877 mm) long level cripple walls constructed of construction grade Douglas Fir. Their research was mainly focused on retrofitting of unbraced cripple wall. Bracing was added only to the interior face of the cripple wall, since retrofit is usually assessed from the crawl space below the first floor framing. Three retrofit schemes were investigated:

1. Two tests of cripple walls retrofitted by four 1 in by 6 in (25 mm by 152 mm) timber diagonals, which are sloped at 26 degrees from the horizontal.

2. One cripple wall retrofitted with five Simpson MST68 steel straps. Two 10d nails were used to fasten the steel straps to the wall studs. The steel straps were 12-gage, 2.06 in (52 mm) wide and 48 in (1219 mm) long. In this retrofit scheme, additional holes were drilled in the straps to allow nailing of the straps to the cripple wall studs.

3. Two cripple walls were retrofitted with plywood sheathing for the full length of the cripple wall. The plywood sheathing was attached with 10d nails at 4 in (100 mm) centers, and the plywood used was 0.5 in (12 mm) thick CDX grade.

The cripple walls tested were subjected to a combined vertical compression and reversed cyclic lateral displacement of increasing magnitude. Four full cycles of reversed loadings were imposed, followed by a monotonic increment until failure. The vertical compression consisting of a uniformly distributed load of 300 lbs/ft (4.38 kN/m) was applied to the top of the cripple wall. A double-acting actuator was used to impose the lateral force to the top of the cripple wall. The base of the cripple wall was firmly anchored to a strong floor. Figure 7 shows the lateral force versus lateral displacement envelope curves for the seven cripple walls. The envelope curves correspond to the measured response of the cripple wall during the first cycle to each displacement increment. It can be seen from Figure 7 that the ‘as-built’ cripple wall without...
bracing was very flexible. The lateral displacements for the two ‘as-built’ cripple walls were 0.988 in (25 mm) and 0.954 in (24 mm) at the maximum reported lateral force of 650 lbs (2.89 kN) and 700 lbs (3.11 kN), respectively. These displacements corresponded to an average drift ratio of about 4.0%. It was reported that the failure of the ‘as-built’ cripple walls was mainly due to pullout of toenails from the wall studs and from the top and bottom plates.

The three retrofit schemes tested by Shepherd and Delos-Santos [1991] showed a significant increase in the lateral stiffness and strength over that of the ‘as-built’ cripple walls. It can be seen from Figure 7 that the largest increase in lateral stiffness and strength was for the cripple walls braced with plywood sheathing. At a lateral force of 20,000 lbs (89 kN), the lateral displacements were 0.956 in and 1.06 in (24 mm and 27 mm) for the two braced walls. The corresponding secant lateral stiffness was about 30 times that of the unbraced cripple wall. Figure 7 also shows that timber diagonal brace is effective in enhancing the lateral stiffness and strength of the unbraced cripple wall. The average maximum lateral force for the cripple wall braced with timber diagonals was about 18.5 times the maximum lateral force for the ‘as-built’ cripple wall. The corresponding increase in lateral stiffness was about 7.5 times that of ‘as-built’ cripple wall. Final failure of the cripple wall braced with timber diagonals involved gouging and splitting of the timber braces, and of the top and bottom plates. Figure 7 also shows that cripple wall retrofitted with steel straps resulted in the smallest increase in the maximum lateral force and lateral stiffness. Buckling of the steel straps, which occurred fairly early on during testing, affected the hysteretic response of the cripple walls. Failure of the cripple walls was precipitated by nail pullout from the wall studs and shearing off of nail-heads at the steel straps.

![Figure 7](image_url)

**Figure 7** Comparison of Lateral Strength Envelopes for Retrofitted and As-Built 16 ft by 2 ft (4877 mm by 610 mm) Cripple Walls (Data from Shepherd and Delos-Santos, 1991)
(b) Tests by Steiner [1993]

In a follow-up test program at University of California, Irvine for retrofit of cripple wall, Steiner [1993] tested five level cripple walls retrofitted with 3/8 in (9.5 mm) thick plywood and OSB. Table 1 shows the corresponding test matrix. The cripple walls were taller (6 ft by 3 ft or 1829 mm by 914 mm) than the cripple walls tested earlier by Shepherd and Delos-Santos [1991], and had a height-to-length ratio of 0.5. Test variables include types of structural wood panels, namely (i) plywood and (ii) oriented strand board (OSB), and types of retrofit designs, namely (iii) a single panel with continuous nailing, and (iv) a split-panel with blocking at mid-height. The retrofit design using a split panel allows an easier installation of the panel in limited access and working space. An additional test was conducted with over-driven nails to investigate their effect on the lateral response of the level cripple wall.

Figure 8 shows the details for the test specimens, which were 6 ft long by 3 ft height (1.83 m by 0.91 m). Ventilation holes of 3 in (76 mm) diameter were provided at the top and bottom of the structural panel. It should be noted that the cripple wall frame in these tests were constructed of lumber salvaged from old homes. Pre-drilling of the old lumber was necessary before nailing of the lumber. Note that 6d common nails, as noted in Figure 8, were used for the panel nailing, but are less common in practice. Gravity load on the cripple wall was simulated by a dead load from a concrete block, which was mounted on top of the cripple wall. The magnitude of the dead load was 233 lbs/ft (3.4 kN/m). Increasing magnitude of lateral force was applied to the top of the cripple wall by a servo-controlled hydraulic actuator at a frequency of 0.5 Hz.

Test results indicated that very little difference existed between the lateral response of cripple walls braced by CDX plywood and OSB. Figure 9 shows the lateral strength envelope curves for all five specimens. With the exception of un blocked frame braced with 3/8 in (9.5 mm) thick OSB, the lateral stiffness of the cripple walls was about the same. For the two cripple walls with split panel, the lateral force-displacement envelope curves were very similar. Despite the use of smaller nails for attachment of the panel to the wall frame, the cripple wall failed primarily by splitting of the sill plates. The attachment of OSB with 60% overdriven nails showed a lower lateral force compared to a similar unblocked frame braced with 3/8 in (9.5 mm) OSB. The lateral response of the cripple wall with 60% overdriven nails, however, was similar to that of the unblocked frame with CDX plywood and split panel with blocking braced with either CDX plywood or OSB.
Table 1: Test Matrix by Steiner [1993]

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Single panel without blocking – 3/8 in (9.5 mm) thick CDX plywood</td>
</tr>
<tr>
<td>2</td>
<td>Single panel without blocking – 3/8 in (9.5 mm) thick OSB</td>
</tr>
<tr>
<td>3</td>
<td>Split panel with blocking – 3/8 in (9.5 mm) thick CDX plywood</td>
</tr>
<tr>
<td>4</td>
<td>Split panel with blocking – 3/8 in (9.5 mm) thick OSB</td>
</tr>
<tr>
<td>5</td>
<td>Single panel without blocking – 3/8 in (9.5 mm) thick OSB and 60% of nails overdriven by 3/16 in (4.8 mm)</td>
</tr>
</tbody>
</table>

Notes:
Top Plate Connection: 2-16d end-nails per stud
Sill Plate Connection: 2-8d toe-nails each side per stud
Panel Nailing: 6d-2 inch, 4 inch o.c. edge nailing
12 inch o.c. field nailing

![Diagram of wall configurations](image)

**Figure 8** Braced Cripple walls with and without blockings tested by Steiner [1993]
Summary

In this preliminary report, damage to residential buildings from past California earthquakes is summarized. In-plane failure of cripple walls, with subsequent collapse of the upper stories, porches and utility lines, was the main cause of such damage. Failure mostly occurred in older buildings, which were built before the introduction of modern codes. In those cases, only exterior cladding was used to finish the cripple wall, and was normally constructed of stucco or horizontal wood siding. Consequently, these cripple walls have very little lateral resistance to effectively transfer the inertial force from the upper story framing to the foundation. Insufficient or non-existent bolting of the sill plate to the foundation beam further compounded the problem. Another factor contributing to the failure of cripple wall was due to rusting of the anchor bolt and rotting of the lumber in the cripple wall over the years due to high moisture in the crawl space underneath the first floor. Field evidence, however, indicated that bracing of cripple wall with structural panels is effective in mitigating the seismic hazard associated with cripple wall failure. Damage or collapse of hillside buildings was also extensive in recent earthquake. These failures were mainly due to the connection failure between the uphill foundation wall and the flexible horizontal diaphragm at the first floor, and torsional response of the building resulting high local stresses in the shortest panel of the stepped cripple wall.

Despite the extensive damage to buildings supported on level or stepped cripple walls, very limited research had been conducted to-date to characterize their seismic response. The most notable experimental research on cripple wall was conducted at the University of California, Irvine after the 1989 Loma Prieta Earthquake. The number of specimens tested in the program was small, and was limited to level cripple walls only. No stepped cripple wall tests have been conducted to-date.
References


Task 1.4.4 - Shear Walls
Literature Review

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Summary

The damage to wood framed construction in recent earthquakes, especially the 1994 Northridge earthquake, has shown that the expectation of seismic design codes has not yet been met. The seismic design code in California, the Uniform Building Code, was expected to limit property damage and make the collapse of wood framed structures highly improbable. In particular, the level of damage to wood framed structures that had been constructed a short time before the Northridge earthquake was unexpected. While there have been numerous research efforts in the past to characterize the lateral load and deformation characteristics of timber components and assemblies, funding prospects following the Northridge earthquake have provided new opportunities to investigate this old problem.

The City of Los Angeles-UCI shear wall test program, for instance, has been developing an understanding of the dynamic behavior of panels, especially wood panels, attached to primarily wood framing by connectors such as nails or screws. The goal of this experimental research has been to find if the nailed shear wall systems—from a test matrix with over one hundred 8' x 8' (2.44 m x 2.44 m) panels – can be modeled as principally shear deforming systems and to develop “back bone” force-displacement relationships for commonly used nailed systems.

Despite the fact that a substantial number of shear walls have been tested in the City of Los Angeles-UCI project, many test configurations still remain. For instance, that on-going project has only considered 8’ x 8’ (2.44 m x 2.44 m) walls subjected to the SEAOSC test protocol. Accordingly, walls that are longer or shorter than 8’ x 8’ (2.44 m x 2.44 m), or walls with perforations or walls subjected to a test protocol other than the SEAOSC test protocol will be considered during the shear wall test program of the CUREe-Caltech Woodframe Project.

The overall goals of Task 1.4.4 of the CUREe-Caltech Woodframe Project are to: (1) define and quantify seismic performance levels for common residential wood framed wall surface finishes, (2) establish engineering parameters and damage threshold deformations of wall systems, (3) explore innovations to wall finishes that can improve their seismic performance, and (4) compare shear wall component test results with those obtained from the shear walls used in the UCSD shake table, two story model house.
Published and anecdotal information is available that begin to address these goals. The purpose of this report is to establish a better understanding of the available information through a review of published literature.

**Introduction**

**Background**

The damage to wood framed construction in recent earthquakes, especially the 1994 Northridge earthquake, has shown that the expectation of seismic design codes has not yet been met (Andreason and Rose, 1994). The seismic design code in California, the Uniform Building Code (ICBO, 1997), was expected to limit property damage and make the collapse of wood framed structures highly improbable. In particular, the level of damage to wood framed structures that had been constructed a short time before the Northridge earthquake was unexpected (Comerio, 1995). While there have been numerous research efforts in the past to characterize the lateral load and deformation characteristics of timber components and assemblies, funding prospects following the Northridge earthquake have provided new opportunities to investigate this old problem.

The City of Los Angeles-UCI shear wall test program (Pardoen, et al, 1999a), for instance, has been developing an understanding of the dynamic behavior of panels, especially wood panels, attached to primarily wood framing by connectors such as nails or screws. The goal of this experimental research has been to find if the nailed shear wall systems – from a test matrix with over one hundred 8’ x 8’ (2.44 m x 2.44 m) panels – can be modeled as principally shear deforming systems and to develop “back bone” force displacement relationships for commonly used nailed systems.

All testing has been conducted using the computer controlled, cyclic test protocol recommended by the SEAOSC/CoLA Ad-Hoc Committee on Cyclic Testing Standards (Shepherd, 1996). The protocol uses displacement-controlled, fully reversing cycles and multiple cycles to determine a stabilized stiffness. To date the program has investigated the size and spacing of fasteners, the type of sheathing – in combination and separately – as well as minor variations of the framing. The test program includes code-specified assemblies and connectors, including plywood, gypsum wallboard and stucco sheathing, thus verifying their adequacy or suggesting code revisions.

Despite the fact that a substantial number of shear walls have been tested in the City of Los Angeles-UCI project, many test configurations still remain. For instance, that on-going project has only considered 8’ x 8’ (2.44 m x 2.44 m) walls subjected to the SEAOSC test protocol. Accordingly, walls that are longer or shorter than 8’ x 8’ (2.44 m x 2.44 m), or walls with perforations or walls subjected to a test protocol other than the SEAOSC test protocol will be considered during the shear wall test program of the CUREe-Caltech Woodframe Project.

The overall goals of Task 1.4.4 of the CUREe-Caltech Woodframe Project are to: (1) define and quantify seismic performance levels for common residential wood framed wall surface finishes,
(2) establish engineering parameters and damage threshold deformations of wall systems, (3) explore innovations to wall finishes that can improve their seismic performance, and (4) compare shear wall component test results with those obtained from the shear walls used in the UCSD shake table, two story model house.

Published and anecdotal information is available that begins to address these goals. The purpose of this report is to establish a better understanding of the available information through a review of published literature and a survey of building contractors.

While the information related to the testing and analysis of wood framed structures is not as rich as that of steel, concrete and masonry structures, there has been and continues to be a significant number of research programs that have added to the understanding of timber structures and timber assemblies. In the United States, much of the research effort has been conducted at government labs, universities, and trade associations. While much of the timber testing in the US has been in accordance with ASTM standards, most of the testing has been developed partially within and, at times, completely outside of ASTM standards. Shear wall tests in the past several years by a variety of researchers, for instance, have adopted the ASTM E-564 setup but have replace the three half-cycle monotonic loads of that standard with a variety of cyclic lateral load test protocols.

Similarly, test programs investigating the effects of narrow shear walls (2:1, 3:1 aspect ratios) or shear walls with door/window openings have adhered partially to the ASTM standards. The standards should permit the use of other geometric configurations or load sequences provided that the response parameters measured are consistent with the requirements of the standard. It is important that the experimental data increases the understanding and improves the performance of timber structures under earthquake loading yet provides fundamental results that can be used to develop, refine and calibrate analytical models. Furthermore, the experimental data must be useful to the practitioners as well as to those writing the building codes.

Fortunately, the research opportunity afforded by the CUREe-Caltech Woodframe Project can have a significant impact on the profession by filling in the ‘gap’ in testing, analysis, and code development.
Scope of Review

This literature review is intended to be a document that will continue to be augmented as the CUREE-Caltech Woodframe Project matures. While it is recognized that the literature cited in this report does not complete review task, those references that have been difficult to obtain to date will not only be included in the Task 1.4.4’s final report literature review, but other references will surely emerge and be included in the final report as well. The 80+ references that are cited in this report have been grouped into the following nine arbitrary categories:

a) Test Protocol  
b) Wide/Narrow Shear Walls  
c) Perforated Shear Walls  
d) Shear Wall Analyses  
e) Shear Wall Design  
f) Full-Scale Houses  
g) Nailed Connections  
h) Framing  
i) Diaphragms

Within each category the references are presented in chronological order.
Timber Shear Wall Topics

Test Protocol

While much of the timber testing in the US has been in accordance with ASTM standards, most of the testing has been developed partially within and, at times, completely outside of ASTM standards. This is not to be critical of any researcher desiring to determine strength and stiffness properties of timber components in a heuristic manner, but rather a reflection of the absence of test protocols for other than a few specific conditions. There has been a desire to standardize the significant number of test methods that have evolved for timber testing (Foliente, 1996), particularly for wood framed assemblies that are subjected to seismic loads. The evolution from monotonic loading to reversed cyclic loading to shake table loading requires a sense of uniformity if results across different test beds are to be meaningful to the profession. The CUREe-Caltech Woodframe Project will help accomplish this goal.

Monotonic Loading

The test methods in ASTM E-72 (ASTM, 1995) cover the procedures for determining the structural properties of segments of wall, floor and roof constructions. Of particular interest to the earthquake engineering community is the evaluation of sheathing materials on a standard wood frame subjected to racking loads. The E-72 test method measures the resistance of panels – standard wood frame sheathed with sheet materials such as plywood, oriented strand board (OSB) and gypsum wallboard – to a racking load. The relative performance of sheathing materials is the principal test objective of E-72, since a standard 2.44 x 2.44 m frame is used.

While the racking load behavior gives useful information about the sheathing material, the loading procedure – applied monotonically in half cycles to levels 0-3.5 kN, 0-7.0 kN, 0-10.5 kN and 0-failure – is more suited to wind loads than to seismic loads.

The E-72 setup and procedure was followed by several research investigations. The major thrust of Tissell’s (1993) six shear wall test groups for APA, for instance, was to develop design information for: (a) unblocked shear walls, (b) stapled shear walls, (c) sheathing over metal studs, (d) double-sided walls, (e) panels over gypsum wallboard sheathing and (f) stud spacing and width effects. The results from over one hundred shear wall tests in this project provided a comprehensive table of design values for structural-use, panel shear walls subjected to short-term wind or seismic loading.

In another APA test program, Rose and Keith (1997) considered the effects of gypsum wallboard as well as door/window openings on shear walls. Since racking forces act primarily in one direction, as for wind loads, the shear stiffness and strength of dissimilar materials – such as plywood sheathing and gypsum wallboard – were found to be additive. However, for fully reversing cyclic loads that are characteristic of earthquakes, materials that are brittle or soft – such as gypsum wallboard – were found to only contribute to the shear stiffness and strength at small displacements but not to the maximum shear strength of the assembly. The results of three,
mixed material shear walls (plywood and gypsum wallboard) – from a test matrix of seven shear walls – were obtained using the E-72 test setup and procedure.

The remaining four shear walls of this test program considered only 2.44 x 3.66 m shear walls to determine the influence of door and window openings. The test program confirmed that wood structural panel sheathing under and/or over wall openings contributed to the shear capacity of shear wall panels. While most of the E-72 test protocol was followed, the standard does not cover longer shear walls.

