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Publisher's version / Version de l'éditeur:
https://doi.org/10.4224/21268917
Research Report (National Research Council of Canada. Construction), 2013-09-24

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Seismic Performance of Wood Mid-Rise Structures

Research Report No. RR-345

Date of Issue: 24 September, 2013

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NRC CONSTRUCTION PORTFOLIO

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by

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ABSTRACT

This report provides results of a state-of-the-art literature review of studies and surveys on seismic performance of wood structures. The review was performed to develop an understanding of the seismic response and design of mid-rise/multi-storey wood buildings and to explore gaps and challenges in the seismic design of these structures. This report includes summaries from results of the previous experimental, numerical and analytical studies as well as post-earthquake surveys on seismic response/resistance of wood buildings. The main structural systems considered in this study were wood light frames, commonly used in North America. The results of this review showed the dynamic response characteristics of multi-storey wood buildings, e.g. effects of higher modes, would play a major role in the seismic design of such structures. However, a proper design of timber structures would mitigate their damage and failure in the event of earthquakes.

Contents

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1 INTRODUCTION

Recently, interest in design and construction of mid-rise wood structures has increased worldwide. Many studies have been carried out to develop new mid-rise wood-based structural systems with corresponding design and construction tools.

Wood structures are widely used in single- and double-family dwellings, low-rise commercial and multi-storey residential buildings. The most prominent wood-based structural system used in North America is light wood frame construction. The main lateral load resisting elements in this type of construction are the wood-frame shearwalls. A typical light-frame shearwall is composed of nominal 2x4 or 2x6 framing studs spaced either 400 mm or 610 mm on centre sheathed with plywood or oriented strand board panels. The framing which in most cases is blocked forms a stand to which sheathing panels are attached using nails or screws. In addition, hold-down devices are used to resist overturning forces, and provide a continuous load path along the building height.

Properly designed light wood-frame structures have generally performed well during past earthquake events. This is mainly attributed to high strength-to-weight ratio of the timber as a material, ductility and redundancy of the overall system. However, structural integrity of wood-frame constructions is not necessarily guaranteed under the action of natural hazards, such as earthquake and hurricane events, especially in multi-storey constructions (Lam et al. 2002). Structural and non-structural damage observations following, for example, the 1994 Northridge and 1995 Kobe earthquakes pointed out inadequacies related to design and construction practice of light frame structures. This prompted engineers to carry out a great deal of experimental effort for the quantification of static and dynamic characteristics of timber constructions. A primary motive was the development of analytical tools to predict seismic behaviour of light frame structures. Yet, nonlinear performance of wood assemblies is complex and difficult to model, and there is a need to develop more comprehensive tools for the assessment of nonlinear static and dynamic properties of wood structures.

With the expansion of wood-frame structures to 6 storeys as is the case in British Columbia and as is proposed by 2015 in the rest of the country, a better understanding of the seismic response of such structures is needed.

The main objective of this review is to develop a summary of the main parameters that affect the seismic performance of wood-frame structures including different failure mechanisms.

2 LESSONS LEARNED FROM PAST EARTHQUAKES

The 1994 Northridge earthquake caused heavy economic losses in the metropolitan region of Los Angeles due to inadequate quality control and poor construction practice of light frame structures. The event demonstrated that the then design and construction practice of wood constructions were relatively effective in mitigating life losses but less effective in controlling property losses (Schierle 1996). While causing a total of 58 fatalities (most of them attributed to a couple of poorly designed buildings in the Marina District in San Francisco), records indicate that there were approximately 48,000 uninhabitable houses and property losses of $40 billion, of which 50% was attributed to damage of wood construction (Goetz and Dimitry 1999).

Damage observations after earthquakes have identified different parameters that contribute to the seismic vulnerability of the wood structures. These parameters include (Schierle 1996): missing hold-down to anchor walls to the foundation for resistance of overturning moments; inadequate shear-wall anchor bolts to the foundation to secure lateral slippage; lack of wall-to-wall tie-down devices to tie upper shear walls with the walls below to resist overturning forces; inadequate floor-to-wall anchorage for proper shear load transfer; need for blocking devices in housing constructions to establish an adequate load path for shear transfer from floor joists to adjacent panels; improper nail spacing due to poor construction practice, and high shear-wall proportion (aspect ratio), and poor material and construction quality control. It was also taught that non-structural wall-board material, such as stucco and gypsum, exhibits poor cyclic performance. This is in addition to buildings with integrated parking that are susceptible to catastrophic collapse during intense shaking, as a consequence of torsional demand.

FEMA 547 (2006) was prepared based on post-earthquake observations and studies, to provide information on weaknesses related to construction and rehabilitation practice of wood residential structures. Major deficiencies indicated in FEMA 547 (2006) were: use of weak and brittle materials as sheathing panels, such as stucco and gypsum wall board; inadequate load path due to weak connections; inadequate shear wall overturning strength; presence of unbraced crippled walls; limited shear capacity of diaphragms; poorly attached masonry veneers; uncertainties related to seismic performance of mid-rise construction (four to 6 storeys); questions regarding functionality of hold-down devices and narrow walls in mid-rise construction; landslide and torsional potential of hillside houses due to inadequate anchorage to the foundation.

Destruction and economic losses associated with past events (i.e. 1994 Northridge earthquake) might have been prevented if proper construction quality control was followed (Wong 1990). Mann (1984) stated that minor investment in construction quality control could substantially reduce large repair and replacement costs.