**Reversed Cyclic Loading**

It is recommended to use the ASTM E-564 (ASTM, 1994) protocol if one desires to measure the performance of the complete wall rather than just the sheathing material. The effects of (a) sheathing type (thickness and orientation), (b) stud spacing (material and size) and (c) fastener (type, density, application method) can be determined from the E-564 test procedure. The E-564 procedure is designed to evaluate the static shear capacity of a typical framed wall under simulated load conditions and to provide a determination of the stiffness in shear of the structure and its connections. Whereas the sample is subjected to four half-cycles of pre-determined loads in E-72, the target load within each of the three cycles of E-564 is a function of the panel’s ultimate load.

Dolan and Madsen (1992b) presented the results for monotonic (ramp load) and slow cyclic racking tests of timber shear walls. The tests were part of an extensive experimental and analytical study to investigate the behavior of timber shear walls subjected to earthquakes. The results of full-size shear wall tests showed the important influence of the nail connection between the sheathing and the framing on the load-displacement characteristics of shear walls. The premise that the hysteresis for the shear wall is contained within an envelope defined by the monotonic load-displacement curve for the wall was confirmed. Also, the hysteresis was studied to determine the physical behavior that causes the shape of the hysteresis to be dependent on the maximum displacement of the wall. Both the monotonic and the cyclic racking behaviors of timber shear walls were established, and the performances of plywood and waferboard sheathed shear walls were compared.

Shepherd (1996) reported the progress made, over an eighteen month period, by an ad hoc committee of the Structural Engineer’s Association of Southern California in the development of guidelines to be followed if consistent experimental investigations, representative of the structural performance of mixed material systems under strong seismic excitation are to be achieved. Aspects covered include the issue of load or deflection control, the rate of load application, the pattern of repetitive load or displacement application, the number of components or systems to be tested and the interpretation of the results.

Aravena (1996) attempted to analyze the racking behavior of timber-framed shear walls using full-scale tests under reversed cyclic loading defined by the BRANZ P21 test procedure used in New Zealand. The 2.4 x 2.4 m walls were composed by sheathing of plywood nailed to a frame with 35 x 90 mm studs and sills. The static cyclic tests were used to determine the hysteretic
behavior of the shear walls. Based on the results of the walls subjected to reversed cyclic loading, the damping ratio was calculated using the equivalent viscous damping method.

Deam and King (1996) noted that the best method of determining the seismic capacity of a bracing element is to test it by racking it with cycles of increasing displacement. The maximum mass it can restrain is then assessed analytically but this requires verification with a dynamic test. They noted further that the pseudo-dynamic test method is well established as an alternative to shake table testing in that it provides increased accuracy and enables much larger specimens to be tested.

Currently, the most common method of conducting the dynamic simulation is by applying an incremental displacement to the specimen, pausing while measuring its response, and calculating the subsequent displacement increment. The authors note that two significant modeling errors are introduced because relaxation reduces the load after the displacement increment is applied and the specimen is subjected to high acceleration pulses at the beginning and end of the displacement increment. A continuous test system that keeps the specimen moving while the computation for the subsequent increment takes place was developed at BRANZ to overcome these errors. The BRANZ test facility is described and a proposed research program is briefly outlined. The effect of time scale (or loading rate) on the measured loads is also discussed.

Korin (1996) proposed a new test method for the determination of the shear strength using the five-point loaded beam rather than the direct shear test. A three-point loaded “I” beam was also proposed where the “I” beam is prepared by slotting longitudinal grooves at the two sides of the beam along its centerline. By choosing the correct ratio between the width of the original beam and the “web” of the “I” beam, it is often possible to obtain shear failures, without mid span bending failures, or support zone crushing damage. The paper reports shear tests performed on a few types of timber.

Foliente (1996) noted much work needed to be done to increase the understanding as well as improve the performance of timber structures under earthquake loading. The paper discussed the advantages and limitations of common seismic testing procedures and issues related to the selection of load histories and seismic performance evaluation of structural timber joints and systems. The proposed concepts of yielding in timber systems were briefly presented and the calculation and application of equivalent viscous damping was clarified. It was noted that a lack of understanding of some aspects of cyclic response of timber systems poses some difficulties in developing cyclic test standards that are direly needed to provide a consistent basis for exchange of data and cooperative research in the development of models and design methodologies. It was suggested that at the very least a common data reporting protocol is needed.

Kamiya, Sugimoto, and Mii (1996) conducted pseudo-dynamic tests to evaluate the seismic performance of walls sheathed by 7.5 mm, 9.5 mm and 12 mm thick plywood panels and 12 mm thick gypsum wallboard. Some walls were tested at the same loading speed as an earthquake. For instance, the maximum deflection response of a wall with the greatest mass that the design shear allowed the wall to sustain was found to be 1.1 cm to 2.8 cm depending on the thickness and material of the covering when subjected to the El Centro NS, 1942 earthquake record. The
maximum load response was almost consistently 2.0 to 2.3 times the design shear. When the mass was quadrupled the maximum deflection response of the plywood-sheathed walls increased by a factor of 2.3 while the load response increased by only 3.8%. A fair relationship was obtained between the maximum deflection response and the ratio of the mass to the ultimate strength of the wall. This relationship may be helpful when one reevaluates the design shear of the wall. It was found that the damping ratio affected the response significantly. The time-history responses obtained should be useful for verifying the load-deflection, hysteresis loop models which have been or will be developed in the future since they are free from the error associated with modeling.

Skaggs and Rose (1996,1998) reviewed standardized methods for testing wood framed shear walls, presented a history of North American shear wall design value development, discussed the evolution of the standard test methods to address new concerns, and presented an overview of a research effort to study the difference in the measured behavior of walls tested following the different standardized test procedures.

Their papers noted that two standard monotonic test methods have been used traditionally in the United States for testing wood structural panel shear walls. Engineers, researchers and code officials have questioned the validity of the standard test methods when used to evaluate shear walls which were subjected to cyclic (reversed) loads such as occur in earthquakes. This doubt has spurred the development of test standards that more closely mimic the cyclic nature of earthquake events. Although the proposed cyclic test method was an improvement over the traditional test method, it was noted that the results would be significantly different if another loading protocol was used.

The relationship between the different tests was being studied in an APA research program consisting of forty-five 2.4 by 2.4 m walls. It was noted that buildings are subjected to a variety of loads during their lifetime. For instance, gravitational loads (snow load, dead load) react vertically on the structure, and are typically transferred to the foundation through load bearing walls. Wind and earthquakes cause horizontal forces on the structures, thus leading to lateral loads. Lateral loads are transferred to the foundation through vertical shear resisting elements. For light-frame construction, the gravitational forces are typically resisted by nominal dimension lumber in the form of wall studs and wood structural panel sheathed shear walls commonly resist the lateral loads.

Their papers reviewed the method in which current shear wall values are derived, discussed some of the questions raised about shear wall performance based on monotonic and reversed cyclic load testing, and outlined a research effort conducted by APA. While shear wall tests only evaluate one component of a building system, it is believed that cyclic shear wall tests closely predict actual shear wall performance. The papers note that these tests provide additional information on shear stiffness and capacity of wood sheathed shear walls that will assist designers in making recommendations that are not only safe, but economical to build.

Rose (1998) tested eight shear walls in order to provide preliminary information on shear wall performance under reversed cyclic loads. Seven shear walls used the SEAOSC test protocol
whereas an eighth shear wall used the reversed cyclic displacement time history specified by ATC-24. Although no tests were conducted with monotonic loading, the expected maximum shear strength, obtained after (reversed) cycles of displacement, averaged 18% less than the maximum shear strength reached in the initial displacement cycle.

Dinehart and Shenton (1998) conducted static (monotonic) and dynamic (reversed cyclic) tests on wood framed shear walls to establish a comparison between static and dynamic load effects. Twelve panels (four static, eight dynamic) using the E-564 setup were tested to: (a) determine the shear wall’s resistance to lateral loading, (b) examine the wall performance under fully reversed cycle of dynamic loading and (c) compare the static and dynamic performance as measured using the same test facility. Four specimens were tested statically using the traditional ASTM E-564 test procedure: three-half cycles, loading monotonically to failure. Eight specimens were tested dynamically, using a proposed test standard that was recently developed by the SEAOSC. A comparison of the static and dynamic test results demonstrated several key differences.

Ficcadenti, Steiner, Pardoen and Kazanjy (1998) noted that no national or international standards for conducting cyclic lateral load testing of wood framed assemblies are recognized. ASTM E-564, for example, does not specify a cyclic lateral load testing protocol. The study applied ASTM E-564 to three identical sets of plywood shear wall assemblies using three different cyclic lateral load test sequences in order to investigate the effects of loading sequence on the test results. The first sequence had a large number of cycles patterned after a sequentially phased displacement test procedure defined by Porter (1988). The second sequence had only large excursions used to study the effects of large pulses in the near-field of an earthquake. The third sequence had a moderate number of cycles occurring in progressively increasing excursion increments.

Twenty-four 2.44 x 2.44 m shear walls with 9.5 mm thick plywood panel sheathing were tested. Four different nail styles were used for the identically sheathed samples. Two samples of each configuration were tested under each of the three loading sequences to obtain the load-displacement and load-capacity characteristics of each assembly. The test results provided an understanding for the performance of wood-framed shear walls under cyclic lateral loads and the factors that govern their performance.

Ficcadenti, Castle, Steiner and Kazanjy (1998) addressed the situation in which engineers who design and/or investigate the seismic resistance systems of timber residential structures are often faced with the evaluation of “as-built” shear wall assemblies that vary from those with prescribed values found in the Uniform Building Code. In order for engineers to provide cost effective solutions to these field conditions, the authors suggest that laboratory testing be used to determine the strength of these non-code prescribed assemblies. The key to the successful use of laboratory testing is to select testing protocols that compare the strength and displacement characteristics of the non-code prescribed systems to the same characteristics of a comparable code prescribed system.

Foliente, Karacabeyli, and Yasumura (1998) noted that joints are the most difficult to tackle in the timber standards harmonization process because of the differences in materials and
configurations used in different countries and the lack of internationally acceptable test standards. The authors state that international standards are needed to provide a consistent basis for performance comparison of joint systems and exchange of technical information as well as to facilitate cooperative efforts to develop analytical models and improved design procedures. ISO (International Organization for Standardization) Technical Committee 165 convened a working group in 1995 to develop standards for joints made with mechanical fasteners. The paper discussed developments in establishing international test standards for joints in timber structures under earthquake and wind loads. Some key features of a proposed ISO cyclic (earthquake) load test standard (ISO 1997) and some background on the development of an Australia-New Zealand repeated (wind) load test standard (SAA 1997) were presented. It was noted, however, that these were still working documents and would be subject to revisions. Some items for future work were identified. Product manufacturers, analysts, experimentalists and the general engineering community need to be aware of these developments, and should support and, if possible, participate in these efforts. In the coming era of performance-based engineering, many will increasingly rely on the performance data provided by experimental testing. The authors note that the closer the test specimens, set-up and procedures approach in-service conditions, the more effective the evaluation method becomes as a component of performance-based standards for timber structures.

Zacher (1999) noted that while testing of light wood frame wall and floor/roof assemblies has been ongoing for over fifty years, the test procedures up until ten to fifteen years ago had been monotonic. Cyclic loading protocols have not only been used in many recent tests but have been proposed for adoption. It was noted that the knowledge gaps in shear wall performance require the test conditions to be more representative of actual construction. Some of the missing information could be provided with small scale testing and that the results of the proposed CUREe Woodframe project, when compared with strong motion response, could permit the development of reliable analytical models in the future.

It was noted that in the past the testing of shear walls with openings had been conducted to establish relative strengths. It was suggested that the following construction variables be considered to define the performance of these systems:

- Panel edges occurring at edges of openings when compared with panel edges extending beyond edges of openings.

- Depth and aspect ratios of headers above and spandrels below openings compared with width and aspect ratios of wall segments.

- ‘Hold-down’ devices at edges of openings with different aspect ratios of header, spandrel and wall segments.

- Effects due to multi-story construction.

Foliente, Paevere, Saito, and Kawai (2000) presented a modified version of the BRANZ procedure for the lateral capacity rating of bracing walls that was used to determine the
sustainable lateral mass of a 910-mm wide "2 x 4" timber shear wall. The key modifications involved: (1) the use of a multi-criteria system identification method to determine a structural model that fits test data from both cyclic testing and pseudo-dynamic testing and (2) probabilistic treatment of ground motions (i.e., using suites of site-specific earthquake records with 2%, 10% and 50% exceedance probability in 50 years as input loads in a Monte Carlo simulation). The reliability index for the wall system that was rated according to the modified BRANZ procedure was estimated when subjected to a range of earthquake intensities in Tokyo. For this particular wall, the authors obtained reliability indices (at the safety limit state) ranging from 0.94 to 5.20, depending on the displacement capacity determined from the static cyclic test, and the suite of earthquakes from which the sustainable mass was calculated. Thus, it is desirable to quantify and include the inherent uncertainty in displacement capacity and ground motions in the analysis. The method is general and can be applied to allow the direct use of laboratory data from cyclic or pseudo-dynamic testing for dynamic and seismic reliability analyses of lateral resisting systems with no distinct yield point.

Wide/Narrow Shear Walls

Soltis and Patton-Mallory (1986) presented the strength and ductility data obtained from tests on plywood- and gypsum-sheathed shear walls in timber-framed buildings undergoing seismic lateral forces. Two hundred small-scale shear walls of both one- and two-sided wall sheathing and four aspect ratios (length-to-height of wall) were tested.

Zacher and Gray (1989) dynamically tested thirteen 2.44 x 2.44 m shear panels and fifteen 406 x 457 mm shear panels at the University of California, Berkeley’s Structural Testing Laboratory at the Richmond Field Station. The panels’ behavior – when subjected to cyclically reversing deformations – indicated the importance of fastener characteristics and installation, the effect of sheathing action, and the value of dynamic testing. The tests grew out of concern during investigations of the many distressed wood-frame multifamily buildings in San Francisco, in which about 80 percent of the nails in 9.5 mm plywood sheathed shear walls were overdriven, with the nail heads 3 mm or more below the surface.

Schmid, Nielsen, and Linderman (1994) recommended decreased shear values for plywood shear walls subjected to cyclic loading and additional decreased shear values for walls with a height-to-width ratio of two-to-one. This recommendation was based upon the practice that engineers commonly provide a horizontal member to act as a drag strut or shear collector across the top of the opening, which transfers the lateral diaphragm forces to the narrow shear wall panels at either one or both ends of the opening. In plywood, shear wall systems, the narrow shear panel often consists of a stud wall sheathed with a single 1.22 m wide by 2.44 m height sheet of 13 mm plywood. Because the panel is narrow with respect to its height it typically requires substantial tie-down anchors to resist the overturning moment generated by the lateral forces especially if the adjacent opening is large, allowing substantial forces to accumulate in the drag strut.

Based on these results, decreased allowable shear values for plywood panels with height-to-width ratios greater than one-to-one were recommended to be codified to reduce the likelihood of
permanent set. Furthermore, it was noted that lower shear resistance values make narrow plywood panels of questionable utility.

Enjily and Griffiths (1996) describe a research project concerned with providing a suitable design method for panels larger than those currently within the scope of the UK code. The code of practice for timber frame walls does not cover wall panels in excess of 2.7 m in height and with the development of the commercial and industrial building market there has been an urgent need to extend the design to taller walls. Consequently, the Building Research Establishment (BRE) and the University of Surrey carried out a timber frame research project with the objective of providing the needed design method for panels larger than those currently within the code in the UK.

The paper includes information regarding design and fabrication details of the tested panels. Fourteen different configurations were tested; seven walls with openings and seven without openings. The wall dimensions varied from 2.4 to 4.8 m and the materials used were plywood, oriented strand board, chipboard and tempered hardboard. The load was applied monotonically and both strength and stiffness tests were conducted for each configuration. The strength criterion controlled for walls without openings whereas stiffness controlled the walls with openings. A summary of the test results is given and a discussion with regard to improving the Code’s design method for use with tall wall panels is presented.

Johnson and Dolan (1996) described an experimental investigation of the performance of long wood shear walls with and without openings. The ten walls tested utilized the same materials and fasteners and were subjected to both monotonic and sequential phased displacement until failure. Solid sheathed walls as well as walls with openings were constructed to validate a rational method of designing wood shear walls with openings. The research confirmed the current method in use for developing a shear wall capacity reduction factor for walls with openings was conservative. The project also established a linear relationship between monotonic and cyclic capacity for walls with varying sheathing area ratios.

Lam, Prion, and He (1997) summarized the results from the first phase of a study to evaluate the lateral resistance of wood based shear walls without openings. The walls were built with regular and nonstandard large dimension oriented strand board panels and subjected to monotonic and cyclic loading conditions. Results indicated that under monotonic loading, a substantial increase in both stiffness and lateral load carrying capacity, as well as comparable ductility ratios, could be achieved by shear walls built with oversize panels. The walls built with regular panels, however, were found to dissipate more energy under cyclic loading.

Shepherd and Allred (1998) presented the results of tests involving reversed cycle in-plane loading of narrow plywood shear walls. The 2.44 x 0.70 m specimens were typical of walls found on either side of garage doors in residential framing. Standard hold-down elements were used to anchor these panels that were modeled with the maximum Uniform Building Code allowable height to depth ratio of 3.5:1. It was demonstrated that substantial inherent in-plane flexibility was present in most of the tests conducted resulting from distortion in the hold-down devices. The resulting horizontal deflection of the top of the shear wall would most probably
allow excessive sway in response to even moderate earthquake motions, with consequent unacceptable damage to linings, cladding and other non-structural components.

Selenikovich and Dolan (1999) presented the monotonic and reverse cyclic test results of 55 full-size, light-frame timber shear walls with aspect ratios of 4:1, 2:1, 1:1, and 2:3. The walls were tested in the horizontal position with oriented strand board sheathing on one side. The overturning restraint conditions represented engineered construction (tie-down anchors) and conventional construction (nails or shear bolts). Unlike conventional walls, the engineered walls performed independently from aspect ratio with the exception of the narrow (4:1) walls, which exhibited high ductility and energy dissipation but a very low strength relative to the other specimens. The results permitted an evaluation of the design parameters for shear walls. For instance, the results revealed a lower strength and deformation capacity for walls with sheathing attached at minimum (10-mm) edge distance across the bottom plate compared with walls with 19-mm edge distance. Based on the reversed cyclic load tests, the design values for shear walls should be reduced. It was shown that current nailing schedules for the attachment of conventional walls to lower structures are not adequate to prevent wall overturning.

**Perforated Shear Walls**

Wong and Saudi (1982) explained the inaccuracies involved in the customary method for the in-plane stiffness calculation for structural walls with openings. The source of error was identified; and, through numerical examples, it was shown that, for ordinary wall and opening dimensions, the stiffness could be overestimated considerably. A modified method used by some designers was also explored, and it was demonstrated that even this method could overestimate the stiffness by about 50%. A finite element program was used to obtain the “true” stiffness for a series of single-hole walls with different wall and opening dimensions. Charts that provide the correction factors to be used in conjunction with the conventional method are presented.

Line and Bradford (1996) reviewed the development and technical elements for a new design procedure for perforated shear walls and provided an example of the application of the method. The method has been published in both the *Standard Building Code* and the *Wood Frame Construction Manual for One- and Two-Family Dwellings*. The procedure not only gives designers an additional design method and analytical tool but it also provides the verification of conventional construction techniques, allows for fewer uplift restraints and more efficient use of construction materials.

Dolan and Heine (1998) noted that, historically, wood shear walls have been divided into two classifications: engineered or conventional construction. Engineered walls require a full rational analysis, with overturning moments being resisted by mechanical connections at the ends of each fully sheathed wall segment. Conventional construction provides a prescriptive method to construct walls to provide acceptable performance. To provide information on the effects of overturning restraint and validate the "Perforated Shear Wall Method" adopted by the *Wood Frame Construction Manual for One- and Two-Family Dwellings* from the *Standard Building Code High Wind Edition* (1995) and the *Standard Building Code* (1996), a study of the monotonic and cyclic response of light-frame shear walls was conducted.
The results of two investigations of the cyclic response of light-framed wooden shear walls were presented and discussed in this paper. Comparisons between the cyclic and monotonic response were made along with recommendations for how the results might be incorporated in design. Twenty-two walls, 12 m in length and 2.4 m in height, were tested using monotonic and sequentially phased displacement (SPD) patterns. The results for a total of five different wall configurations, three anchorages, and two loading conditions are included.

The results reveal that the ultimate capacity and stiffness increase with increasing overturning restraint. Ultimate capacities reached in the SPD tests were 14-23% lower for walls with maximum overturning restraint when compared to monotonic test results. However, walls without restraint obtained ultimate capacities from the SPD tests that were 3-8% higher than capacities recorded during monotonic testing. A similar trend was observed for the amount of opening present with fully sheathed walls experiencing an 18% reduction in capacity and walls with the maximum amount of opening experiencing a 1% reduction in capacity when compared to monotonic capacities.

A shift in failure mode was observed when the overturning restraints were omitted. Additionally, the monotonically tested walls sustained higher loads at displacements beyond capacity than walls subjected to SPD loading. While the results indicate that the perforated shear wall method of design is conservative, the method is restricted to effectively resisting the lateral loads at any one level for a building. The authors suggest that the cumulative effects of overturning moment and vertical loads need to be accounted for by using rational design methods.

Qamaruddin, Al-Oraimi, and Hago (1998) noted that three methods have been employed to estimate the lateral stiffness of shear walls with openings. Existing methods assume fixity at the pier-spandrel junction of the wall piers to estimate their stiffness. The authors present a new method to determine the lateral stiffness of the shear walls with openings in which the spandrels are assumed flexible, and can translate and rotate under lateral load. Unlike the lateral stiffness obtained by the older methods, the stiffness obtained by the new method is in good agreement with the results of a linear elastic finite element analysis. It seems that the new method provides a simple, accurate and reliable alternative over the three known methods for estimating the lateral stiffness of shear walls with openings mainly for design office use. Based on the new method, design charts were developed to estimate the lateral stiffness of different piers in a shear wall with openings in terms of their three non-dimensional parameters.

He, Manusson, Lam, and Prion (1999) reported the test results from a study investigating the influence of openings on the lateral resistance of wood-based shear walls built with both standard and oversize oriented strand board panels under monotonic and cyclic load conditions. The test results showed that the application of nonstandard oversize panels significantly improved the performance of the perforated shear walls compared with standard 1.2 x 2.4 m panels. Door and window openings caused a significant decrease in the strength and stiffness of the walls and precipitated a change in failure mode, especially for walls with oversize panels. Although nail failure modes were commonly observed in walls without openings, a combination of nail and panel failures were observed in shear walls with openings. A newly proposed cyclic load test protocol was used that consisted of fewer but more severe displacement excursions, compared
with many other test protocols. This was believed to better reflect typical earthquake excitation and avoid low cycle nail fatigue failures, which were observed previously with long sequence cyclic test protocols.

Ni, Karacabeyli, and Ceccoti (1999) reported that when designing shear walls containing openings, hold-downs are traditionally required at the ends of each wall segment between openings. The shear wall containing openings is then designed as an element with multiple shear wall segments. The design capacity of the shear walls is assembled to be the sum of the capacities of each shear wall segment. Wall components above and below the openings are also treated as walls with the edges adjacent to wall segments representing the top and bottom plates. The vertical displacements around openings, however, were found to be much larger for walls without vertical loads. It was noted that the vertical load significantly reduced stud uplifts around openings.

**Shear Wall Analysis**

Medearis (1969) presented the transcendental equation defining the motion of the basic shear wall-roof mass system model from which a series of curves were derived defining the fundamental period of the system as functions of roof load, shear wall weight, and equivalent spring constant. Utilizing generalized concepts, appropriate numerical methods, and experimentally derived equivalent damping ratios and spring constants, response spectra curves for displacement, velocity, and acceleration of shear wall-roof mass systems were obtained using a digital computer. The use of existing experimental values, which are appropriate for design purposes, in conjunction with period and response spectra curves derived in the paper provide a useful analytical tool for engineers working with plywood shear wall dynamics.

Bower (1974) noted that to properly analyze a plywood diaphragm, the diaphragm must be considered flexible, and structural elements that interfere with its deflection must be considered. The paper noted the published formula for plywood diaphragm deflection at that time was only applicable for the mid-span of a simple rectangular diaphragm with uniform load and uniform nailing. A method was presented for calculating diaphragm deflection of any point with any pattern of loading, including concentrated loads, and with any variation of diaphragm shape or nail spacing.

Naik, Kaliszky and Soltis (1984) developed a three-dimensional model to determine the coupled lateral and torsional natural frequencies and mode shapes for low-rise timber buildings with shear wall and diaphragm construction. Results were presented for a two-story building of a type commonly used in residential construction in the United States.

Itani and Cheung (1984) presented a finite element model for predicting the quasi-static load-deflection behavior of diaphragms. The objective of this investigation was to present a nonlinear formulation for the determination of the load-deflection characteristics of diaphragms (walls, floors, ceilings, and roofs) in low-rise-framed wood buildings due to quasi-static lateral forces. To show the validity of this formulation, results of analyses for two- and three-panel full-scale nailed wood walls were compared with experimental data.
Gupta and Kuo (1985a) showed in previous tests that the behavior of wood-framed shear walls was primarily governed by the nail force-slip characteristics. Additionally, the bending stiffness of studs and the shear stiffness of the sheathing were shown to play an important role in providing the stiffness to the system. Lastly, by constraining the stud deflection shape, they stated that one could reduce the degrees-of-freedom of the model from six to three.

McCutcheon (1985) presented a theory that predicts the racking deformations in wood-stud shear walls. The energy method employed defines the wall performance in terms of the lateral nonlinear load-slip behavior of the nails, which fasten the sheathing to the frame. Using power curves to define the nail load-slip relationship, the theory predicted that wall deformation due to nail slip would also be defined by a power curve. The theory also included the linear deformation due to shear distortion of the sheathing material, and provided accurate estimation of wall performance up to moderate load levels. The method presented should be of interest to engineers who design light frame structures, to researchers, and to those who are concerned with building codes.

Dowrick and Smith (1986) discussed the principles of timber shear walls, gave guidance on modeling, analysis and detail and reviewed relevant research on behavior under cyclic loading. The scope of the paper was limited to walls sheathed with plywood, particle or fiberboard sheets, nailed to the framing. The main issue in analysis and modeling was the understanding of stress distributions and that stiffness rather than ductility was often the prime design consideration.

Gupta and Kuo (1987) presented a simple model for the lateral behavior of wood-framed shear walls with stud uplifting. The single-story wall had five degrees of freedom: the shear rotation of the frame, two relative rotations of the sheathings, the uplifting of the windward stud and the uplifting of the windward panel. For walls of two or more stories, two additional degrees of freedom were introduced for relative rotations of the sheathings in higher stories, thus creating a total of seven unknowns. Results from this model were compared to available test results and the effect of vertical load on the load-deflection curve was also evaluated.

Schmidt and Moody (1989) presented the results of a study done to develop and validate a simple structural analysis model to predict the behavior of light-frame buildings under lateral load. The model was limited to the racking response of shear walls arranged in a rectangular fashion beneath a rigid ceiling/floor diaphragm. Nonlinear load-slip behavior of fasteners was utilized in an energy formulation to yield three-degree-of-freedom representation of each story of the building. Predicted behavior from the analysis model agreed favorably with results form full-scale test.

Dolan and Foschi (1989) presented a numerical modeling procedure for the nonlinear analysis of timber shear walls. The model includes the nonlinear behavior of the connectors between the sheathing and framing, bearing effects between adjacent sheathing panels, and the effect of out-of-plane bending of the sheathing elements. The ability of the numerical model to accurately predict the nonlinear behavior of timber shear walls was examined in detail. Comparisons with an experimental investigation – consisting of seven, static shear wall tests – were made to show that both the stiffness and ultimate load capacity of the shear walls were modeled accurately.
The paper noted that shear walls are commonly used in low-rise timber buildings to provide lateral support against wind and earthquake loads. The most common type of shear wall construction consists of a light timber frame, clad with a panel product such as plywood, waferboard, or oriented strand board. The sheathing is usually attached using nails that are spaced sufficiently close to provide the necessary stiffness and strength in resisting in-plane lateral loads. The racking resistance of shear walls is the major factor in determining the performance of most timber buildings, when subjected to either high winds or an earthquake.

Accordingly the paper noted that a method to predict the racking stiffness and strength of walls constructed of various materials, and in various configurations, is necessary to predict the performance of the complete structure. Three main methods used for predicting the racking performance of shear walls were noted. The first uses empirical relations, derived from test data, and is limited to the materials and configurations used for the test specimens. The second method uses simplified mathematical derivations whereas the third method uses finite elements to model the wall. These programs have allowed for general wall configurations to be modeled such as walls with openings.

The paper presented a numerical solution procedure that extends the analysis of shear walls to include the bearing effects between adjacent sheathing panels, the out-of-plane behavior of sheathing panels, and the prediction of the ultimate load capacity of the shear wall. The formulation was implemented in the computer shear wall program, SHWALL, at the University of British Columbia.

Filiatrault (1990a) noted that shear walls are commonly used to provide lateral stiffness and strength in wood buildings and, therefore, accurate predictions of the seismic behavior of timber shear walls are necessary in order to evaluate the safety of existing timber buildings and improve design practice. Accordingly, the paper develops and validates a simple structural analysis model to predict the behavior of timber shear walls under lateral static loads and earthquake excitations. The ability of the model to accurately predict the lateral stiffness, the ultimate lateral load capacity, and the complete earthquake response of timber shear walls was clearly demonstrated. It was these factors that contributed to the reduction of the natural frequencies of wood structures so that these structures fell within the expected frequency range of strong ground motions.

Filiatrault (1990b) proposed a new concept for the earthquake-resistant design of timber shear wall structures. By providing friction devices in the corners of the framing system of the shear wall, its earthquake resistance and damage control potential could be enhanced considerably. The author notes that friction devices slip during severe earthquake excitations and a large portion of the seismic energy input is dissipated by friction rather than by inelastic deformation of the sheathing-to-frame connectors. A simple numerical model was developed and the results of inelastic time history dynamic analyses showed the improved performance of the friction-damped timber shear walls compared to conventional shear wall systems.

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of-plane bending of the sheathing elements. The ability of the numerical model to accurately predict the nonlinear behavior of timber shear walls was examined in detail. Comparisons with an experimental investigation – consisting of seven, static shear wall tests – showed that both the stiffness and ultimate load capacity of the shear walls were modeled accurately. Corrections, based on recent test results, were made to the test data used to verify the numerical accuracy of the model.

Filiatrault (1991) developed and validated a simple structural analysis model that predicts the behavior of timber shear walls under lateral static loads and earthquake excitations. The model is restricted to two-dimensional shear walls with arbitrary geometry. The nonlinear load-slip characteristics of the fasteners were used in a displacement-based energy formulation to yield static and dynamic equilibrium equations. The predictions of the model were compared with full-scale racking and shake table tests. The model accurately predicted the lateral stiffness, the ultimate lateral load capacity, and the complete earthquake response of timber shear walls.

Foliente, McLain and Singh (1993) presented a new hysteresis model for wood joints and structural systems that was based on a modified endoebronic theory. The model incorporated experimentally observed characteristics such as non-linearity, strength and stiffness degradation and pinching and was in a form suitable for both deterministic and random vibration analyses. Its capability was demonstrated under cyclic loading.

White and Dolan (1995) presented a finite-element program, WALSEIZ, capable of performing nonlinear analysis of a timber shear wall to monotonic or dynamic loads. The analytical model assumed that each wall could be composed of four elements: a beam element to model the framing, a plate element to model the sheathing, nonlinear springs to model the sheathing-to-framing connectors (the load-displacement properties of which vary depending on whether monotonic or cyclic loads are applied), and bilinear spring to model bearing between adjacent sheathing panels.

WALSEIZ was developed to gain more insight into the performance characteristics of timber shear walls. The program is capable of performing both static and dynamic analysis of a shear-wall model and of calculating the resulting displacements, forces, and stresses. The benefits of the program are that it provides a more economical means of performing dynamic analysis on shear walls and a more efficient means of analyzing different wall sizes and configurations than experimental testing. Furthermore, the program enables one to determine the detailed information on load paths, internal forces, and anchorage requirements.

It was reported that the framing component of the shear wall model resists most of the out-of-plane bending and some vertical load, while the sheathing resists most of the in-plane shear. The in-plane strength, which is a measure of the magnitude of load that a wall can resist before a limit state occurs, and stiffness, which is the relative deformability of a material under load, were found to be the most important characteristics in the determination of the adequacy of a wall subjected to monotonic loading.
Foliente (1995) characterized the general features of the hysteretic behavior of wood joints and structural systems and reviewed the available hysteresis models for wood systems. A general hysteresis model for single- and multiple-degree-of-freedom wood joints and structural systems, based on a modification of the Bouc-Wen-Baber-Noori model, was presented and used in the nonlinear dynamic analysis of single-degree-of-freedom wood systems. The hysteretic constitutive law produced a smoothly varying hysteresis that modeled previously observed behavior of wood joints and structural systems, such as load-deformation nonlinearity, strength and stiffness degradation, and pinching. The model takes into account the experimentally observed dependence of wood joints’ response to the input and response at an earlier time. Hysteresis shapes produced by the model were shown to compare favorably with experimental hysteresis of wood joints with (1) yielding plates; (2) yielding nails; and (3) yielding bolts.

White and Dolan (1996) presented the results of a parametric study performed to determine the effect of aspect ratio and openings on the response of timber shear walls subjected to seismic loading. Twenty-five wall models were constructed and analyzed using a finite element program developed specifically to analyze shear walls. Results of the analysis were presented along with modification recommendations for the current design methodology.

**Shear Wall Design**

Tissell (1979) provided some Seismic Zone 4 design considerations based upon a test program that conducted 11 full-scale (4.88 x 14.63 m) diaphragm tests. The test program included such design considerations as the high density of fasteners, openings, glued diaphragm construction, and two-layer systems in the high shear areas. Only four diaphragms of conventional construction and high-density mechanical fasteners were discussed in this report.

Buchanan (1983) briefly described the major factors affecting the behavior of wood structures in earthquakes. The paper noted that recent developments in timber engineering had increased the interest (a) in wood as an engineering material and, (b) in the earthquake resistance of wood structures, particularly in New Zealand. The paper suggested how recent advances in timber engineering and earthquake engineering could be combined to produce improvements in the design of wood structures for seismic regions.

Tissell (1989) showed the design method currently used for unblocked diaphragms could be used for shear walls. Additionally the paper confirmed that walls identically sheathed on both sides have double the strength of a single-sided wall.

Han, O’Halloran, Zylkowski and Yeh (1996) presented the results of the continued research by APA in developing engineering design information on structural-use panels. Under the APA Quality Assurance Program, new data had been collected since the publication of APA’s recommended design values in 1988, TN375. These data offer an opportunity for reevaluation of the published design capacities and reflect up-to-date capacity level of APA trademarked panels.

Yasumura, Ohta, and Fukuda (1996) noted that since the introduction of a revised design manual in Japan in 1992, designers have been given more freedom in design of lateral load resisting
elements due to the introduction of equations used to calculate the yield strength of nail joints as well as calculations for mixed structural systems. The paper presented and validated the design method for wood-framed buildings containing both shear walls and glu-lam frames with moment resisting joints. A model was designed and built, containing both shear walls and a frame with moment resisting joints, and then exposed to a shake table test. By presenting both the design procedure and test results, the behavior of this model and the design method were deemed adequate.

Stricklin, Schiff and Rosowsky (1996) noted that light-frame wood construction continues to be the most economical and widely used method for building one and two-family residential structures. As a result of hurricanes Hugo and Andrew, there was an increased interest in the ability of light-frame structures to resist the forces of natural disasters, in general, and high wind events, in particular. The authors noted that there was an effort to pre-engineer and establish acceptable methods of construction for these traditionally non-engineered structures so that they could better resist high winds. Establishing the vertical load continuity in a structure was considered important when transferring the wind loads to the foundation. It was noted that loads are typically transferred from the rafter or roof truss to the top plate of the wall using a metal strap and then transferred down the wall through the exterior sheathing or with tie-rods. Their paper reported on an investigation of the uplift capacity of 8 ft by 8 ft wall sections by considering the effects of sheathing orientation as well as various nailing schedules.