A review study by NRC (Rainer and Karacabeyli, 2000) on casualties in past earthquakes that involved wood houses and seismic performance of platform-frame wood houses revealed that the overall performance of wood frame houses concluded that when reasonable care is taken in design and construction, platform-frame wood housing performs well in earthquakes from the human-safety perspective and in some cases from a damage-prevention point of view.

3 STUDIES ON SEISMIC RESPONSE OF WOOD STRUCTURES

Results from large numbers of experimental and analytical studies have contributed to a better understanding of the structural behaviour of light frame structures under different loading scenarios, including the seismic load.

Some of the major research programs on seismic performance of timber structures are summarized here.

Polensek and Schimel (1991) conducted tests on different types of light frame wood subsystems, including shear walls. The researchers reported cyclic tests data as a benchmark for analyzing and designing wall assemblies under seismic and wind loads. Deam et al. (1991) executed cyclic tests on a full-scale 3-storey shear wall that was designed according to the then available New Zealand code. The results indicated satisfactory performance of the facility during earthquake. Touliatos et al. (1991) performed experimental tests on a 2-storey wood structure for assessment purpose of the then available Canadian code standards.

Nakajima et al. (1993) discussed a system level performance of a full-scale 3-storey building representative of Japanese conventional houses. Leiva-Arevena (1996) investigated the damping ratio of shear walls using equivalent viscous damping coefficient. The study indicated that shear walls have considerable inherent energy absorption and damping capacity. Rose (1998) provided test data of full scale shear wall specimens to improve understanding of influencing factors on the performance of wood shear walls under cyclic loading, and state standardized protocol for cyclic tests of shear wall assemblies. Dolan (1989), Tissell (1993), and Durham et al. (2001) performed a series of experimental programs on full scale walls to improve information for better understanding of seismic performance and design practice of wood shear walls.

Filiatrault et al. (2000) conducted an experimental program, as part of CUREE-Caltech (Consortium of Universities for Research in Earthquake Engineering–California Institute of Technology) wood frame project, on a box type one-storey, 2-storey, and 3-storey shear walls with different configurations and finishing materials. The purpose was to treat the need for more test data on the seismic behaviour of full-scale wood frame structures; in order to improve the state-of-art and state-of-practice of design and analysis of wood structure, enhance construction technology for targeted performance levels, and education.

Miyazawa (2004) reported a series of shake table tests on six full-scale 2-storey buildings tested from 1995 to 2003. The researchers suggested that ordinary wood houses are capable of resisting strong seismic excitations (e.g. 1995 Kobe Earthquake). They also indicated that it is possible to construct wood buildings that can resist earthquakes stronger than the Kobe earthquake.

Skaggs and Martin 2004 reported test data of 59 light frame walls as part of the Engineering Wood Association (APA) program, with different aspect ratios. The purpose was to verify suitability of available methodologies to compute deformation of shear walls and diaphragms.

Koshihara et al. (2004) and Miyake et al. (2004) conducted series of shake table tests on a 2-storey Japanese conventional building to investigate the deformation response and safety of the building when subjected to combined ground motions. They illustrated that combined ground motions induce larger deformation on buildings in comparison to the effect of one-directional ground motion. It was also suggested that the initial stiffness of buildings, connections of diagonal braces, and bending strength of shear walls impact the collapse mechanism of the structural system during seismic excitation.

Sakamoto et al. (2001) conducted a project in Japan, to develop high performance hybrid timber members, and structural systems; to develop performance assessment measures, and advance design methods for hybrid timber facilities. Lam et al. (2002) performed shake table and monotonic tests to improve fundamental understanding of static and dynamic behaviour of two- and three-dimensional frame and panel assemblies subjected to earthquake and wind loadings. The tests were performed at the University of British Columbia between August 1997 and July 2001. The total budget for the project was approximately $1.2 million, funded by Forest Renewal British Columbia. The study consisted of different stages including, development and validation of analytical models and computer software for assessment of wood-frame structures, as well as the development of framework for reliable studies and design of wood-frame systems under earthquake.

Pryor et al. (2000) and Ventura et al. (2002) performed series of unidirectional shake table tests, at the University of British Columbia, on one-storey and 2-storey buildings. The lateral resisting systems consisted either of conventional shear walls, or narrow Simpson Strong-Walls. The buildings were designed according to the 1997 Uniform Building Code standards. The purpose of the study was to improve the design and construction practice of wood-frame buildings. The project also examined the contribution of exterior stucco in the resistance of wood structures, documented various damage levels of the buildings, and reported significant damages of non-engineered walls within the buildings.

Pardoen (2000) performed a large experimental program at the University of California, Irvine, to test full-size timber shear walls with different configurations under reversed cyclic loadings. The project was funded by the Federal Emergency Management Agency (FEMA) and the city of Los Angeles for the purpose of examining current design standards for timber structures in light of the damages sustained by wood construction during the 1994 Northridge earthquake. The objective of the study was to provide benchmark test data of full-scale wood frame structures.

Pardoen et al. (2003) tested 52 one-storey shear walls and four 2-storey shear walls to characterize the engineering parameters and performance level of shear wall assemblies, as well as, the contribution of non-structural finishing materials. Van de Lindt (2004) summarized 31 experimental tests, from 1983 to 2001, on wood shear walls. The purpose of the tests was to analyze the structural characteristics of light frame shear walls with different sheathing types, connections, and loading protocol. Most of these walls were fabricated of plywood/OSB panels, nailed fasteners, and 38 x 89 mm studs.