Griffiths and Wickens (1996a) attempted to justify the empirically-based, current method for evaluating wall-racking resistance in use in the UK. The paper considers the background to design methods for this evaluation and shows its close tie to the top loading test method. A comparison was made with other methods notably the ASTM-based TRADA and Eurocode 5. Experimental data of the racking resistance of long walls was compared to the design model and appraised. The inclusion of the deflection limit makes the UK method appear to be conservative relative to other methods but the realistic model for timber frame wall behavior justified this difference. Although the finite element method can and has been used as a valid design tool, the cost involved in the inclusion of this type of analysis is usually not justified by the overall size and cost of timber frame projects. In general, and unlike other methods, the UK method concentrates more on the racking of the wall itself and less on the base connection.

Griffiths and Wickens (1996b) discussed a method of deriving design values for estimating the safe racking resistance of timber frame walls from experimental tests. The paper compared UK test methods with earlier US work and more recent European proposals. Conclusively stated, the US method gives the designer no freedom in wall construction and is based on static loading, which underestimates most lateral loads. The European method unrealistically assumes stiffness is not the governing factor in design but compliments the UK method more adequately than the US method. Finally, the UK method utilizes a statistical approach and method of reduction of results to design less conservative and more appropriate walls than either of the other two methods.

Kamiya and Itani (1998) developed a simplified procedure for determining the stresses in diaphragms with openings. The specific objectives of the paper included the experimental
verification of the developed method and the comparison of the calculated stresses with those determined by the existing methods. Horizontal loading tests were conducted on floor diaphragms with a variety of openings or solid sheathing and little to no difference was observed in the ultimate load of these diaphragms. Using this newly developed method, the observed unit shear around the openings was very similar to the calculated value and the measured deflection was slightly larger than the expected value. From the test results all three of the existing methods as well as the new method performed well as practical design tools. Their new method can be used for calculating shear forces in the sheathing and forces in the chords for various configurations of the diaphragm, including those that have the shape of notched beams.

**Full-Scale Houses**

Soltis (1984) examined past cases and common causes of failure in wood-frame buildings subjected to extreme wind, seismic, or snow loads. The performance of marginally engineered or non-engineered structures was compared to that of engineered structures. The importance of connection or anchorage failures between major components was discussed. Based on the study’s results, a technology transfer effort to incorporate engineered features in more wood-frame buildings was recommended.

Gupta and Kuo (1985) described the modeling of a wood-framed house.

a) The shear wall was tested with plywood sheathing only, without any door and window openings.

b) At stage 2 of the test, the window and door openings were provided.

At stage 2, openings were provided in the shear walls for a door and a window. The openings resulted in five discontinuous studs. The stiffness offered by the sheathing-nail connections to the discontinuous studs were found to be much less than the stiffness of the sheathing-nail connection to the continuous studs. For walls of about equal height and length, the uplifting in studs can dramatically decrease the stiffness of the wall.

Deam, Dean, and Buchanan (1991) described the cyclic testing of three-story, plywood-sheathed shear walls together with the test results. The behavior of single-story, plywood-sheathed shear walls was studied and a procedure for the seismic design was developed. The research project described and applied this procedure to a series of three-story shear wall specimens 3.6 m wide and 9.0 m high. The specimens were constructed and then tested to destruction.

Phillips, Itani, and McLean (1993) noted that contemporary analysis and design procedures for light-frame wood buildings did not give consideration to the complex three-dimensional structural response of the buildings. A full-scale single-story wood house was constructed and tested under lateral loads at various stages of loading to evaluate the structural response and load-sharing characteristics. Different sheathings, fastener arrangements, and openings were incorporated to create shear walls with varying stiffness. Extensive force and displacement readings were made of the building during testing to quantify the structural response.
Suzuki, Fujino, and Kawai (1996) determined the dynamic properties such as natural frequencies, damping factors and mode shapes of three full-scale two-story houses using two types of experimental modal analysis. The horizontal acceleration responses in two directions for each house were measured utilizing two excitation devices, an impact-hammer and an electro-dynamic exciter. The experimental and analytical results indicated that experimental modal analysis with the excitation by the impact-hammer revealed comparable dynamic properties to those established utilizing the electro-dynamic shaker.

Nailed Connections

Tuomi and McCutcheon (1977) presented the derivation of a method for calculating the racking strength of light-frame panels and a comparison of the theoretical values with several actual laboratory tests. The derived equations require the input of the panel geometry, the number and spacing of nails and the lateral resistance of a single nail. Whereas the theoretical results gave close agreement with experimental data, the method was not applicable to stiffness computations but only to strength predictions.

Filiatrault and Foschi (1991) presented an experimental investigation into the seismic behavior of timber shear walls fastened with nails alone or with nails in combination with wood adhesive. The experimental results showed that under dynamic, earthquake-induced conditions, introduction of the adhesive makes shear walls much stronger but also more brittle than conventional nailed walls. The increased strength was controlled almost entirely by the connections between the framing members and the anchoring of the wall’s base plate when the adhesive was utilized. Accordingly, the study concluded that these details controlled the capacity of the frame to sustain the higher load introduced by the stiffer adhesive joints.

Dolan and Madsen (1992a) presented the results of monotonic and cyclic nail connection lateral tests. The tests were part of an extensive experimental and analytical study to investigate the behavior of timber shear walls subjected to earthquakes. The results and the nail connection tests were used in a larger study of timber shear walls. The nonlinear load-deflection curves were used to model the nail connection between the sheathing and the framing of the shear walls. The dependency of the nail connection on the grain orientation of the timber materials was investigated, along with evidence that the material properties of the nails were the primary parameters for the load-displacement characteristics of the connection made with hot-dipped, galvanized common nails. Both monotonic and cyclic lateral, behavior of the connections were established. The premise that the hysteresis for the nail connection is contained within an envelope defined by the monotonic load-displacement curve was confirmed. These connection characteristics translate into similar behavior for nailed timber shear walls.

Ficcadenti, Castle, Sandercock, and Kazanjy (1995) conducted laboratory tests to investigate the performance of pneumatically driven box nails used in 9.5 mm inch thick plywood sheathed shear wall panels. The panels were built with 8d pneumatically driven box nails and were compared to panels built with 8d common wire nails. The panels built with different percentages of over-driven box nails (where the nail head pierced the face of the plywood sheathing by at least 3 mm) were also tested to determine the effect of overdriving on panel strength.
Karacabeyli and Ceccotti (1996) presented an outline of a five-year research program in which the lateral load resistance of timber structures was to be investigated on 2.44 x 4.88 m walls under ramp and cyclic displacement schedules. Code implications as well as effects of fastener type and contributions of gypsum wallboard were discussed. The combination of gypsum and either OSB or plywood was found to have increased ultimate strength but lower ductility and nails demonstrated greater ductility and strength than drywall screws.

Richard (1998) presented a cooperative research effort, including the Laboratoire de Mecanique et Technologie of Cachan, France, that was being developed in Japan for the analysis of structural performances of timber shear walls under seismic loading. The project included a series of full-scale pseudo-dynamic and shake table tests of walls made of posts, beams, and sheets. Rigid walls prevented the structure from vibrating in directions perpendicular to the motion. The cooperative research program was envisaged to conclude with a series of dynamic, pseudodynamic, and static tests of two-story buildings with asymmetrical shear wall composition that are supposed to produce more complicated vibration modes such as torsion and three-dimensional movements.

The Laboratoire de Mecanique was involved in the finite element modeling of the timber shear walls. In the proposed finite element modeling, all the phenomena of dissipation were assumed to occur in the joints. A hysteresis modeling was presented of the nailed connection that was identified during tests on single nailed plywood-to-lumber joints under static monotonic and reversed cyclic loadings. The model was implemented in the finite element code CASTEM 2000 for the simulation of the nonlinear response of shear walls. Nonlinear joint elements linked degrees-of-freedom of lumber and sheathing panels. Pseudo-dynamic tests were carried out on three shear walls made of plywood sheathing panels connected by nails to lumber framing members. The prescribed experimental displacement was obtained using the 1995 Hyogokken-Nanbu earthquake acceleration. Simulation results of these tests were shown. Static calculations were performed to test the validity of the modeling. The experimental responses were compared with the responses calculated using the finite element modeling.

**Framing**

Ying, Lian, Reihua, Zhengchang, Jingkai (1984) described a structure with precast shear wall panels installed using vertical prestressing and horizontal friction slide joints formed between the tops of the panels and the bottoms of the beams. Test results were given for a single-story reinforced concrete specimen and a three-story timber model. A design for a five-story experimental building with the new shear wall was also presented.

Mallory, Wolfe, Soltis, and Gutkowski (1985) provided racking strength and stiffness information on full-size and small wall tests. Wall length and effective wall length (for wall with openings) were related to racking strength and stiffness. The composite wall behavior was evaluated in terms of individual component test data. Three conclusions resulted from the analysis of the wall racking test data:
a) Ultimate racking strength of test walls with gypsum sheathing is proportional to wall length for aspect ratios (length: height) between one and three. However, racking stiffness is not linearly proportional to wall length. Racking strength of plywood test panels is proportional to wall length for aspect ratios between one and four.

b) The proportionate effective wall length is a measure of the racking strength of test walls with door and window openings. Proportionate effective length tends to overestimate stiffness of walls with door and window openings.

c) The racking stiffness and strength of a composite wall is equal to the sum of the resistance measured in tests of its components at a given displacement and at ultimate load when the components tested are single sheathed walls.

Polensek and Schimel (1991) tested component subsystems in light-frame wood buildings to provide information useful in analyzing and designing for seismic and dynamic wind effects. Static cyclic tests on shear wall panels were performed to determine the effect of panel displacement on damping ratio. Static cyclic tests on bending panels were also performed. Connection subsystems between floors, foundation, and walls were tested on a shake table under steady-state vibration (perpendicular or parallel to the foundation length) at prescribed, constant panel-displacement amplitudes.

Soltis and Falk (1992) updated a previous literature review paper on the performance of wood-frame buildings during earthquakes and summarized recent research related to understanding seismic behavior of low-rise wood buildings.

Kasal, Leichti and Itani (1994) provided an analytical method to investigate the behavior of light-frame wood structures loaded by static loads. Special attention was given to load sharing among wall components. A one-story wood-frame building (4.9 x 9.8 m) was tested under cyclic quasi-static loads. Results of the experiment were used to verify a nonlinear finite-element model of the full building. The full-structure model was an assemblage of super-elements, representing the floor and roof, and quasi-super-elements, representing the walls and inter-component connections. The special quasi-super-elements were energetically equivalent to the detailed three-dimensional finite-element models developed to represent the walls and inter-component connections. Boundary conditions and loads used in the experiment were applied to the model, and deformations and reaction forces were compared.

Foliente (1995b) noted that on-going work in the U.S. and other countries on new procedures related to the seismic design of light-frame wood and wood-based buildings requires a comprehensive appraisal of our understanding of their seismic performance. Brief overviews of current knowledge obtained from damage assessment surveys, testing and analytical modeling of the earthquake performance of light-frame wood buildings, and a summary of research that is needed to help better understand and improve their seismic behavior was presented. A coordinated effort of experimental and analytical investigations to develop practical design and construction methodologies was proposed.
Anderson and Kelley (1996) tested 24 light-gage metal on dimension lumber framed diaphragms to failure. Strength and stiffness data for the eight diaphragm constructions representing changes in purlin spacing and four seam fastener patterns were reported. The strength of diaphragm components were found to be additive whereas stiffness was not. The sheet buckling strength for purlins 0.76 m on-center was determined but longer screws were necessary to establish the sheet buckling strength for the tested sheet-to-purlin fastening pattern for purlins 0.61 m on-center. Seam slip, purlin twist and purlins-to-rafter connection slip were found to be significant slip components of the total diaphragm deflection. The purlin twist, the purlin-to-rafter connection slip and seam slip were found to be major components of diaphragm slip. The buckling strength of the sheet was not determined for the 0.61 m purlin spacing because the screws pulled out of the purlin allowing the sheet to buckle. It was suggested that longer screws would most likely increase the load required to buckle the sheet.

Martensson, Thelandersson, and Enockson (1996) noted that deflections and movements in a building must be limited to avoid damage and other undesirable effects in service and especially in lightweight structures, such as timber, that are prone to exhibit deformations. For timber-framed buildings one of the important factors noted was the combined effect of loading and moisture changes. A discussion about the relevance of the criteria used normally in different countries in order to restrict the deformations was also presented. Such criteria are in most cost codes given as restrictions of the bearing stress. A more rational approach suggested that one consider the bearing behavior as a serviceability problem, since this often is the main problem with floor joints. This paper addressed the movements in wood at critical connections where wood is under high bearing pressure perpendicular to grain with specific focus on the interaction between moisture changes and load. Short- as well as long-term tests were performed to assess this behavior. The long-term tests show that the effects of creep and moisture changes were comparatively larger than the initial load effects.

Serrette and Ogunfunmi (1996) presented detailed results on the shear behavior of 2.44 x 2.44 m light-gauge steel stud walls for three different shear resisting systems: framed walls with 20 gauge flat strap X-bracing on the face—type A; framed walls with 12.5 mm single-ply gypsum wallboard on the back and 12.5 mm single-ply gypsum sheathing board on the face—type B; and framed walls with 12.5 mm single-ply gypsum wallboard on the back, 12.5 mm gypsum sheathing board on the face, and 20 gauge flat strap X-bracing on the face—type C. The steel framing used in these tests is typical of framing used in residential construction. The behavior of the type A walls was governed by the yield strength of the straps with practically no resistance provided by flexure in the studs. In the type B and type C tests, the measured maximum load was controlled by the breaking of the wallboard along it edges. The failure mechanism was initiated by a rotation of the screws at the edges. This was followed by the partial pull-through of the screws at the edges of the gypsum board and simultaneous breaking of the board at the edges. Each 1.22 x 2.44 m gypsum panel was observed to behave independently during loading. A comparison of the results from the type B and type C tests showed that the use of 50.8 mm, 20 gauge tension bracing prevented cracking of the boards at the perimeter, reduced the lateral displacement, and increased the maximum load capacity of the wall by as much as 28%.
Heine, Dolan and White (1999) noted that the engineered design of exterior shear walls containing window and door openings involves the use of multiple, fully-sheathed shear wall segments typically restrained against overturning forces at both ends. Results of two investigations of monotonic and cyclic response of light-frame shear walls were presented. Twenty-two full-scale wall specimens were constructed employing methods typical for platform construction and tested using monotonic and sequential phased displacement (SPD) patterns. A total of five different wall configurations with various door and window openings, three anchorage, and two loading conditions were used. All walls were 2.4 m high and 12.2m long.

**Diaphragms**

In a particularly ambitious APA research program, Tissell and Elliot (1981) tested 11 diaphragms of 4.88 x 14.64 m to develop design and construction recommendations for diaphragm shears using two layers of plywood, thicker plywood or a greater number of fasteners than commonly used in practice. The evaluation of field-glued plywood and diaphragms with openings as well as the use of staples instead of nails were secondary objectives of the test program. The test configuration of the diaphragms was similar to that of a simply supported beam subjected to a uniform load. A uniformly distributed load was applied normal to the diaphragm’s long edge, whereas the diaphragm was supported at the extreme ends of the long edge. The distributed load was applied in three distinct phases through hydraulic cylinders spaced 0.6 m on center.

Two diaphragms were constructed with openings to test their effect of diaphragm strength. One diaphragm had a 1.22 x 1.22 m opening centered 2.44 m from each end. While the forces generated by the opening could be calculated by applying the principles of statics, it was noted, however, that chord forces do not increase significantly when the openings are relatively small and it is usually sufficient simply to reinforce the perimeter framing and assure that it is continuous. It was noted further that continuous framing should extend from each corner of the opening both directions into the diaphragm, at a distance equal to the largest dimension of the opening. Another diaphragm with a 2.44 x 2.44 m opening was tested to determine the effect of larger openings on diaphragm performance. The shear transfer resulted in increased tension and compression stresses in the framing on each side of the 1.22 m widths of the diaphragm remaining along the openings.

Smith, Dowrick, and Dean (1986) discussed the principles of horizontal timber diaphragm behavior under in-plane loading. Guidelines were proposed for the analysis, design, and details and relevant research was reviewed. Plywood, particle board, and solid timber boarding were all considered as sheathing materials for wind and earthquake resistance.

Falk and Itani (1987) presented the results from an experimental study performed to measure the static and dynamic properties of several plywood and gypsum sheathed diaphragms. Four walls, three floors and three ceilings were tested to determine the natural frequencies, damping ratios and nonlinear stiffness characteristics. Experimental results showed that the natural frequency decreased with increased displacements and the damping ratio increased with increasing displacement. Also, the decrease of stiffness in diaphragms with openings was proportional to the area occupied by the opening.
Pardoen, Hamilton, Del Carlo and Kazanjy (1999b) tested large timber diaphragms to provide experimental data on representative floor and roof sections within structures that were being seismically retrofit. These experimental results provided the structural designers with representative, in-situ data for their nonlinear analysis models of these structures. The fabrication of 4.88 m x 4.88 m roof and floor diaphragms for a California historic structure was particularly challenging in that many of the 19th century redwood timber sizes for the framing and sheathing were not commercially available. Two floor and four roof diaphragms were tested using a reversed cyclic loading (ATC-24) test protocol. Different nailing patterns were investigated for the floor diaphragm whereas the roof diaphragm samples considered the effects of different nailing patterns as well as the stiffness influence of timber shingles. The results indicated that increased nail density in the floor diaphragms increased the stiffness and yield force. Furthermore, it was noted that timber shingles were the predominant means of load transfer and energy dissipation in skip-sheathed roofs. In a 20th century application, Pacific Gas & Electric – a very large, California-based utility – sponsored research to reduce the earthquake vulnerability as well as to improve the system reliability and safety of its electrical and gas transmission and distribution systems. The seismic retrofit of over 100 tilt-up wall buildings was a key component of the PG&E project. Six roof diaphragm samples considering different nailing schedules within a given, but representative, PG&E building were subjected to racking in the long (4.88 x 6.10 m) and short (6.10 x 4.88 m) directions. The results indicate that there was higher strength and more ductility in these roof diaphragms that had been anticipated.