Varoglu et al. (2006) introduced a new concept in the design practice of shear wall assemblies. The concept involved re-arrangement of shear wall components to improve load-displacement characteristics of wall assemblies with respect to the conventional configuration of standard shear walls. Yasumura et al. (2006) performed pseudo-dynamic tests on one- and two- storey post and beam (P&B) walls with plywood panels in order to provide test data for the assessment of building model.

Christovasilis et al. (2007) conducted tests, within the NEESWood Project, on a full-scale 2-storey wood frame structure with an integrated two-car garage using two three-dimensional shake tables at the University of Buffalo at the Network for Earthquake Engineering Simulation (NEES) site. The structure was designed according to the 1988 Uniform Building Code to represent wood construction of the 1990's in Southern California. The purpose of the study was to provide a series of test data as a benchmark for the performance of wood frame, and for the development of analytical models for nonlinear seismic analysis of timber structures. The need for efficient analytical tools was to provide the base for the development of performance based seismic design philosophy for midrise wood construction. The design philosophy is intended to provide the necessary mechanisms to safely increase the height of wood construction in seismically active areas, as well as to mitigate potential damages to low-rise wood construction. The experimental results indicated that side sway mechanisms of the first level would dominate the collapse mechanism of the structure.

Filiatrault et al. (2010), as part of NEESWood project, performed triaxial shake table tests, at the State University of New York in Buffalo, on a 2-storey town house with an integrated two-car garage. The building was designed according to the 1988 Uniform Building Code to benchmark the existing building in California. The test illustrated that the structure performed well in terms of securing life safety, but incurred costly damages.

Van de Lindt et al. (2010) executed shake table tests on a full-scale 6-storey wood frame building, at Japan's E-Defence shake table, to examine a proposed performance based design philosophy, improve understanding of the seismic response of mid-rise light wood frame structures, and provide landmark test data for the earthquake engineering research community. The building consisted of 23 units and a living space of 1337 m^2. The building was designed according to a performance based design criteria developed as part of the NEESWood program to meet predefined inter-storey drifts for stated performance levels. The building configuration differed from the typical low rise wood structure in several aspects, including: a continuous tie-down system with a combination of steeltie-down rods and mechanical anchorage at the end of all shear walls to prevent overturning and reduce uplifts; intensive compression studs were installed in lower storeys and shear transfer details from wall to floor systems. Gypsum wall boards were installed on all ceilings and walls. The structure was subjected to three earthquakes ranging from 72 to 2500 year event for Los Angeles. The results indicated the following:

(1) Building performed well with little damage, even when it was subjected to 2500 year event. (2) Design of light wood frame by performance based seismic design approach performed better than force based designed buildings. It was reported that the acceleration forces were reasonable at upper storeys, and acceleration-induced non-structural damages could be prevented by suitable precautions. The structure also sustained a global drift of 0.25 m and the maximum inter-storey drift ratio of 2%. (3) The structure experienced torsional demand, despite the structure being symmetric and the seismic mass was evenly distributed. As so, the researchers recommended the need to include torsion design in the performance based seismic design provisions for wood frame structures.

Mosalam and Mahin 2007 reported experimental results of a large-scale 3-storey wood frame residential building with a tuck-under parking, subjected to multi-directional motions, to improve the understanding of seismic performance of multi-storey wood-structures and for the development of an analytical model. The building configuration was asymmetric and discontinuous in elevation with a tendency to twist around its vertical axes and form a soft storey mechanism. The purpose of the test was to examine the effectiveness of traditional seismic patterns to enhance lateral resistance, and reduce torsional demand of a 3-storey wood frame residential building with a tuck-under parking. The retrofits consisted of using a welded moment resisting steel frame around the garage opening, and strengthening the diaphragm to header beam connection. The findings indicated that the selected retrofits were effective means to reduce storey drift and control rotation of the building around its vertical axis. The results also revealed sensitivity of the structure to multi-directional motion. The impact of the multi-direction motion was exacerbated by the observed asymmetric damage patterns, indicating complex response of the structure.

Varoglu et al. (2006) introduced a new concept in the design of shear wall assemblies. The proposed configuration consisted of installing a plywood layer at the center between a series of pairs of studs oriented 90 degrees with respect to those in standard design configuration. The new wall design was, thus, referred to as a “midply” shear wall. The function of the mid-ply layer was to allow the nail connecting framing members to sheathing panels to exert double shear action, thus enhancing the performance of the shear wall. The stud members were rotated to increase the lateral strength capacity of the wall, and fastened sheathing panels to the wide face of the studs. This provided an edge distance for fasteners on the perimeter of sheathing panels, thus reducing the likelihood of nail tear out failure. Experimental tests were performed to illustrate and compare the performance of the new design wall relative to the standard design. The results highlighted that the load-deformation characteristics of the midply shear wall design exceeded those of standard shear wall configurations.

The following subsections discuss various experimental studies that examined different aspects related to the behaviour of wood assemblies. These aspects include: failure mechanisms of shear wall assemblies; fundamental period of timber assemblies; the influence of opening on the load-bearing capacity of timber structures; improve the understanding of the diaphragm role within timber assemblies; use of innovative construction techniques; investigate the effectiveness of utilizing hold-down devices; establish the benchmark database for the development of analytical models; assess the available design methodologies; and compare the effect of different loading protocols on the response and failure pattern of shear walls. More detail and information for some of these aspects is provided in the following sections.

3.1 Failure mechanisms of wood frame connections

Several research reports have indicated that the failure of the fasteners or the nails, e.g. pull through, withdrawal or fatigue, is the main failure mechanism of the wood-frame connections under lateral loads.

Stewart et al. (1988) examined the performance of shear walls under reversed-cyclic static loadings and shake table tests. The researchers reported that the failure mechanism of the connections was represented by the breaking off of sheathing nail heads, and nail withdrawal from the frame. Lam et al. (1997) studied the failure pattern of full-scale shear wall assemblies under monotonic and cyclic tests. The study outlined that the failure mode of the connections was characterized by nail withdrawal under monotonic test, and nail fatigue under cyclic test.

The American Plywood Association (Rose 1998) discussed the failure mode of full-scale shear wall connections under cyclic tests. The study noted that the failure mode of the connections was characterized by the fasteners fatigue under cyclic tests. The fatigue started near the corner of the panel after the maximum shear capacity was reached, and progressed along the edges.

Nelson et al. (1985) reported a failure pattern of seven shear walls subjected to simulated wind loads typically used in manufactured housing. The study revealed that the location of failure was at the connection of shear wall to the floor.

Dinehart and Shenton (1998) discussed damage patterns of full-scale shear walls sheathed with either plywood or OSB under static and cyclic tests. Under static tests, the failure pattern of shear walls was characterized by the sheathing pulling away from the frame, pulling the nail along with it and the splitting of the wall bottom plate at the uplift corner, where tension forces were concentrated. The split up took place along the sheathing nails in the bottom plate, and was induced by the sheathing pulling up and away from bottom plate. Also, it was observed that some interior studs twisted along their length and split at the bottom or top along the sheathing-to-frame nail connections. Under cyclic tests, most of the damage was restricted to the sheathing fasteners. Nails experienced fatigue and broke off. The nail damage was concentrated along the sheathing edge, top corner, and interior edges. There was little or no damage to the interior nails. In addition, conversely to static tests, the sheathing panels did not pull away from the frame.

He et al. (1998) illustrated the failure mechanism of timber shear walls subjected to cyclic loading. He et al. (1999) executed monotonic and cyclic tests to illustrate the influence of the opening on the failure mechanism of shear walls connection with standard and oversized OSB panels. Wall specimens with no openings experienced most of the deformation in the nails along the panel edges, whereas wall specimens with an opening exhibited a combination of nail and panel failures. The nail failure was in the form of nail withdrawal. Panel failures were demonstrated by panel crushing and buckling, and tearing around the perimeter of the opening. Lam et al. (2002) reported the failure mode of box-type wood structures with large openings on one side. The researchers indicated that the failure pattern was represented by nail pull through and nail withdrawal.

3.2 Natural period of timber buildings

Although the natural period of wood-frame buildings has been experimentally investigated by various researchers, most of the studies were on low-rise structures. Since wood-frame buildings are relatively light, finishing materials and exterior cladding can greatly contribute to increasing the natural frequency of the buildings.

Yokel et al. (1973) executed a dynamic test to measure the natural frequency of a full-scale 2-storey wood structure representative of residential houses in the United States. The structure was 14.3 m long, 7.9 m wide and 5.2 m high. The fundamental period of the structure was found to be 9 Hz. However, due to limits in the recording equipment, the dynamic test was inconclusive. Falk and Itani (1987) carried out free-vibration tests to determine the natural frequencies and damping ratios of wall specimens. Yasumura et al. (1988) reported the fundamental period of a 3-storey wood structure when subjected to reversed cyclic lateral loading. The results illustrated that the natural frequency of the specimen was reduced from 5.8 Hz to 3.1 Hz after static loading because of partial damage in the wall bearings and lessening of the joints.

Seo et al. (1999) performed a shake table test on two single-storey one-quarter scale wood frame models. The purpose of the study was to measure the natural frequency and damping. The first model was subjected to the 1985 Nahanni Earthquake, and the second model was subjected to the 1979 Imperial Valley Earthquake. The results suggested a 1.66 Hz natural frequency for the full-scale prototype.

Fischer et al. (2001) carried out a shake table test to study the influence of non-structural finishing material on the fundamental period of a 2-storey single family wood house. The wood frame house was tested in ten different phases. In each phase, different structural configurations of the house were tested. The variation in structural configurations between the phases ranged from sheathed shear walls, symmetrical and unsymmetrical openings, and presence of non-structural finishing materials. The results indicated that the natural frequency of the structure varied between 3.96 to 6.49 Hz due to the use of finishing material.

Kharrazi (2001) conducted a series of shake table tests at the University of British Columbia to quantify vibration characteristics of a full scale, 2-storey single-family house. The experimental tests were an integral phase of the “Earthquake 99 Wood Frame

House Project" (EQ 99), a collaborative university-industry research initiative that was initiated in late 1999 and completed in 2001. Principal objectives of the EQ 99 project were to investigate seismic performance of existing housing stock in British Columbia (BC) and California; and develop improved design and retrofit procedures to substantially reduce earthquake-induced damages in residential wood frame constructions. The tests were divided into two main categories: engineered and non-engineered systems. Engineered systems included wood-frame structures designed according to the 1997 Uniform Building Code (ICBO 1997) with and without OSB sheathing. Non-engineered systems included structures not designed according to seismic design provisions. The tested specimens were subjected to ambient vibration testing and sinusoidal sweep testing before and after subjecting the specimen to real earthquake motion for the purpose of documenting the stiffness degradation due an earthquake. More detailed information concerning the shake table testing can be found in Kharazzi (2001). Natural frequencies obtained from the tests were reported to be in the range of 1.66 to 3.33 Hz with damping ratios between 3.6 and 16 percent.