Pardoen, Del Carlo, and Kazanjy (1999c) conducted an experimental program to provide representative stiffness properties for timber roof diaphragms as well as strength properties for the timber roof-to-wall connections. The stiffness and strength properties provided PEER researchers with experimental values for representative, in-situ roof diaphragm and wall-to-roof connection conditions. The roof diaphragm’s stiffness properties provided researchers with the load-deformation characteristics needed as input for nonlinear finite element analyses. Additionally, the roof-to-wall’s strength properties provided researchers with the experimental data that could be compared with the predicted connection loads from the output of the finite element analyses.

The stiffness tests considered the load-deflection characteristics of six representative roof diaphragm panels – five 4.88 x 6.10 m panels (W x H) and one 6.10 x 4.88 m – under different load, nail and stud spacing conditions. The strength tests of the timber roof-wall connections determined the pullout loads for 12 different roof-to-wall connections. The roof-to-wall connections – representative of old and new construction methods – were:

- glu-lam beam-to-column connection,
- diaphragm-to-75 x 300 mm ledger (purlin-to-wall) connection,
- diaphragm-to-75 x 150 mm ledger (subpurlin-to-wall) connection.

The report summarized the (a) stiffness tests of the roof diaphragms, (b) strength tests of the glu-lam beam-to-column connections, (c) strength tests of the purlin-to-wall connections and (d) strength tests of the subpurlin-to-wall connections.
Conclusion

There probably is never a ‘conclusion’ to a literature review and the enclosed survey is no exception. There have been references that were difficult to obtain in timely manner to review and other references that, unfortunately, were inadvertently overlooked.

This literature review is intended to be a document that will continue to be augmented as the CUREe-Caltech Woodframe Project matures. While it is recognized that the literature cited in this report does not complete review task, those references that have been difficult to obtain to date will not only be included in the Task 1.4.4’s final report literature review, but other references will surely emerge and be included in the final report as well. The members of Task 1.4.4 would appreciate the assistance from colleagues by identifying relevant references that would make this literature review more complete.
References


Task 1.4.6 - Seismic Performance of Gypsum and Stucco Walls
Literature Review

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Summary

The two most common methods of finishing woodframe structures are gypsum drywall and portland cement plaster. This report summarizes the common construction practices, the experimental data currently reviewed, and a representation of the current engineering design information. In addition a survey of practitioners was conducted in the Northern California region to establish common construction practice.

Gypsum drywall research is available in the engineering literature. The force-displacement graphs of nine experimental specimens have been reviewed at this time. The test programs had varying objectives and so limited correlation is available between the test specimens. Wall shear strengths varied from 170 to 640 lb./foot. Secant stiffness, measured at the point of peak load, varied from 370 to 2300 lb./inch/foot. Other research sources are being investigated and when appropriate will be included into the final report.

To gain clearer understanding of current construction methods, a survey was distributed to practitioners in the industry. The survey was completed and returned by 19 of the 67 industry personnel contacted. The survey results indicated a variety of wallboard thickness and fastener details in common use.

This report represents the literature research reviewed in the first two months of the current project. The literature review will continue throughout the entire project. The information
provided here is tentative and represents the current understanding by the authors. The authors reserve the right to revise and alter the report during the course of the project.

**Literature Review**

Task 1.4.6 of the CUREE-Caltech Woodframe Project has several aggressive goals. The overall goal is to define the performance levels of common residential woodframe wall surface finishes. In addition the engineering parameters of wall systems are to be quantified. A third primary goal is to explore innovative means of attaching wall materials to framing in the hope of decreasing the potential damage in future earthquakes.

Published and anecdotal information is available to base initial answers to these goals. The purpose of this report is to establish the level of understanding of these issues in the engineering literature today. Two research methods have been used to determine the current state of the art: a survey of retrieved published literature, and a survey of current practitioners in construction.

Literature on testing of these wall finish materials is being collected and reviewed. Much of the reviewed literature is based upon wall systems used as portions of the building’s lateral restraint system. Currently, test results from San Jose State, the University of British Columbia, the American Plywood Association, Cal Poly and Lund Institute of Technology have been reviewed.

To obtain insight into the state of the art of current building practices a three-page questionnaire was mailed to 67 drywall contractors in the San Jose region. The addresses for these contractors were obtained from the San Jose Yellow Pages. To date a total of 19 questionnaires have been returned and processed.

Woodframe structures are built with a wide variety of architectural finishes on the walls. Modern structures usually rely upon plywood or wood product shearwalls for lateral strength during earthquakes. Plywood is seldom the final surface of the wall, instead some form of cementious material, such as gypsum or portland cement (stucco) is installed as the final wall surface before painting.

In the past, many buildings were built with the expectation that these cementious wall finishes would provide a significant amount of lateral strength and stability. Over the past thirty years the assumption that the gypsum drywall can assist in resisting lateral force has been significantly reduced. The reduction in strength has generally developed in result to the significant damage occurring in lightframe structures in past earthquakes. The consensus has been that these buildings need support from shearwalls built from more robust materials, usually plywood or some other wood product.

A brief history has been compiled of the engineering of residential construction in California (Federal Emergency Management Agency, 1997, pp.8-1). In 1934 structures began to be designed for seismic lateral loads but single-family residential structures generally were not engineered. Platform construction using cripple walls was generally discontinued around 1950 and later seismic building code requirements for timber buildings began to be widely enforced.
After 1970 structures generally were designed with well-defined lateral-force resisting system, although construction quality and code enforcement varies widely.

**Gypsum Wall Construction**

The most common wall finish for modern construction is gypsum wallboard as a part of a gypsum drywall system. Gypsum drywall is a common material used for the covering of timber and steel stud framing. After attachment, sealing of the joints, possible texturing, and painting, a finished wall surface is available. The base of this wall finish system is wallboard, gypsum slurry solidified into large panels (4 foot by 8 foot is standard).

**Common Practice of Gypsum Drywall Installation**

Wallboard was originally developed in the late 1880’s by Augustine Sackett (Bureau of Industrial Education, 1972, pp.13). At the time spackling material was used to fill the joints, which were then taped with muslin cloth. The wallboard was square-edged and time consuming to cover the joints with these materials. Gypsum wallboard and joint compounds were greatly improved starting in the late 1920’s. Highly calendered ivory paper replaced the previous covering paper. Paper fibers were added to the basic core material. Wallboard with tapered edges was made. Perforated paper tape and specialized joint compounds were made that allowed for faster and easier closing of the joints. Later mechanical taping and finishing tools were developed. Joint compounds have continuously been improved for workability and durability.

In the 1950’s construction using wallboard greatly increased in market share. Another common wall finish using gypsum lath (button board) also became popular after World War II. In the 1970’s up to 89% of new residential construction was made using gypsum drywall. This increased usage is based upon various advantages of the drywall system: sound control, speed and relative cleanliness of construction, availability of attractive and unique final finishes. Today drywall is widely accepted in commercial, industrial, and institutional construction.

Various components make up a drywall system: the supporting framing, the gypsum wallboard, the joint tape and the joint compound. Different methods of installation are used, from the relatively labor intensive manual installation to the high production mechanical installation methods. Manual installation is performed using trimming knives, wallboard hammers, and spackling broad knives. Mechanical installation uses power screwdrivers, nailguns, automated taping systems and automated compound applicators.

A survey was conducted of local subcontractors to supplement the knowledge gained from the available literature. Questionnaires were mailed to 67 local drywall contractors in the San Jose area. These addresses were obtained from those listed in the local phone directory. The questionnaire is included in Appendix A. A total of 19 replies were received, a response rate of 28%. The results of the survey show a wide variety of drywall practices for residential construction. One strong showing was the use of drywall screws as the primary means of attachment to the framing. Paper tape also was seen to be the dominant means of supporting the
drywall joint. Screw spacing, stud material and topping compound were seen to vary between several options.

**Experimental Testing of Gypsum Drywall Specimens**

Experimental testing of wall systems is usually conducted following the protocol defined by the American Standards of Testing and Materials in their specification E 564-94 (ASTM, 1994). Important engineering parameters include ultimate shear strength, internal shear displacement, and internal shear stiffness. To aid in engineering design these values are usually determined based upon a unit length of wall.

Experimental testing of drywall systems has been conducted by various institutions and a comparison of the engineering parameters is shown in Table 1. The American Plywood Association has a long history of testing wall systems for residential construction. One report (Adams, 1974, pp. 23-26) discusses several wall types, including those with gypsum wallboard. Test Series 5 included gypsum wallboard alone and in combination with plywood as listed in Table 2. The objective of this test series was to determine the performance of various constructed walls built from plywood. Gypsum wallboard was included in the study because of the fire-resistance that gypsum would contribute.

### Table 2:
Wall Systems Tested by the American Plywood Association

<table>
<thead>
<tr>
<th>Test</th>
<th>Sheathing</th>
<th>Attachment of Wallboard</th>
</tr>
</thead>
<tbody>
<tr>
<td>APA 25</td>
<td>1/2” Gypsum wallboard on Side 1, 5/16” Structural I plywood on Side 2</td>
<td>Nails 7 inches on center</td>
</tr>
<tr>
<td>APA 26</td>
<td>1/2” Gypsum wallboard on Side 1, Side 2 unsheathed</td>
<td>Nails 7 inches on center</td>
</tr>
<tr>
<td>APA 27</td>
<td>1/2” Gypsum wallboard covered with 3/8” Group 4 (Cedar) Channel groove plywood on Side 1, Side 2 unsheathed.</td>
<td>8d galvanized casing nails at 4” on center at edge and 12” on center in the field</td>
</tr>
<tr>
<td>APA 28</td>
<td>1/2” Gypsum wallboard covered with 3/8” Group 4 (Cedar) siding on Side 1, Side 2 unsheathed.</td>
<td>8d galvanized casing nails at 4” on center at edge and 12” on center in the field</td>
</tr>
<tr>
<td>APA 29</td>
<td>1/2” Gypsum wallboard covered with 3/8” Group 4 (Cedar) Channel groove plywood on Side 1, Side 2 unsheathed.</td>
<td>8d galvanized casing nails at 6” on center at edge and 12” on center in the field</td>
</tr>
<tr>
<td>APA 30</td>
<td>1/2” Gypsum wallboard covered with 3/8” Structural I plywood on Side 1, Side 2 unsheathed.</td>
<td>10d common nails at 4” on center at edge and 12” on center in the field</td>
</tr>
</tbody>
</table>

The test specimens were 8 feet high and 8 feet long. Test protocols appear to be forerunners of the current ASTM standards. The drywall nails used were 1-1/4” long, with 0.098” diameter, annular ring nails with 1/4” diameter heads.
When the study compared bare gypsum wallboard to that of wallboard and plywood, the behavior was seen to be additive.

Karacabeyli and Ceccotti (1996, pp. 2-179 to 2-186) report on a series of shearwall tests, built from plywood sheathing, 3/8” oriented strand board, and 1/2” gypsum wallboard, Type X. The tests were conducted on specimens 8-feet (2440 mm) high and 16-feet (4880 mm) long. Framing was placed 16” on center (406 mm). The top edge of the wall was restrained from vertical movement and actuators were used to produce a vertical load on the wall. Walls were horizontally blocked at midheight.

Test specimens were loaded either monotonically or with increasing-displacement cycles. For cyclic tests, each displacement-control level was repeated three times to determine a stable relationship. From the series of tests, four specimens are of particular interest to this report, as listed in Table 3.

<table>
<thead>
<tr>
<th>Test</th>
<th>Sheathing</th>
<th>Attachment of Wallboard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall 24</td>
<td>Gypsum wallboard on Side 1, Side 2 unsheathed</td>
<td>Screws</td>
</tr>
<tr>
<td>Wall 25</td>
<td>Gypsum wallboard on Side 1, Side 2 unsheathed</td>
<td>Roofing Nails</td>
</tr>
<tr>
<td>Wall 26</td>
<td>Oriented strand board on Side 1, gypsum wallboard on Side 2</td>
<td>Screws</td>
</tr>
<tr>
<td>Wall 27</td>
<td>Oriented strand board on Side 1, gypsum wallboard on Side 2</td>
<td>Roofing Nails</td>
</tr>
</tbody>
</table>

The gypsum wallboard was attached with either 1-1/4”-long screws or 1-1/2”-long roofing nails having a diameter of 1/8” and head diameter of 0.4 inch. Both screws and roofing nails were placed at 8 inches on center around the perimeter and 12 inches on center on the interior framing supports.

The reported findings included several issues related to the current project. Defining a yield point was found to be difficult because the load-deflection curves were nonlinear from very low loads. The goal of the project was to determine reliable information for the use of gypsum wallboard as a lateral force-resisting element of a building. To obtain these values, the authors suggested that the results from monotonic (ramp) loaded specimens be adjusted with undetermined factors to account for strength degradation due to cyclic loading. The authors used the stabilized cycle, their third cycle at a given displacement level, to develop backbone curves of the cyclic test specimens.

This is one of the reported studies where gypsum wallboard was tested in a wall system combining two different wall covering, specifically wallboard and oriented strand board. The superposition of wall materials (oriented strand board and gypsum wallboard) was found to match the combined material specimens up to a lateral displacement level of 1.25 inch. Likewise the addition of wallboard to a wall system with oriented strand board indicated an additional resistance to force, but a lower level of ductility after peak resistance. In addition the peak
resistance occurred at a lower level of displacement when gypsum was added to OSB. Apparently the wallboard significantly increases the stiffness of the wall system.

In another study of shearwall systems, gypsum wallboard was tested under load-controlled cyclic testing (Merrick, 1997). Wall systems tested were plywood, oriented strand board and 1/2” gypsum wallboard. The wallboard specimen was 8 feet high and 13’-1” long with wallboard on both sides of the specimen. The wallboard was attached with 1-5/8”-long wallboard nails with a head diameter of 0.086 inch and a shank diameter of 0.281 inch. Wallboard was installed with the panel vertical, allowing for all edges to be blocked. Horizontal load was applied at the top of the wall and the wall was anchored using commercially available hold-downs.

These tests were done under cyclic load control, at rates approaching dynamic loading (5 cycles per second). Relevant test specimen results are shown in Table 1 and Figure 1. The walls failed by the edges of the gypsum panels weakened and allowed the nails to pull through the face of the panel. Due to the force-controlled testing, lateral displacement was recorded to drift levels far beyond usable limits. The energy reported in the table is for a backbone curve recording this entire record.

Rose (1999) has recommended additional testing of wallboard systems to determine engineering requirements. His recommendation is for testing of 1/2” gypsum wallboard with horizontal joints and 5d wallboard nails spaced 7 inch oc.

A written survey was conducted of industry personnel to provide additional information about the construction of wall systems. The majority of surveys were sent to drywall sub-contractors listed in the phone directory for San Jose. In addition, surveys were distributed to design professionals familiar with drywall installation. A total of 67 surveys were distributed and a total of 19 were returned and analyzed. The survey is included in the Appendix of this report. Of the respondents 11 were subcontractors, 3 were drywall retailers, 3 were engineers and 1 was an architect. The average respondent had been involved in the industry for 25.8 years. The respondents wrote that their responses represented tract housing, custom home, multi-family and commercial construction.

Typical drywall thickness was essentially split between 5/8 inch and 1/2 inch, with 5/8 being indicated more often. Other reported thicknesses were 3/4 inch and 1 inch. The initial fastener used to hold the wallboard in place was either drywall screws (11 responses) or phosphorus coated nails (7 responses). When asked the type of fastener used to complete the attachment of the wallboard to the framing, the use of screws increased (15 responses to 3). The spacing between fasteners along the edge of the panel was 8 inches (10 responses), 7 inches (5 responses), 6 inches (3 responses) and either 12 inch or 16 inch (1 response each). Typical field spacings were given as 12 inches (9 responses), 8 inches (3 responses), 7 inches (3 responses), and either 18 inch or 24 inch (1 response each).

Reinforcement for the joint was given as paper tape (17 responses) or fiberglass strip (2 responses). Joint embedding compound was either all-purpose compound (12 responses), or vinyl ready-mix or quick-setting compound (3 responses each). The joint topping compound was
vinyl ready-mix (10 responses) all-purpose compound (4 responses), or quick-setting compound (3 responses).

One issue was not considered in the survey but may have relevance to this research project. Traditionally, wallboard was attached with fasteners on all four edges. In recent years the drywall industry has begun promoting the use of floating edges. This construction method eliminates the fasteners on the top edge of the wallboard sheet. One obvious advantage of this is the reduced labor required for fastener installation. Moreover, the finished wall apparently has an improved service life as cracking is reduced in the top joint. This floating edge development is apparently a response to cracking caused by settlement, moisture or temperature, but may also be beneficial for reduced seismic cracking. The survey did not address this issue, but conversations with subcontractors indicate that this floating edge construction is common, but far from universally applied.

FIGURE 1: Load-Deflection Curve for 1/2” Gypsum Wallboard

From Merrick, 1996.
Existing Engineering Design Guidelines for Gypsum Drywall

Engineering of residential construction in California is generally governed by the guidelines of the Uniform Building Code. Historically gypsum wallboard systems were sufficient to support one-story single-family residences. Recently the shear strength assigned to wallboard has been decreased significantly, to the point that the viability of designing a structure supported only by gypsum wallboard is limited.

**Figure 2:**
Load-Deflection Design Curves for 1/2” Gypsum Wallboard and Portland Cement Wall Systems


Currently the 1997 Uniform Building Code establishes a strength of 1/2 inch, blocked with 4” nail spacing gypsum drywall shearwall to be 150 lb./foot. This value is reduced by 50% for seismic zones 3 and 4. The resulting design strength is so small that its contribution is often dismissed during design.

FEMA-273 provides guidance on the seismic behavior of many structural systems, including lightframe wood shear walls. It discusses wall finishes and provides estimates of the force-displacement relationship for some of these materials, as shown in Figure 2.

Non-engineered woodframe construction is identified as conventional construction in the Uniform Building Code (International Council of Building Officials, 1997). This prescriptive section of the code defines maximum wall spacing and limited information about fastener use. However, the industry appears to follow these requirements loosely, and thus limited value can be obtained from this portion of the building code.
Portland Cement Construction

An alternative wall finish to gypsum wallboard is the coating of supporting members with a cementitious material made of sand and portland cement. This wall system is officially titled portland cement plaster, but also is commonly referred to as stucco by the general public. Cement can be supported by gypsum lath, wood lath, or interwoven wire. This supporting material is in turn supported by some form of framing, often lumber. Cement plaster was the common interior wall finish until gypsum wallboard became commonly available. Since the rapid growth and use of gypsum drywall, plaster and portland cement have become limited to exterior wall finish today.