Van de Lindt et al. (2010) executed shake table tests on a full-scale 6-storey wood frame building at Japan's E-Defence shake table, to examine a proposed performance based design philosophy, to improve the understanding of seismic response of midrise light wood frame structure, and to provide landmark test data for the earthquake engineering research community. The structure was subjected to three levels of earthquakes from 72 to 2500 years return periods for locations such as Los Angeles, CA. The natural period of structure measured by a white noise excitation was approximately 0.41s

Table 1 compares the natural period of a few tested timber buildings which was determined, according to the NBCC's formula, for the seismic design of structures in Canada. The table indicates that the natural periods calculated using the formulas for buildings with shear walls and others provide a close estimation for some of the tested specimens or field houses. However, this table provides only a few experimental results with different structural systems. Therefore, more research and experimental data is required to achieve a reliable conclusion for calculating a natural period of timber buildings.

Table 1. Natural period of wood structures compared to the NBC's formula.

Authors	Structure system	Height (m)	Measured Natural Period (seconds)	NBC's simplified formula for Natural Period (seconds) (NBCC 2010)		
				other moment frame	braced frame	shear wall and others
Yokel et al. (1973)	Conventional wood-frame 2-storey house with gypsum panels (US)	5.2	0.11	0.20	0.09	0.17
Falk and Itani (1987)	A 3-storey timber house braced with nailed shear walls	10*	0.17 (damaged =0.32)	0.30	0.14	0.28
Fischer et al. (2001)	2-storey Single-Family House with OSB sheathing	6*	0.25~0.15**	0.20	0.10	0.19
Van de Lindt et al. (2010)	6-storey light-frame wood building with shear wall	17	0.41	0.60	0.21	0.42
Kharrazi (2001)	2-Storey timer house	4.8	0.30	0.20	0.08	0.16
Kharrazi (2001)	4-storey timer house (field test)	12.72	0.50	0.40	0.17	0.34
Kharrazi (2001)	4-storey timer house (field test)	11.90	0.39	0.40	0.16	0.32
Kharrazi (2001)	4-storey timer house (field test)	11.97	0.39	0.40	0.16	0.32

*Approximate height, **with and without non-structural finishing.

3.3 Effects of openings

In buildings where shear walls and non-structural walls contribute to the seismic resistance of the structures, openings in the walls could affect the overall seismic response of these buildings. Most of the studies indicated reduction of strength and stiffness of the wall with the openings. Furthermore, there is a tendency of torque (torsion) of timber structures with openings in some of the walls which results in an asymmetric structure.

Hayashi (1988) examined the effect of the wall opening ratio on the seismic response of conventional P&B shear walls. The results indicated that the strength and stiffness of the wall decreases as the ratio of wall opening increases. He et al. (1999) presented cyclic and monotonic tests concerning the influence of openings on the strength and stiffness of full-scale shear walls with standard and oversized OSB panels. The results illustrated that the presence of openings significantly decreased the strength and stiffness of the walls due to the reduction in the effective sheathing area, as well as the asymmetric failure pattern of the walls.

Yasumura et al. (1988) performed cyclic tests to investigate the influence of window and door openings on the seismic response of a 3-storey wood structure. The tested specimens had window and door openings on one side, and window openings on the other side. Monotonic, cyclic, and force vibration tests were performed. A rotating mass generator was placed on top of the third storey and electromagnetic pickups were used to record the dynamic behaviour. The results illustrated that a discrepancy was observed in the shear resistance of north and south longitudinal walls attributed to the presence of the openings, inducing a tendency for torsional deformation.

Mosalam et al. (2003) tested the influence of the opening, as part of the CUREE-Caltech Wood frame project, on the seismic performance of a 3-storey apartment building with a tuck under garage. The findings indicated that a building with a tuck under garage is susceptible to torsional demand, and subsequent soft storey collapse mechanism. The tendency of torsional response of wood structures with openings was also reported by Lam et al (2002). In the latter study, a box-type wood structure with a large opening on one side was tested using monotonic and shake table tests. The shake table test was executed using unidirectional and bidirectional ground motion records from Joshua Tree Station 1992 Landers, California earthquake.

Patton-Mallory and Wolfe (1985) carried out a series of monotonic tests to address the influence of an opening on the rocking resistance of full-scale shear walls. The findings suggested that the door and window openings can be neglected in determining the rocking resistance of the shear walls.

3.4 Influence of non-structural finishing materials

Non-structural finishing systems have been reported to contribute to the seismic resistance of structures by increasing the overall lateral resistance, reducing the lateral deformation, increasing the torsional moment resistance and reducing the natural period of the structure.

Patton-Mallory and Wolfe (1985) studied the contribution of gypsum layers on the response of shear walls. The study concluded that a layer of gypsum board contributed to the response of the wall and added to the lateral resistance of the plywood.

Tanaka et al. (1998) highlighted the effect of gypsum and siding board sheathings on the dynamic behaviour of a full-scale 2-storey wood frame. The structure was tested using the 1940 El Centro earthquake, and 1995 Kobe earthquake. The findings indicated that the non-structural finishing material resisted a significant portion of the lateral loadings, added to the stiffness, and reduced the drift sustained by the structure.

Filiatrault et al. (2002a) conducted a shake table test, as part of CUREE Caltech Woodframe project, on a rectangular 2-storey wood building with an integrated one-car garage to study the contribution of finishing materials (i.e. gypsum wallboard and stucco). The structure was subjected to two ground motion records of the 1994 Northridge earthquake. The study revealed that the finishing materials increased the stiffness and strength, and dramatically reduced the seismic response of the building. Furthermore, it was illustrated that the presence of the finishing material influenced the distribution of anchor bolt forces.

Christovasilis et al. (2007) performed two three-dimensional shake table tests to address the contribution of gypsum and stucco boards on the performance of a full-scale 2-storey wood frame structure. The results illustrated that the installation of gypsum and stucco on interior walls substantially increased the strength and stiffness, reduced fundamental period, and reduced the displacement response of the structure. The findings also suggested that the use of gypsum boards increased the in-plane stiffness of roof diaphragms. On the other hand, it was found that the application of gypsum sheathing had minimal effect on floor systems. This was explained to be attributed to the rigid action of diaphragms, which was caused by the structural system of the floors. Further, it was observed that the use of stucco reduced inter-storey drift of both levels.

Mosalam and Mahin (2007) examined the contribution of finishing material (i.e. gypsum board) on the seismic response and damage susceptibility of a 3-storey residential building with tuck-under parking. The findings suggested that the implementation of non-structural materials significantly reduced the maximum storey drift, shear strain of the damaged shear wall, and increased the torsional moment resistance. Consequently, it was recommended that non-structural finishing material be considered during the performance assessment for accurate prediction of twist susceptibility and torsional moment demand associated with asymmetric buildings.

Filiatrault et al. (2010) studied the influence of non-structural materials on the performance of a 2-storey town house with an integrated two-car garage using shake tables at the State University of New York in Buffalo. The study reported that non-structural finishing materials (i.e. gypsum wall board and stucco) contributed to the stiffness and strength of the 2-storey town house tested using the shake table.

3.5 Effects of Diaphragm on the Structural Behaviour

In seismic design of buildings, the assumption of a rigid floor diaphragm simplifies the design process significantly. A diaphragm is considered rigid if it can distribute the lateral forces to the vertical lateral load resisting elements in proportion to their relative stiffness.

Yasumura et al. (1988) investigated the cyclic response of floor diaphragms of a 3-storey wood structure. It was found that the shear deformation in the floor diaphragms was small, and thus they were treated as rigid elements. Phillip et al. (1993) examined the load carrying capacity and behaviour of horizontal diaphragms. The researchers observed that the roof diaphragm behaved like a rigid body.

Yokel et al. (1973) tested the behaviour of shear walls and floor diaphragms of a full-scale 2-storey structure. It was observed that the roof diaphragm acted as a flexible diaphragm, whereas the second floor diaphragm behaved as a rigid body.

Aoki et al. (2000) conducted static and dynamic tests on thirty half-scaled buildings to evaluate the effect of different arrangements of shear walls and diaphragm flexibility. The study indicated that the symmetric arrangement of shear walls and high in-plane stiffness of diaphragms are essential to reduce torsional demand and limit shear deformation of walls.

3.6 Sheathing material

Experimental studies have been carried out to understand the effects of different types of sheathing materials in the seismic response of timber buildings. Most of these studies have concluded that the plywood and OSB sheathings would not significantly affect the overall lateral seismic response of the wood structure. However, in some cases, earlier failure of the fasteners using a certain type of sheathing, e.g. OSB, has been reported.

Dolan and Madsen (1992) reported that there is no significant difference attributed to using plywood or waferboard sheathings on the behaviour of shear walls. Serrette et al. (1997) indicated that the shear capacity of OSB and plywood sheathing is similar.

Rose (1998) performed a series of cyclic and monotonic tests on full-scale shear walls at the University of California, Irvine. The project examined the influence of using plywood and OSB sheathings on the load-displacement characteristics and load-carrying capacity of the walls. The study reported that shear strength of walls with OSB panels exhibited 15% less compared with walls sheathed with comparable thickness and grade of plywood panels. This was explained to be attributed to earlier fatigue failure of sheathing fasteners in the case of using OSB sheathing. The earlier fatigue of fasteners was due to connecting higher density panels, such as OSB layers, in comparison to plywood panels. Stefansescu (2000) found that OSB sheathed walls dissipated more energy than diagonally braced walls (e.g. inverted V bracings) when subjected to the same cyclic load protocol.

3.7 Sheathing-to-framing connection

Nails have been used mostly for connecting the sheathings to the frames in light-frame timber buildings. Experimental studies on the seismic response of such timber structures revealed that the nail characteristics and the sheathing-to-framing connections play a major role in the overall seismic performance of the structures. Increasing the nail spacing and length would substantially increase the seismic resistance of the shear wall and therefore the whole timber structure.

Atherton (1983) conducted a series of cyclic tests to examine the influence of the sheathing pattern, sheathing thickness, nail spacing, and nail size on the strength of a full-scale wood frame diaphragm sheathed with particleboards. The study revealed that the nail spacing poses a significant impact on the strength of the diaphragm.

Similarly, Cheung et al. (1988) tested the influence of nail spacing, mechanical properties, and joint stiffness on the response of shear walls. The sensitivity analyses were carried out using a previously developed analytical model for sheathed diaphragms (Itani and Cheung 1984). The researchers reported that connections between the sheathing and framing members dominated the overall behaviour of the diaphragms. The results also indicated that moduli of elasticity of sheathing and framing member had a negligible effect on the overall stiffness, while nail spacing had a significant effect.

Dolan and White (1992) outlined that using adhesives to attach sheathing materials to the framing member increased the stiffness and decreased the ductility of wall assemblies, as compared with standard shear walls.