The wall system has four primary components: portland cement plaster, reinforcing mesh, membrane, and wood framing (Dost and Botsai, 1990). Plaster should be made from portland cement with a 2.5-inch slump. Similar to concrete, plaster shrinks as it cures. It is desired that the cracks caused by shrinkage be several small hairline cracks rather than a few large ones. Large cracks are usually a result of uneven thickness of the plaster. Plaster is applied in at least three coats, first a scratch coat, second a brown coat, and finally at least one finishing coat. The reinforcing mesh can be of various weights and strength. One recommendation is the use of galvanized 1-1/2 inch 17-gauge wire mesh. This mesh can be installed on the wood framing using gasketed nails to allow for relative movement between the framing and plaster. The membrane is a heavy-duty material; the use of a #30 felt is common. Alternatives to the wire and felt membrane include self-furred, paper-backed lath and less recommended factory-laminated felt/paper system. The perimeter of the wall is surrounded with drip screens to allow for the weepage of moisture from the wall.

There are disadvantages to the use of plaster, in addition to the potential problems with shrinkage cracking mentioned above. During application, sand/plaster introduces literally tons of water into a construction job, increasing the chance of warpage of the wood framing. Another disadvantage to stucco is the potential delay in construction time due to the drying time between the multiple coats of plaster can delay construction time. A third disadvantage is that the thermal expansion of plaster is not consistent with that of lumber, leading to the formation of thermal expansion cracks. For all of these reasons, plaster is often cracked before an earthquake event. One recent change to plaster construction has been the use of glass or polypropylene fiber to reduce cracking.

Construction of plaster wall systems can be made using the line-wire installation technique. When properly constructed the resulting system allows for limited slip to occur between the plaster skin and the lumber framing. This slip is made possible by the introduction of a slip sheet between the membrane and the plaster during construction. This slip sheet can be almost anything, including newsprint or Grade D building paper.
Experimental Testing of Portland Cement Walls

Currently, no experimental testing of portland cement walls has been retrieved and studied by the research team. Additional research will be conducted to determine if unknown sources of data exist.

Existing Engineering Design Guidelines for Portland Cement Walls

Information for the engineering behavior of portland cement, plaster, or stucco wall is available from the International Council of Building Officials. A summary of some of the information available is listed in Table 3. As can be seen, the allowable design strength varies significantly for these walls, based upon the year of construction. Additionally, the federal government has provided guidance for the nonlinear behavior of these types of walls. There appears to be no variation between “engineered” construction to provide lateral strength and “unengineered” construction not intended to be part of the lateral force resisting system.

The FEMA 273 (Federal Emergency Management Agency, 1997, pp. 8-19) document also attempts to provide a force-displacement relationship for portland cement walls. In addition to the yield strength given in Table 3, the guidelines recommend a strength reduction of 80% after the deflection ductility at full strength is reached. Considering no deflection of the anchorage and a shear modulus of 14,000 lb./in., they recommend a yield deflection of 2.4 inches for an 8'-0" tall wall. Deflection ductility at full strength is given as 3.6 for wall aspect ratios of 1.0 and 2.5 for wall aspect ratios of 2.0. Likewise, an ultimate deflection ductility of 4.0 is recommended for the stout walls and a ductility of 3.0 for the slender walls. For this project it is convenient to show the displacement as a dimensionless drift value instead.

Table 3:
Engineering Design Guidelines for Portland Cement Walls

<table>
<thead>
<tr>
<th>Source</th>
<th>Allowable Shear Strength</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1997 Federal Emergency Management Agency – FEMA 273</td>
<td>350 lb./foot</td>
<td></td>
</tr>
<tr>
<td>1997 Uniform Building Code</td>
<td>180 lb./foot</td>
<td>7/8 inch thick plaster, unblocked, with nails spaced 6 in. oc.</td>
</tr>
<tr>
<td>1955 Uniform Building Code</td>
<td>675 lb./foot</td>
<td>1-1/2 inch thick plaster</td>
</tr>
</tbody>
</table>

1955 values based upon 1500 psi concrete wall strength per Section 4713 and Table 26-B.

Analytical Modeling of Wall Finish Materials

Various types of models can be envisioned to analyze gypsum wallboard partitions, depending on the degree of resolution desired in the results. At a very localized level, detailed nonlinear continuum finite element models can be employed to investigate concentrations of stress and
deformations around fasteners, corners of openings, and joints between panels. Modeling at the next level might consider the panel as a quasi-elastic with nonlinear fastener models, recognizing data from tests that shows that this is often the primary source of flexibility and nonlinearity in the wall panel system. Finally, at a more global level, the overall hysteretic response of panel assemblies can be represented by equivalent nonlinear struts or springs. These provide a practical means for incorporating the nonlinear panel response into the analysis of an overall building system.

**Analysis Conducted by Previous Researchers**

Of the three levels of analysis described above, there is some previous research on modeling at the intermediate and global level. In a project focusing on the lateral wind load behavior of gypsum sheathed houses, Alsmarker (1992) developed a nonlinear connection spring element that he incorporated in analyses where the walls are discretized into individual panel elements. Nonlinear force-deformation characteristics of the connectors were determined through simplified connector tests, and the entire analytical procedure was then compared to subassembly tests of gypsum wallboard ceiling panels. Overall, this project provides useful data and ideas on how to idealize the wall panels and the nonlinear connectors. However, from the standpoint of the current project, Alsmarker’s work is limited in the sense that it is geared toward lower stress/deformation levels associated with wind loading, and all the nonlinear analyses and tests consider only monotonic response.

Projects by Kunnath et al (1994) and Smith and Vance (1996) provide examples of more global models to simulate nonlinear hysteretic behavior of the entire wall panel. Kunnath et al. developed and implemented a model in the program IDARC2 using data from poured gypsum roofing systems. Smith and Vance developed and implemented a model in DRAIN2D based on previously published data on gypsum wall board systems.

**Cost-Damage Relationships Available in Literature**

Economic cost is an important measure of the damage from earthquakes. This monetary representation reflects the direct economic loss that a region will receive, and can be used to estimate insurance levels and to compare relative risk mitigation programs.

To improve understanding of the performance levels associated with this research project information has been collected about the damage levels of historic earthquakes and the costs associated with relative tasks of construction and reconstruction. In addition it is crucial to global risk management strategies that the total value of existing structures and their relative materials be established. Although this global risk evaluation is beyond the scope of this task, relevant information is presented here as a starting point for a more ambitious project in the future.
Cost Estimates from Past Earthquakes

During the 1971 San Fernando earthquake damage to residential construction was substantial and post-earthquake investigations reviewed 12000 residential structures (Oakeshott, 1975, pp. 329-334). In their report they categorized building damage as being severe, moderate or slight. Severe damage represented buildings with permanent distortion, separation of the framing from the foundation, portions of the plaster were loose or had broken free, wallboard required replacement or retaping of the joints, and/or roofs at incipient collapse. Meanwhile slight damage indicated minor cracking of the wall finish material, the enlargement of pre-existing cracks, or repair could be achieved by spackling of the plaster or wallboard. Structures were rated moderate when they appeared between these two extremes.

Of the structures reviewed, 78.0% were rated with slight damage to interior gypsum wallboard, with 6.5% rated moderate, 3.4% rated severe, and 12.1% with no damage. Likewise 78.4% were rated with slight damage to interior plaster, 11.1% rated moderate, 6.3% rated severe, and 4.2% with no damage. As for exterior stucco finishes, 74.1% of the structures were rated slight, 4.0% were rated moderate, 1.2% were rated severe, and 20.7% were rated with no damage. In addition, the investigators found a strong correlation between the ratio of damage cost to pre-earthquake value and whether the construction was pre-1940, two-story dwellings, and dwellings combining both one and two story portions. Of the categories, combination one and two story dwellings showed the most damage, two-story dwellings were next worse, and pre-1940 construction was third.

Relative Cost of Wall Construction Components

The Department of Commerce publishes construction industry cost statistics on a 5-year basis. The 1992 census reports an annual volume of total value of construction work of $14,055,774,000 in the United States. Of that amount, 58.7% is for construction of detached single-family homes and attached multi-family residential units.

The census also reports the amount of money going to specialty sub-contractors, which can be used as an estimate of the relative cost of various portions of the drywall system. In 1992, from the $14.2 billion total cost, $6.6 billion went to drywall, sheetrock, spackling and finishing subcontractors (46.5%), $1.4 billion went to plastering contractors (9.9%). Thus the ratio of construction work of drywall is apparently an 82.5% market share, nationwide. In comparison to the wall materials, the painting subcontractors accounted for $0.12 billion dollars. Thus it would appear that of the total cost of building and painting wall frames ($8.12 billion), the cost of the painting is only 1.5% of the total cost of the project. Hence, repainting after an earthquake should not cause appreciable dollar losses, that instead the cost of repair is in the repair and/or reconstruction of the wallboard and framing.
Conclusions from the Literature Review

After reviewing the available literature and the results of the practitioners’ survey, the following conclusions were drawn.

1. The two most common methods of finishing walls for residential woodframe construction are gypsum drywall construction and portland cement plaster construction. Gypsum drywall accounts for approximately 80% of the wall finish market.

2. Testing of drywall systems has been conducted on a limited basis to attempt to quantify the engineering parameters of strength and stiffness. However results have varied significantly and the population of tests has been small. No experimental test results for portland cement systems has been identified.

3. Engineering parameters for design have been tabulated for several years, however the values have changed significantly over time. In addition the parameters for the force-deflection relationship seem to vary significantly from published experimental data.

4. While limited experimental work has identified some engineering parameters, no published data quantifying the cost-damage relationship has been found.

5. While engineered shear wall systems seem to be well defined, the detailing of conventional construction has been limited in publication. As a result, the survey results showed a wide variation in the specific construction practices used today.
Table 1: Experimental Results of Gypsum Drywall Experiments

<table>
<thead>
<tr>
<th>TEST</th>
<th>WALL THICKNESS</th>
<th>CURVE</th>
<th>MAX. SHEAR LOAD</th>
<th>INITIAL STIFFNESS</th>
<th>SECANT STIFFNESS</th>
<th>ENERGY DISSIPATED</th>
</tr>
</thead>
<tbody>
<tr>
<td>Units</td>
<td>inch</td>
<td></td>
<td>Lb./foot</td>
<td>Lb./inch/foot</td>
<td>Lb./inch/foot</td>
<td>Inch-lb./foot</td>
</tr>
<tr>
<td>APA-26</td>
<td>0.50</td>
<td>Monotonic</td>
<td>400</td>
<td>600</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>K-24</td>
<td>0.50</td>
<td>Monotonic</td>
<td>280</td>
<td>6000</td>
<td>720</td>
<td>50</td>
</tr>
<tr>
<td>K-24</td>
<td>0.50</td>
<td>Stabilized</td>
<td>230</td>
<td>3800</td>
<td>920</td>
<td>20</td>
</tr>
<tr>
<td>K-25</td>
<td>0.50</td>
<td>Monotonic</td>
<td>390</td>
<td>4800</td>
<td>500</td>
<td>70</td>
</tr>
<tr>
<td>K-25</td>
<td>0.50</td>
<td>Stabilized</td>
<td>230</td>
<td>6000</td>
<td>590</td>
<td>20</td>
</tr>
<tr>
<td>A-Ia</td>
<td>0.51</td>
<td>Monotonic</td>
<td>170</td>
<td>N/A</td>
<td>370</td>
<td>10</td>
</tr>
<tr>
<td>A-IIc</td>
<td>0.51</td>
<td>Monotonic</td>
<td>270</td>
<td>3500</td>
<td>640</td>
<td>10</td>
</tr>
<tr>
<td>M-GB1-B</td>
<td>0.50</td>
<td>Maximum</td>
<td>640</td>
<td>3500</td>
<td>1100</td>
<td>N/A</td>
</tr>
<tr>
<td>M-GB2</td>
<td>0.50</td>
<td>Maximum</td>
<td>640</td>
<td>2900</td>
<td>2300</td>
<td>N/A</td>
</tr>
</tbody>
</table>

The data for this table is approximate, determined from printed reports rather than digital data. Use of the data for actual engineering projects is not recommended.

1. Initial stiffness is determined as the tangent at initial loading.
2. Secant stiffness is determined at maximum load.
3. Energy is determined using a backbone curve for cyclic tests.
References


Task 1.4.7 – Innovative Systems for Seismic Protection of Wood Structures

Literature Review

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Summary

This report provides a literature review for Task 1.4.7 – Innovative Systems of the CUREE-Caltech Woodframe Project. Specifically, literature on the application of base isolation systems and supplemental damping systems to light-framed wood structures is reviewed. The reviewed literature demonstrates that innovative systems offer promise for enabling light-framed wood structures to resist strong earthquakes.

Introduction

Light-framed wood construction has generally been regarded as performing well during strong earthquakes. Such performance is primarily due to the low mass of light-framed construction combined with its ability to deform inelastically without inducing collapse of the structure. Although light-framed wood structures typically do not collapse during strong earthquakes, the inelastic response is generally associated with significant structural and non-structural damage that may be very costly to repair. As an example of the magnitude of the damage to light-framed wood buildings during strong earthquakes, consider the 1994 Northridge Earthquake in which there was in excess of 20 billion dollars worth of damage to such structures (Kircher et al., 1997). Obviously, the 1994 Northridge Earthquake provides clear evidence that conventional light-framed wood buildings are prone to significant damage when subjected to strong earthquake ground motions.

Innovative Seismic Protection Systems

One approach to mitigating the effects of strong earthquakes on light-framed wood buildings is to incorporate an innovative seismic protection system within the building. For example, the response of the building can be de-coupled from the earthquake ground motion by introducing a flexible interface (i.e., a base isolation system) between the foundation and superstructure. The isolation system either shifts the fundamental period of the structure to a large value or limits the amount of force that can transferred to the structure such that interstory drifts (and thus structural damage) and floor and roof accelerations (and thus nonstructural damage) are reduced significantly. Alternatively, the seismic response of a building can be reduced by introducing a supplemental damping system within the framing of the building. The supplemental damping
system dissipates a portion of the seismic input energy, reducing the amount of energy dissipated via inelastic behavior within the structural framing.

The number of applications of innovative seismic protection systems within buildings has been steadily growing within approximately the past ten years. Nearly all of these applications have been within either steel or concrete structures (e.g., see Soong and Constantinou, 1995). The two most common types of base isolation systems utilize either rubber bearings or sliding bearings between the foundation and superstructure. In contrast, there are a wide variety of supplemental damping systems available for implementation in buildings (Constantinou and Symans, 1993a and Constantinou et al., 1998). However, the most rapid growth in the application of supplemental damping systems to buildings has occurred for fluid dampers. Since the first experimental studies on a scale-model building frame in 1993 (Symans and Constantinou, 1993b), the number of implementations of fluid dampers within bridge and building structures has grown to 30 with installation pending in 16 additional structures. Although there are many factors that have contributed to this rapid growth, one of the primary reasons is the high energy dissipation density of fluid dampers. That is, fluid dampers are capable of dissipating a large amount of energy relative to their size.

The number of studies that have been conducted on the application of innovative systems for seismic protection of wood frame structures is relatively small. However, based on the results of the studies that have been conducted, innovative systems appear to offer promise for improving the seismic response of light-framed wood construction. Base isolation systems for wood-framed construction have been studied by Delfosse (1982), Reed and Kircher (1986), Sakamoto et al. (1990), Pall and Pall (1991), and Zayas and Low (1997) while supplemental damping systems have been studied by Filiatrault (1990), Dinehart and Shenton (1998), and Dinehart et al. (1999).

**Base Isolation Systems for Light-Framed Wood Buildings**

Delfosse (1982) discusses issues associated with the application of base isolation systems to light structures. It is noted that the design of an elastomeric bearing isolation system for light structures may be difficult due to the low mass of such structures. As a rough approximation, assume that a light structure supported on an isolation system behaves as a rigid body having mass $m$. In this case, the natural period of the base-isolated structure is given by

$$T = 2\pi \sqrt{\frac{m}{k}}$$  \hspace{1cm} (1)

where $k$ is the lateral stiffness of the isolation system (assumed to behave elastically). Assuming the desired range of natural periods of base-isolated structures is from 1.5 to 4 seconds, the corresponding range of stiffness to mass ratios is:

$$2.5 < \frac{k}{m} < 17.5$$  \hspace{1cm} (2)
Thus, for structures with low mass (such as light-framed wood construction), the stiffness must be low enough to provide stiffness to mass ratios of less than about 20. The requirement for very low stiffness tends to result in slender elastomeric bearings which may be prone to excessive shear strain and buckling. In spite of these apparent difficulties, Delfosse (1982) demonstrates, through an example design, that it is feasible to utilize an elastomeric base isolation system for a single-story wood frame house.

Reed and Kircher (1986) discuss the seismic upgrade of a five-story wood-frame building using two different isolation system configurations; one with elastomeric bearings and the other with sliding bearings. The building, constructed in 1886, is part of the Naval Postgraduate School in Monterey, California and has both historical and architectural significance to the U.S. Navy. For both isolation system configurations, a horizontal steel truss system was designed to stiffen the first floor so as to achieve rigid diaphragm action immediately above the isolation level. In addition, a vertical truss system was designed to transfer loads between the isolation system and the wood-framed superstructure.

Furthermore, two restraint systems were developed; one to maintain rigidity under wind loading and the other to prevent building collapse under extreme earthquake loading. As is often the case for base-isolated buildings, the isolation systems were designed to limit the forces transferred into the superstructure such that the superstructure remains essentially elastic under the design earthquake. Thus, the wood framing above the first-floor required no modification, which is an important consideration for structures having historical value. The fundamental natural period of the base-isolated structure that utilizes elastomeric bearings is 2.3 seconds. The structure with the sliding isolation system configuration is characterized by a sliding coefficient of friction of about 1%. Numerical analyses of the existing and retrofitted (i.e., base-isolated) structures subjected to three earthquake time-histories, scaled to match the peak ground acceleration of the design spectrum, produced peak base shear response reductions ranging from 74% to 98%. Thus, the isolation systems appear to be very effective in terms of limiting the force transferred into the wood-framed superstructure.

Sakamoto et al. (1990) present an experimental and analytical study of a two-story light-framed wood building supported on a base isolation system. The building was constructed at the University of Tokyo for experimental testing purposes. Plan and elevation views of the structure are shown in Figure 1. The footprint of the building is 7.28 m x 10.92 m and the total weight is 549 kN. The base isolation system consists of six laminated elastomeric bearings located along the perimeter of the foundation (see Figure 1). Three different types of bearings were used in the experimental testing (see Figure 2). The hysteretic behavior of the isolation system for the building with multi-stage rubber bearings (Type B: Center bearing of Figure 2) is shown in Figure 3. All of the bearing types were designed to provide a fundamental natural frequency of 0.5 Hz at a displacement amplitude of 15 cm, an equivalent viscous damping ratio of 15%, and an allowable deformation of 25 cm. For bearing displacements larger than 15 cm, a fail-safe support mechanism is activated in which the bottom of the first floor contacts the top of a rigid pedestal. Additional motion of the isolation system is impeded by sliding friction between the first floor and pedestal.
Free vibration tests of the isolated building in the direction of the short plan dimension revealed two dominant frequencies at about 0.8 Hz (first mode) and 8.5 Hz (second mode). The motion of the building with the multi-stage rubber bearings was also recorded during three different earthquakes. The strongest ground motion recorded and associated acceleration response of the building in the direction of the short plan dimension (i.e., the X-direction of Figure 1) are shown in Figure 4. Note that the peak ground acceleration at the building site was 0.108g. The effectiveness of the isolation system is demonstrated by the reduction of acceleration (approximately 70%) transmitted from the ground level to the first floor. The dominant frequencies in the recorded acceleration response occurred at about 2.1 Hz and 9.7 Hz. The difference in fundamental frequencies for the free vibration response (0.8 Hz) and earthquake loading response (2.1 Hz) is attributed to the nonlinear behavior of the rubber bearings. The maximum displacement in the free vibration and earthquake loading tests was 15 cm and 0.1 cm, respectively. Since the effective stiffness of the bearings reduces with increasing displacement (see Figure 3), it is expected that the free vibration response (larger amplitude, lower stiffness) would be dominated by lower frequencies of motion. Finally, numerical simulations were performed for the building in a fixed-base and base-isolated configuration and subjected to the same earthquake as that described above. The simulations indicated that the peak acceleration at the roof of the fixed-base structure would be about ten times as large as for the base-isolated structure.

Figure 1 – Plan and Elevation Views of Two-Story Light-Framed Wood Building with an Elastomeric Base Isolation System (adapted from Sakamoto (1990)).
Figure 2 – Configuration of Elastomeric Bearings used in Experimental Wooden Building (adapted from Sakamoto (1990)).

Figure 3 – Hysteretic Behavior of Isolation System for Building with Multi-Stage Elastomeric Isolation Bearings (adapted from Sakamoto (1990)).
In 1998, a base isolation system was implemented within a two-story light-framed wood house in Montreal, Canada (Pall and Pall, 1991). A photograph of the house is shown in Figure 5 and a plan view showing the location of the isolation bearings is shown in Figure 6. The house has two stories above grade and a basement below grade. The basement walls are reinforced concrete and the superstructure consists of light-framed wood construction with brick veneer. The isolation bearings used in this application are flat sliding bearings. A total of 15 bearings were installed along the perimeter of the basement wall. Note that the bearings were flat in the center with ramped surfaces along the edges (see Section 1-1 of Figure 6). The ramped surfaces provide increasing resistance at large displacements. The isolation gap is protected from the elements via flashing on the outside of the gap (see Section 1-1 of Figure 6). The coefficient of friction associated with the sliding interface of the bearings is 0.2. The coefficient of friction determines the maximum lateral force that can be transferred to the superstructure. In addition, the sliding friction dissipates energy as the bearing is cycled. Note that the concept of natural period does not apply to structures supported on flat sliding bearings and thus resonant response is not a concern.

Numerical analyses of the house in a fixed-base and base-isolated configuration were performed with earthquake ground motion represented by an artificial earthquake record. The earthquake record was developed to be consistent with the design response spectrum of the National Building Code of Canada. The results of the analyses show that the acceleration at the top of the structure is reduced by about 42% for the design peak ground acceleration of 0.18g. This reduction in acceleration response is accompanied by a peak bearing displacement of 3 mm and a permanent displacement of 1.5 mm. Furthermore, as the peak ground acceleration increases, the
effectiveness of the isolation system increases (i.e., the reductions in response acceleration are larger). Interestingly, the improved seismic performance did not reduce the cost of materials for construction of the house since standard size materials were used. However, Pall and Pall (1991) suggest that the cost of friction base isolation bearings is low and thus there is promise for widespread application of such bearings in low-rise construction including wood-framed houses.

**Figure 5** – Photograph of Wood-Framed House with Sliding Base Isolation System (adapted from Pall and Pall (1991)).

**Figure 6** – Plan View of Wood-Framed House Showing Location of Sliding Isolation Bearings (adapted from Pall and Pall (1991)).
The implementation of sliding bearings in a four-story wood-framed apartment building in San Francisco, California is discussed by Zayas and Low (1997). The building was severely damaged during the 1989 Loma Prieta earthquake and was retrofitted using sliding friction pendulum system (FPS) bearings. The four-story structure has a garage at the first story with apartments in the top three stories (see Figure 7). The damage to the first-story during the Loma Prieta earthquake was so severe that the entire first-story wood framing was replaced with a steel moment-resisting frame. The sliding bearings were installed under the base plates of each column of the steel frame (see Figure 8). Thus, the gravity loads of the structure are no longer transferred to the foundation over continuous sill plates but rather are transferred at the discrete locations of the bearings. The FPS bearings have a curved sliding surface with a constant radius of curvature (i.e., the sliding surface is circular in two dimensions and spherical in three dimensions) (see Figure 9). The motion of the structure when supported on the bearings is similar to that of a simple pendulum. The curved surface results in both energy dissipation due to sliding friction and energy storage due to the rising of the supported weight along the curved surface. In contrast to the behavior of structures supported on flat sliding bearings, structures supported on curved bearings exhibit a natural period that can be determined by analogy to the natural period of a simple pendulum:

\[ T = 2\pi \frac{R}{\sqrt{g}} \]  

(3)

where \( R \) is the radius of curvature of the bearings, \( g \) is the acceleration due to gravity, and it has been assumed that the superstructure behaves as a rigid body. Interestingly, the natural period is independent of the weight supported by the bearings. Thus, the low mass of light-framed wood structures does not pose a problem for an isolation system that utilizes FPS bearings (recall that the potential for buckling of slender elastomeric bearings is due to the low mass supported by the bearing). The final design of the bearings resulted in a natural period of 2 seconds, a coefficient of friction of 10\%, and a displacement capacity of ±19 cm.
Figure 7 – Photograph of Four-Story Wood-Framed Apartment Building Seismically Retrofitted with a Sliding Isolation System (adapted from Zayas and Low (1997)).

Figure 8 – Close-Up View of Column Showing Isolation Bearing Installed Below Base Plate (adapted from Zayas and Low (1997)).
The performance of the base-isolated structure was evaluated via nonlinear time-history analysis. The results for input being a design-level earthquake with peak ground acceleration of 0.75g is presented here. Note that the design earthquake was considered to be representative of a site-specific spectrum corresponding to a magnitude 7 earthquake on the San Andreas fault (approximately 11.3 km from the site). Relative displacement response profiles are compared in Figure 10 for two cases: 1) fixed-base configuration (but with first story strengthened by steel moment-resisting frame) and 2) isolated configuration. As expected, the displacement response of the isolated building is concentrated at the isolation level with very little interstory drift response in the superstructure. In contrast, the non-isolated building experiences significant interstory drift that is concentrated in the second story. The effect of the isolation system is to reduce the peak interstory drift by about 95%. The isolated building remains entirely elastic while the non-isolated building experiences a ductility demand of 38.4 for the second story. Zayas and Low (1997) suggest that such a high ductility demand is beyond the reasonable capacity of the wood-stucco frame and thus there is a significant risk of collapse of the non-isolated building. In summary, the numerical analyses demonstrate that the isolated structure would respond elastically if subjected to a magnitude 7 earthquake on the nearby San Andreas fault. Finally, it is noted that retrofitting the building using a seismic isolation system was cost effective in that its use avoided the need to add shear walls over the height of the structure and to remove and replace ornate finish materials having architectural significance.

Figure 9 - Cross-Section of Friction Pendulum System Bearing.
Supplemental Damping Systems for Light-Framed Wood Buildings

The seismic response of friction-damped timber shear walls has been studied by Filiatrault (1990). The cyclic lateral force-displacement relation for conventional light-framed shear walls exhibits both a progressive loss of stiffness and pinching of the hysteresis loops (see Figure 11). Such behavior is a direct result of damage to the shear wall. Note that, due to the pinching of the hysteresis loops, the shear wall must experience large deformations in order to dissipate the seismic input energy. Filiatrault (1990) proposed the concept of installing supplemental energy dissipation elements at the four corners of a shear wall (see Figure 12). The elements are friction dampers which absorb a significant portion of the seismic input energy, reducing the amount of energy that needs to be dissipated by the framing via inelastic behavior. The friction dampers consist of a slotted slip joint that dissipates energy via friction as the two sides of the joint slide with respect to each other. Sliding motion of the slip joint is induced by deformation of the four corners of the shear wall. Thus, the wood framing of the shear wall must deform to activate the friction dampers. However, since the friction dampers absorb a portion of the seismic energy input, the energy dissipation demand on the framing will be reduced. Note that the triangular configuration of the friction damper (see Figure 12) is beneficial in maintaining the integrity of the wood-framed corners of the shear wall. The stability of the mechanical properties of the friction damper are demonstrated in Figure 13 which shows the repeatability of the force-displacement relation for a single damper subjected to 50 cycles of sinusoidal motion at a frequency of 0.2 Hz and an amplitude of ± 1.52 cm.
Figure 11 – Hysteretic Behavior of Conventional Light-Framed Wood Shear Wall (adapted from Filiatrault (1990)).

Figure 12 – Shear Wall With Friction Dampers Located at Four Corners of Framing (adapted from Filiatrault (1990)).
Numerical analysis of a friction-damped shear wall (2.4 m x 2.4 m) was performed using nonlinear models for the shear wall and the friction dampers. The model of the shear wall accounts for both stiffness degradation and pinching of the hysteresis loops. Experimental test data obtained from shaking table tests of a shear wall without friction dampers demonstrated the suitability of the analytical model of the shear wall. A comparison of the hysteretic behavior of the shear wall with and without the supplemental friction dampers and subjected to the El Centro record (component NS) of the 1940 Imperial Valley Earthquake is shown in Figure 14. The friction dampers improve the seismic performance by reducing the peak force and displacement at the top of the wall and essentially eliminating the pinching of the hysteresis loops. The effectiveness of the friction dampers is also demonstrated in the energy time histories shown in Figure 15. There is a marked reduction in the hysteretic energy dissipation demand on the shear wall. Furthermore at the end of the earthquake, approximately 60% of the input energy is dissipated by the friction dampers.

The dynamic behavior of light-framed wood shear walls with viscoelastic dampers was experimentally evaluated by Dinehart and Shenton (1998) and Dinehart et al. (1999). The experimental tests were conducted on 2.44 m x 2.44 m (8 ft x 8 ft) shear walls framed with 38 mm x 89 mm (2 in. x 4 in., nominal) interior studs spaced at 40.6 cm (16 in.) on center. Two 11.9 mm (15/32 in.) thick plywood sheets were used as sheathing panels (see Figure 16). Four different damper configurations were investigated (see Figure 17 and 18). In each configuration, the viscoelastic damper dissipates energy via shearing action of a viscoelastic rubber-like material. The shearing action is induced by relative motion between the two ends of the damper at their points of attachment to the shear wall. Thus, the shear wall must deform for the viscoelastic dampers to dissipate energy. Note that, in addition to the corner damper...
configuration shown in Figure 17(a) where the dampers are located in the top corners, a separate configuration with the dampers located in the bottom corners was investigated.

Figure 14 – Effect of Friction Dampers on the Hysteretic Behavior of a Wood-Framed Shear Wall (adapted from Filiatrault (1990)).

Figure 15 – Effect of Friction Dampers on the Energy Distribution within a Wood-Framed Shear Wall (adapted from Filiatrault (1990)).
Figure 16 – Configuration of Conventional Light-Framed Wood Shear Wall (adapted from Dinehart et al. (1999)).

Figure 17 – Test Configurations for Shear Wall with Viscoelastic Dampers (adapted from Dinehart et al. (1999)).
The shear walls were supported at the bottom plate and a lateral load was applied at the top plate. The dynamic loading consisted of 72 cycles of varying displacement amplitude (specifically, the sequential phased displacement test protocol was used). The hysteretic behavior for the conventional wall (no dampers) and the wall with two different damper configurations (sheathing-to-stud and diagonal bracing) is shown in Figure 19. The damped wall configurations are those that were found to be most effective. As expected, due to the viscoelastic nature of the dampers, both the effective stiffness and energy dissipation increased by adding the dampers to the wall. However, one may note that the hysteresis loops are not dramatically different for the conventional and damped walls. Thus, the energy dissipated by the conventional and damped walls is not very different. However, it should be understood that the conventional walls must dissipate all of the input energy, primarily via inelastic behavior of the wall. In contrast, for the damped walls a large portion of the input energy is dissipated by the viscoelastic dampers and thus the energy dissipation demand on the wall itself is reduced significantly.

A comparison of energy dissipation per cycle for three shear wall configurations is shown in Figure 20. As for Figure 19, the damped wall configurations shown in Figure 20 are those that were found to be most effective. The energy dissipation is shown for four cycles of motion at a displacement amplitude equal to the so-called first major event (i.e., ±1.80 cm). Note that the
energy dissipation capacity of both the conventional and damped shear walls decreases as the number of cycles increases. However, the degradation for both configurations is due to the inelastic behavior of the shear wall (since the reduction in energy dissipation from the one cycle to the next is approximately equal for both configurations). Thus, the viscoelastic dampers provide a stable source of energy dissipation during cyclic motion of the shear wall. Such behavior is particularly important in terms of the ability of a wood-framed structure to resist strong earthquake aftershocks.

Figure 19 – Hysteresis Loops for Conventional Shear Wall and Viscoelastically-Damped Shear Wall (adapted from Dinehart et al. (1999)).

Figure 20 – Effect of Viscoelastic Dampers on Energy Dissipation (adapted from Dinehart et al. (1999)).
Application of Fluid Viscous Dampers for Seismic Energy Dissipation in Wood-Framed Buildings

To the author’s knowledge, the only application of fluid viscous dampers within a wood-framed structure is within a commercial building in Alaska. The building is a timber structure and was seismically retrofitted in 1997 using fluid dampers within diagonal bracing. Two fluid dampers having a force capacity of 445 kN and a stroke of ±64 mm were utilized. A schematic of a typical fluid viscous damper is shown in Figure 21. The damper dissipates energy by passing a viscous fluid through small orifices at high speeds, resulting in the development of heat energy which is transferred through the damper cylinder to the environment.

As mentioned previously, fluid viscous dampers offer particular promise for application within wood-framed buildings due to their high energy dissipation density (i.e., their ability to dissipate large amounts of energy in proportion to their size). Thus, it is likely that the dampers could be conveniently located within the walls of a wood-framed structure. In addition to their high energy dissipation density, the behavior of fluid dampers is quite unique in that they are incapable of developing appreciable restoring forces for the frequencies of motion expected during an earthquake (Symans and Constantinou, 1998). Thus, the dampers behave essentially as pure energy dissipation devices. The design of structures that incorporate such dampers becomes simplified since the dampers may be regarded as simply adding additional energy dissipation capacity to the structure. Of course, one must recognize that the installation of supplemental dampers will alter the load path for the transfer of forces within the structure.

![Figure 21 – Schematic of Fluid Viscous Damper.](image)

Typical recorded force-displacement loops for a 8.9-kN (2-kip) capacity fluid damper subjected to sinusoidal input motion are presented in Figure 22 for room temperature conditions (23° C) and frequencies of 1, 2 and 4 Hz. In this range of frequencies, typical of light-framed residential wood construction, the fluid damper exhibits insignificant storage stiffness and its behavior is essentially linear viscous (Symans and Constantinou, 1998).
The beneficial effects of fluid viscous dampers are illustrated by the energy response-histories shown in **Figure 23** for a 1:4-scale, one-story steel-framed model building with and without dampers. In **Figure 23**, the structure is subjected to the Taft record (component N21E) of the 1952 Kern County Earthquake. The top plot is for the bare structure (i.e., no dampers attached) and the bottom plot is for the structure with two fluid dampers attached within diagonal bracing. The energy quantities of relevance to the seismic behavior of a structure are the sum of the kinetic and elastic strain energies, $E_k + E_s$, which indicate the level of deformation in the structure, and the irrecoverable hysteretic energy dissipated by the structural elements, $E_h$, which indicates the level of inelastic action in the structure. It is clear from **Figure 23** that the addition of fluid dampers to the structure results in a reduction of both of these energy quantities and thus improves the seismic performance of the structure. In particular, one may note the significant reduction of energy dissipated by the structural components, $E_h$, in exchange for energy dissipation by the fluid dampers, $E_d$. 

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**Figure 22** - Experimental Force-Displacement Loops at Three Different Frequencies of Harmonic Motion for 8.9-kN (2-kip) Capacity Fluid Damper.
Future Research on Fluid Viscous Dampers for Seismic Protection of Light-Framed Wood Buildings

Based on the excellent experimental performance observed for model-scale buildings with fluid dampers subjected to earthquake loading (e.g., see Figure 23), an analytical and numerical study to assess the potential benefits of utilizing fluid viscous dampers for the seismic protection of light-framed wood buildings has recently been initiated by the authors as part of the CUREE-Caltech Woodframe Project. To the knowledge of the authors, this research represents the first study on the application of fluid dampers within wood-framed structures for seismic energy dissipation. The project involves the following four major tasks:

1) A literature review (the results of which are the subject of this report);

2) A numerical study of the dynamic response of light-framed shear walls with and without fluid dampers (similar to the studies performed by Filiatrault (1990) and Dinehart et al. (1999) on the application of friction dampers and viscoelastic dampers, respectively, to light-framed shear walls);

3) Investigation of practical issues such as the cost of utilizing fluid dampers and the placement and installation of fluid dampers;


This research project is ongoing and is scheduled for completion in March, 2001.
Figure 23 - Energy Response Histories for a One-Story Steel-Framed Structure with and without Two 8.9-kN (2-Kip) Capacity Fluid Dampers (adapted from Seleemah and Constantinou, 1997).
References


Task 1.4.8.1 - Effect of Fastener Head Penetration on Sheathing-to-Wood Connections

Literature Review

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Summary

There are two objectives included in this phase of the research. The first objective is the compilation of studies involving various dowel-type connections. The second objective is the supplement the data collected with original testing investigating the effect of fastener head penetration on the strength of vertical shear walls.