Lam et al. (1997) investigated the influence of reinforcing the panel’s perimeter with more nails on the overall response of full-scale shear walls. The results demonstrated a significant increase in stiffness, strength, and ductility of shear walls which can be achieved with reinforcing the panel's perimeter with nails.

3.8 Hold-down devices

Application of hold-down devices in light-frame timber buildings have shown to improve their seismic performance by decreasing the structural uplift and rocking and by increasing their overturning resistance, wall strength and ductility.

Rose (1998) examined the combined effect of using hold-down connectors in conjunction with shear plates in shear wall assemblies to achieve shear load distribution from bolts into end posts with minimum slip, and reduce the eccentricity of bolts. The findings indicated that the use of hold-down devices reduces the displacement of shear walls, and the induced stress on sheathing-to-framing fasteners, especially in critical areas such as corners of sheathing panels

Schmid et al. (1994) illustrated that the uplift at the end of the wall is a significant contributor to the wall lateral displacement, so therefore a proper anchor is needed. It was also observed that vertical loads reduce uplift of the panel, and consequently reduces the lateral displacement caused by panel rotation.

Yasumura (2000) executed static and dynamic tests on 11 plywood walls to investigate the impact of opening, blocking and hold-down devices. The results indicated that blocking members played an important role in transferring a load in case of horizontally installed panels. The study also revealed a considerable decrease in the wall strength when hold-down devices were not installed.

Dolan and Madsen (1992) performed tests on 11 full-size nailed timber shear walls to address the influence of implementing hold-down devices. The study outlined that the application of dead load had no influence on the racking load-displacement characteristics of shear walls in cases where an anchorage was used to resist the overturning moment.

Shipp et al. (2000) compared the contribution of different types of hold down devices on the response of shear walls subjected to reversed-cyclic loadings. The study illustrated that the type of hold down devices precipitated a small effect on the lateral force carrying capacity of shear walls.

Bracci and Jones (1998) investigated the sill plate anchorage failure in response to damage observation during the 1994 Northridge earthquake. The researchers executed ten quasi-static reversed cyclic tests on specimens with several lumber sizes, bolt diameters, and sill plate confinements. Experimental results suggested that using a confining clamp prevents splitting of wood members. As so, sudden loss of strength due to splitting can be minimized, and, consequently, the member is allowed to absorb more energy before failure. The study also examined the effectiveness of using reinforcing clamps for bolted connections. The following conclusions were provided:

1) Utilizing reinforcing clamps in critical bolted connections increases ductility and reduces deflection in the confined connections;
2) It prevents brittle splitting of the plate along the line of anchor bolts in the foundation;
3) It restricts failure to sheathing-to-framing connections, and thus more ductile response is attained and the wood assembly can be easily repaired after earthquake; and
4) it enhances connection between wall end post and sill plate, thus minimizing uplift.

In cases with larger lumber sizes (i.e. 3×4), the confining clamp forced anchor bolts to exhibit double curvature deformation, causing an increase in the post-yield stiffness indicating enhanced ductility of the member.

3.9 Loading protocol

Loading protocol, e.g. static, cyclic, pseudo-dynamic or dynamic loads, have shown to affect seismic performance of the timber structures. Results of experimental studies indicate lower shear strength for shear walls under cyclic loads than that under monotonic loads. The following studies also point out that shear walls under monotonic/cyclic load protocols exhibit nail fatigue fracture under cyclic loads, due to the high demand imposed on the nails, whereas nail pullout and pull-through failure mechanisms were reported under monotonic loads.

Dolan and Madsen (1992) reported the influence of cyclic loading on the lateral load capacity of shear walls. A slow cyclic racking test was executed to develop hysteretic response of shear wall assemblies. The researchers explained that the reduction in the load carrying capacity of the walls during successive cycles was due to damage of the nails surrounding it. Further, the results showed an increase in the load intercept for the hysteretic response, which was suggested to be attributed to the inelastic deformation of the large number of nail connections. Rose (1998) examined the load carrying capacity of shear walls under monotonic and cyclic tests. The study revealed that the wall shear strength is lower under the cyclic test than that under the monotonic test. Schmid et al. (1994) observed a decrease in shear values for shear walls sheathed with plywood under cyclic loadings. Ficcadenti et al. (1998) conducted three different cyclic tests according to the ASTM E 564 loading protocol to investigate and compare the impact of each protocol on the performance of four identical shear walls sheathed with plywood. The study illustrated that loading cycles with excessive cycles in the elastic range reduced the ultimate strength of shear walls.

Shenton et al. (1998) conducted an experimental program on eight full-scale shear walls to characterize the degradation of stiffness and energy dissipation under cyclic loadings. The tests were conducted according to a sequential phased displacement proposed by the Structural Engineering Association of Southern California (SEAOSC 1997). The study suggested that, under cyclic loading, stiffness and energy dissipation capabilities of plywood and OSB wood shear walls are influenced more by fasteners and framing members. It was also observed that the effective stiffness of shear walls decreased linearly with continued cycling at the same displacement level.

Dolan and Heine (1998) compared the load carrying capacity and deformation capacity of 22 shear walls with various configurations under monotonic and reversed-cyclic displacement control loadings. It was indicated that monotonically loaded shear walls exhibited higher load resistance in comparison to walls tested under reversed-cyclic loading.