This paper covers background information relative to the second objective. The prominent research that has been performed relating to the effect of fastener head penetration in sheathing-to-wood connections is reviewed. There have been three independent papers published on the subject. The authors include Zacher and Gray; Andreason and Tissell; and Ficcadenti, Castle, Sandercock, and Kazanjy. The papers have been reviewed for test configuration, testing variables and conclusions.

A survey investigating the levels of fastener head penetration along the residential Wasatch Front is presented. This survey by Rabe and Jones, investigates the prevalence of overdriven fasteners in sheathing-to-wood connections in the vertical shearwalls of light-frame timber structures.

Previous Work

There have been several studies investigating the effects of overdriven fasteners in vertical shearwalls. All of these studies concluded that the level of fastener head penetration affected the strength of the shearwall, although the magnitude of the effect is disagreed upon. The discrepancy in conclusion is most likely due to different testing configurations and selection of sample variables. This is in part due to the large number of variables and cumbersome nature of full panel testing.
Edwin Zacher and Ralph Gray

Zacher and Gray (1989) reported on thirteen shearwall specimens and fifteen nailed wood joints. The shear walls were 8 ft (2400 mm) by 8 ft (2400 mm). All panels were constructed using 3/8 in CD plywood and 8d nails (box or common was not specified). Overdriven panels were constructed with 100% of the nails overdriven 1/8 in (3.175 mm) (Gray and Zacher, 1988, p. 123). Five panels were tested with nails. One was constructed with overdriven nails in controlled laboratory conditions. A local construction company, using materials from their “usual” suppliers, assembled two control panels with flush nails and two panels with overdriven nails. The wood joint samples were made of the same materials but were 16 in (406.4 mm) by 18 in (457.2 mm). The shear walls and wood joints were tested with a deflection controlled cyclic loading sequence running at 2 Hz.

Analysis of the shearwall tests showed overdriven fasteners to be unsatisfactory. The conclusions state that the panels failed suddenly at deformation levels expected to occur in a strong earthquake. Zacher and Gray recommend that further cyclic testing be done to investigate the effect of different panel thickness, material types, and fasteners.

Analysis of the wood joints concluded that the major source of deformation was nail flexure and movement of the nails in the wood. The panels itself exhibited little distortion. This points to the nail connection as the source of shear wall failure.

The results indicated the importance of fastener characteristics and installation, the effect of sheathing action and the value of dynamic testing. More importantly, the performance of plywood sheathed shear panels with overdriven nails was found to be unsatisfactory — the panels failed suddenly at deformation levels expected to occur in a strong earthquake. But, according to Dolan (1999), the results were questionable and were never widely accepted.

Kenneth Andreason and John Tissell

Andreason and Tissell (1994) from the APA - Engineering Wood Association, formerly the American Plywood Association, also investigated the effects of overdriven nails in shear walls. The investigation included coupon tests and full panel tests. Both sets of tests investigated variations of panel type, panel size and nail size.

The coupon tests compared flush-driven common nails to overdriven common nails. Four nail types were considered: 6d “T”, 6d common, 10d common short, and 10d common clipped head. All tests were done in plywood with thicknesses respective to nail sizes of 3/8 in (9.525 mm), 5/16 in (7.938 mm), 15/32 in (11.906 mm) (3 and 4 ply), and 15/32 in (11.906 mm) (3 and 4 ply). Nail penetration was done respective to amount of the first ply penetrated. The overdriven depths ranged from 0.06 in (1.524 mm) to 0.16 in (4.064 mm). Coupon samples were tested monotonically and in compliance with ASTM D1761. Each configuration was tested only once, with a total of 29 tests.
The panel tests compared shear walls with flush–driven common nails to shear walls with all nails overdriven by 1/8 in (3.175 mm). Shear wall specimens were 8 ft (2400 mm) by 8 ft (2400 mm) and constructed with plywood or OSB. A total of five tests were run. Two of the tests were done using 15/32 in (11.906 mm) thick plywood attached with 10d common short nails. Three of the tests were done using ½ in (12.7 mm) thick OSB attached with 8d common nails. The first of each set of tests was done with flush–driven nails. The panel tests were driven using the loading sequence recommended by ASTM E72. This sequence involves four cycles, although deflection never passes through zero.

The discussion summary (Andreason and Tissel, 1994, p. 6) reports a relatively small change in performance between flush and overdriven nails. The ultimate loads were lessened by 2 to 17% for the coupons and 10 to 15% for the shearwalls. The investigation concluded that overdriving will not significantly affect the strength of a shear wall until at least 20 - 25 percent of the nails are overdriven, at which point a 3 - 4 percent reduction in ultimate load occurs as compared to panels with flush–driven nails. This conclusion appeared to be unsupported by the testing done in the investigation, since all tests were conducted on panels with 100% of nails overdriven.

All nails used in testing were common nails. Ficcadenti et al. (1996, p.398) concluded that box nails behave differently than common nails in similar shearwall tests. Pneumatically controlled nail guns use box nails and are the major contributor to overdriving nails. This calls into question the exclusive use of common nails in investigating the effect of nail penetration.

**Seb Ficcadenti, Thomas Castle, Deborah Sandercock, and Robert Kazanjy**

More recently, Ficcadenti et al. (1996) conducted laboratory testing to investigate differences between box versus common nails, and pneumatically versus hand driven nailing. A total of nine tests were conducted. Four walls were built with hand driven common nails and five walls were built with pneumatically driven box nails. Three of the specimens were built with box nails having 20, 50, and 80 percent of the nails overdriven at least 1/16 in (1.588 mm). The remaining panels (4 common and 2 box) had flush-driven nails and were used for control. All sheathing was 3/8 in (9.525 mm) thick CDX plywood.

The shearwalls were tested with a reduced version of the Sequential Phased Displacement loading recommended by the Structural Engineers Association of California (SEAOC, 1996). The reduced version lowered the number of cycles at each increment of displacement.

They concluded that walls built with pneumatic box nails were as strong or stronger than similar walls built with hand driven common nails. Walls with high percentages of overdriven box nails were found to be as strong as the common nail walls and even had higher ductility.

The failure modes differed strongly between the two nail types. The common nails failed most often by pulling through the sheathing. The box nails failed by withdraw from the framing member. The failure mode changes for the box nails as the percentage of overdriven nails increased. An increasing number of the box nails failed by pulling through as a larger number were overdriven.
It is interesting to note that although increasing the percentage of overdriven nails steadily decreased the performance of the shearwall, all three tests with overdriven box nails had higher ultimate strengths and ductility than the two control walls built with flush-driven box nails. The reasons for these counterintuitive results were not clearly discussed. However, because the samples with pneumatic box nails failed by nail withdrawal, it can be deduced that the deeper the nail penetration (to a certain extent), the higher the capacity — deeper nails will require more energy to fail (by withdrawal), thus, increasing the wall capacity.

Survey

Rabe and Jones (1999) conducted a survey to assess the prevalence of overdriven fasteners along the residential Wasatch Front. In this survey, focus was given to nailed shear walls. For each locale, four lines of twenty nails were selected and measured for overdrive. All of the locations included in this survey used 15/32 in (11.906 mm) thick OSB panels and 8d box nails. The results indicate an approximately normal distribution of overdriven nails with a mean value of 0.08 in (2.032 mm) and standard deviation of 0.08 in (2.032 mm). See Figure 1. Statistical analysis of this data indicated a significant deviation of the population mean from being flush ($t = 27.5, \text{df} = 839, P = 0$).

Significance was proven using a matched pairs $t$ procedure. This procedure calculates the chances of having the sample mean deviate from the population mean. The standard deviation for the population is estimated by adjusting the sample deviation with respect to the number of readings in the sample. The value of $t$ then corresponds to the number of population deviations from the assumed mean of the population. The $P$ value is the percent chance that the sample mean deviates from the population mean due to random selection. In this case the mean of our sample, 0.08 in, was compared to the population of nails as a whole with a theoretical mean of 0.0 in. The corresponding $t$ value was 27.5. This alone shows significance, since $t$ values don’t reach much higher than 3.5 before the $P$ values become infinitesimal. For example, a $t$ value of 3.3 in a sample with 1000 readings would indicate a 0.05% chance that the deviance was natural.
The results of the survey show that overdriving nails is common practice. Of the nails surveyed, an incredible 83 percent were overdriven and 29 percent were overdriven more than 1/8 in (3.175 mm). The findings of this survey are consistent with surveys conducted in other regions. Investigations of many distressed wood frame multi-family buildings in the San Francisco Bay area revealed that about 80 percent of the nails in 3/8 in (9.525 mm) thick panel shear walls were overdriven, with many nail heads 1/8 in (3.175 mm) or more below the plywood surface (Zacher and Gray, 1989).
Conclusions

The research conducted to date on shear walls with overdriven nails is incongruous. In review, Zacher and Gray (1989) concluded that performance of plywood shear walls with overdriven nails was not satisfactory. Andreason and Tissell (1994) concluded that overdriving nails will not significantly affect the strength, but a strength reduction should be taken when 20 - 25 percent of the nails are overdriven. Ficcadenti et al. (1996) concluded that overdriven box nails would not decrease the strength of the shearwall below the level of properly driven common nails. Although examination of the data shows that flush-driven box nails (the weakest combination) reached lower levels of load and ductility than the common nails. Indeed, overdriving may increase the strength and ductility of shear walls when compared to flush–driven common nail shear walls.

Clearly, due to the many variables affecting the behavior of shear walls with overdriven fasteners, the investigations conducted so far are simply not sufficient. As with any property of timber, repetition in testing is crucial for getting reliable results from the variable nature wood. Extensive coupon testing is a reasonable way to start understanding the nature of the connection. This also affords the investigation of larger numbers of variables in sheathing material, sheathing thicknesses, fastener sizes, fastener types, and levels of penetration. Once enough coupon tests have been conducted, theories can be tested on full size panels for validation. There is also hope of using coupon tests to fully predict the behavior of full-sized shearwalls.

Coupon testing is an inexpensive alternative to full–scale shear wall testing, since the latter is costly and time consuming. Zacher and Gray (1989) and Andreason and Tissell (1994) conducted a limited number of overdriven wood joint tests. The scope of the testing was limited in number and variation due to the cumbersome nature of full size samples. Thus, there are many combinations of panel thickness, fastener type and size, and fastener head penetration that need to be investigated. McLain (1997) and Dolan (1999) have also suggested that a full investigation of construction tolerances of wood joints, such as those built with pneumatically driven fasteners is needed.

The results of the survey underline the need to investigate the effect of fastener head penetration. If overdriving nails proves to be a source of significant degradation, it is imperative to rectify the situation through adoption of more stringent inspection guidelines. It may also prove necessary to re-analyze built structures accounting for the loss of strength.

The findings of this research are anticipated to be of great value to the wood construction industry, providing conclusive results of the effects of fastener head penetration on sheathing–to–wood connections.
References


Task 1.4.8.2 - Inter-Story Shear Transfer in Woodframe Buildings

Literature Review

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Summary

This report provides a literature review for Task 1.4.8.2 – Inter-Story Shear Transfer in Woodframe Buildings of the CURee Caltech Woodframe Project. The literature review addresses connection behavior at the story level for residential and commercial woodframe buildings and other issues associated with this topic. The reviewed literature demonstrates the need for testing in this area to gain a practical understanding of the load transfer occurring in this system of building construction.
Introduction

Light-frame wood construction has generally been regarded as performing well when subject to lateral loading (Hamburger and McCormick, 1997, p. 77). The Northridge Earthquake demonstrated that current construction methods are relatively effective in preserving lives of the inhabitants of this construction but not as capable of preventing non-life threatening property damage. In some cases buildings collapsed (Goetz and Dimitry, 1999, p. 3). In many cases, seismic safety items were missing or installed improperly.

Refer to the work of Goetz and Dimitry (1999, p. 3):

…one third of seismic safety items were missing or flawed in over 40 percent of surveyed structures… These include shearwall hold-downs, nailing and proportion (aspect ratio), wall-to-wall straps and tie-downs, diaphragm blocking and nailing, drag strut splices, and roof-to-wall anchors.

Whether or not these safety items were properly designed and installed does not address the problems with buildings that were constructed properly and still suffered damage. One area of construction that has seen little engineering consideration is the mechanism that transfers shear between stories.

Inter-Story Shear Transfer in Woodframe Buildings

Shear transfer between story levels is an area in wood construction that has historically been a prescriptive design technique (conventional construction) rather than an engineered one. That is, under certain prescribed conditions the nailing schedule and blocking requirements, for example, are specified based on past performance rather than engineering theory. As a result, the current trend toward engineered design) in low-rise structures needs to be addressed if woodframe buildings are to resist structural damage in earthquake loads such as occurred at Northridge.

Cobeen discusses this issue below (1997, pp. 25-26):

The first trend in structure design to be addressed is the changing level of detail provided in engineered wood light-frame structures. The second… is the range of seismic design methodologies in use.

Designs need to include more than the selection of framing member sizes from published tables. The recent trend is away from conventional construction and towards engineered design (Cobeen, 1997, p. 26). This includes a consideration of how these structures are to resist earthquake loads. Often considered a problem area, shear transfer between stories is one area where detailing and earthquake design methodologies will have a significant impact.

Current design generally consists of prescribed nailing schedules and blocking requirements that are found in the model building codes (e.g. Uniform Building Code (UBC), Standard Building
code (SBC), and the National Building Code (NBC)) sections for wood construction. All of the codes deal with wood construction in basically the same manner: prescriptive design backed up by calculations when necessary. The UBC will be the only code addressed herein since the UBC contains the most advanced seismic loading provisions among the three model building codes.

The UBC (1997) deals with this design issue by providing a table of nailing schedules for the various members in conventional construction. The table is condensed below to show just the member nailing requirements for the shear transfer and applicable studwall elements (shown in Table 1).

Table 1
Nailing Schedule (summarized from Table 23-II-B-1, 1997 UBC, p. 2-282)

<table>
<thead>
<tr>
<th>Connection</th>
<th>Nailing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Joist to sill or girder, toenail</td>
<td>3-8d</td>
</tr>
<tr>
<td>2. Sole plate to joist or blocking, typical face nail</td>
<td>16d at 16” (406 mm) o.c.</td>
</tr>
<tr>
<td>Sole plate to joist or blocking, at braced wall panels</td>
<td>3-16d per 16” (406 mm)</td>
</tr>
<tr>
<td>3. Top plate to stud, end nail</td>
<td>2-16d</td>
</tr>
<tr>
<td>4. Stud to sole plate</td>
<td>4-8d, toenail or 2-16d, end nail</td>
</tr>
<tr>
<td>5. Doubled top plates, typical face nail</td>
<td>16d at 16” (406 mm) o.c.</td>
</tr>
<tr>
<td>Double top plates, lap splice</td>
<td>8-16d</td>
</tr>
<tr>
<td>6. Blocking between joists or rafters to top plate, toenail</td>
<td>3-8d</td>
</tr>
<tr>
<td>7. Rim joist to top plate, toenail</td>
<td>8d at 6” (152 mm) o.c.</td>
</tr>
<tr>
<td>8. Top plates, laps and intersections, face nail</td>
<td>2-16d</td>
</tr>
</tbody>
</table>

1 Common or box nails may be used except where otherwise stated

The UBC (1997) does not provide any guidance for engineering calculations for the connections at this intersection of shearwall and diaphragm. Any connection design is left to the designer. The engineer is responsible for determining the load path and designing the connections that result using either Allowable Stress Design (ASD) provisions or Load and Resistance Factor Design (LRFD) provisions.

Figure 1 shows a cross-section of a typical wall, framed with solid wood and nailed according to UBC (1997) Table 23-II-B-1 requirements. The nailing schedule changes when engineered wood products are used instead of the solid wood components. The UBC (1997) does not address this issue but, the APA Engineered Wood Association has produced a Data File (1999) that provides detailing when APA I-joists and APA Performance Rated Rim Board are used. This Data File also contains various details to encompass different element configurations (APA I-joist rim board vs. APA Rim Board, etc.). Also included in this document is a design example to illustrate the use of APA rated products.

The Wood Frame Construction Manual (WFCM) (1995) also provides a table of nailing schedules similar to that in the UBC. In addition to a nailing schedule table, the WFCM also provides some design equations in the Commentary. This enables designers to detail joints that are not specifically addressed in the table.
Research on Inter-story Shear Transfer

The research conducted in this area of wood construction is limited (Karalic 1997, Hamburger et al 1997). Shear transfer has not been the focus of any research save one, the work done by Karalic (1997), a Structural Engineer for Matrix Timber Ltd. His work involves reinforcing floors and walls with bridging and or truss-like reinforcement.

Refer to Karalic’s work (1997, p. 121):

The system is comprised of industrially manufactured truss-like reinforcing component elements designed to fit into the currently used walls and wall-to-floor interface… These elements are built-up to form a reinforced shearwall of any width with the continuity through the wall to floor interface.
The results of these tests are quite convincing and have led to changes in the span tables approved by the National Research Council of Canada. The change in the span tables was a result of this new system providing effective means for floor stiffening and consequently for improving floor response. The basic idea is to match the stiffness and strength properties at the floor to wall interface. This allows for a more uniform and predictable load path, one that may be easier to design. An example of this kind of construction is provided in Figure 2.

Figure 2 - Wall bracing system
Summary

In light of the literature reviewed herein it is clear that the mechanism of shear transfer between stories must be studied in greater detail. The research that will be conducted in Task 1.4.8.2 will involve isolating and evaluating the inter-story shear transfer mechanism. This will be accomplished by testing various methods of construction commonly used in residential construction. A testing matrix was initially proposed for this task and a final matrix is being completed.
References