Lam et al. (2002) investigated the influence of cyclic loading on the behaviour of nail connections, and consequential implications on the dynamic response of full-scale shear walls. It is considered that several factors dominate the behaviour of nail connections under cyclic loadings such as, mechanical yielding of nailed fasteners, nonlinear bearing of wood elements (i.e. stud and panel), and gap formation between nails and the surrounding wood medium, causing the development of slack. These factors define the characteristics of hysteretic loops of nail connections subjected to cyclic loading. Traditionally, hysteretic curves of nail connections are developed by fitting test data for a given loading protocol. However, Lam et al. (2002) developed a database for hysteretic loops of nail connections under different loading protocols. The following describes the hysteretic behaviour of nails under cyclic loadings.

Under the application of cyclic loadings, the deformation nonlinearly increases with the increase in loading, causing a small gap to develop between the back of the nail and the frame. Depending on the load level, the interaction between the front of the nail and the wood member may induce permanent damage (i.e. deformation) in the wood medium. During the load reversal, the nails move through the gaps before contacting the wood again. This shall induce a force and enlarge the present gaps. Following several cycles, the gaps between the nails and the confining wood medium enlarges. This process causes formation of slack, which is represented by a pinch on the hysteretic loop.

Gatto and Uang (2003) executed 10 tests at the University of California, San Diego, to examine the influence of loading protocols on full-scale shear walls. The studied protocols were: monotonic; CUREE-Caltech standard (CUREE) (Krawinkler et al. 2001); CUREE-Caltech near-fault (NF) (Krawinkler et al. 2001); sequential phased displacement (SPD) (Dinehart and Shenton 1998); and International Standards Organization (ISO/TC 165, 1998). It was found that the loading sequence had a significant influence on the shear wall performance. It was also observed that failure in most tests initiated at nail levels, and that the failure patterns of nails were controlled by the applied loading protocol. Loading protocols with a large number of cycles (SPD) induced nail fatigue fracture due to the high demand imposed on the nails, whereas the failure mechanism of nails for monotonic, CUREE, ISO and NF loading protocols was characterized by nail pullout and pull-through. It was observed that nail fatigue fractures resulted in a 25% reduction in ultimate strength, and 47% in deformation capacity relative to the CUREE protocol. Further, it was recommended that the observed fatigue failure of nails under SPD protocols did not represent the demand imposed during seismic events, and thus SPD protocol shall not be used to assess seismic performance of wood shear wall assemblies. In addition, results of NF protocol indicate a more rapid degradation of strength at large displacement. This was attributed to the fact that the pattern of NF protocol induced strain hardening at nails that increased demand on the other elements, leading to a rapid degradation in strength. Finally, because the failure modes produced by the CUREE protocol was found to be the most consistent with seismic response, it was recommended to use the protocol for future testing of wood frame shear walls.

Yamaguchi et al. (2000) performed shake table tests to analyze and compare failure modes of full-scale shear walls under static and dynamic loadings. The dynamic loading was simulated using the ground motion record of the 1995 Kobe earthquake. The following observations were reported: (1) Envelope of the dynamic and static hysteretic loops do not match. (2) The maximum strength observed on dynamic hysteretic loops was 14% more than the maximum strength observed on static hysteretic loops. (3) The yield point on the dynamic hysteretic loops occurred just before reaching the maximum strength points of shear walls. (4) Nailed fasteners exhibited higher shear strength but less ductility under dynamic tests as compared with static tests. (5) Dynamic loading decreased friction of nails compared to static loading. As so, greater pull out of nails was generated in dynamic tests relative to static tests. (6) Unsymmetrical hysteretic loops were observed after the maximum strength point was reached. This was explained as when the response of the shear wall exceeds the dynamic yield point or peak strength point, nails pull out and never go back to their original position. (7) Pulling out of nails causes dynamic yield and controls the maximum strength of the wall.

Yamaguchi et al. (2000b) compared the force-displacement response of full-scale nailed plywood shear walls under static and dynamic testings. Six testing patterns were considered: slow monotonic loading; moderate monotonic loading; slow reversed-cyclic loading; fast reversed-cyclic loading; pseudo-dynamic loading; and shaking table test. The following outcomes were reported: (1) Force-deformation behaviours of nailed connections were influenced by the rate of displacement, and number of cycles and their amplitude. (2) Fast loading tests predicted smaller ductility than slow loading tests. As such, it was recommended that seismic performance be evaluated using fast loading protocols since slow loading tests may over-estimate the ductility. (3) Loading protocols of relatively longer loading (i.e. pseudo-dynamic test) induce greater strength degradation as well as increase deformation with respect to the shaking table test. (4) Faster reversed-cyclic test produced results in close range to those obtained in the shaking table test.

Shenton et al. (1998) executed a series of tests to characterize the degradation of stiffness and energy dissipation of shear walls under cyclic loadings. The researchers concluded that shear walls under cyclic loading exhibited higher stiffness as compared to quasi-static loading. Yamaguchi and Minowa (1998) performed shake table and static tests to examine and compare the response of shear walls. The researchers noted that walls under the cyclic tests exhibit higher strength but lower ductility than walls tested using the monotonic tests.

Dinehart and Shenton (1998) performed 12 tests to compare load carrying capacity and ductility of shear walls under static and cyclic tests. The results revealed that the ultimate loads measured using the cyclic test are slightly less than those measured by the static tests. However, the wall ductility under dynamic tests was between 34% and 42% less than the wall ductility during static tests.

3.10 Braced Timber Frames

Another seismic resisting system for wood buildings is the braced timber frame that is used when large open space is required. In this system, wood braces are used to transfer the lateral loads induced by the earthquake. Popovski (2000) investigated the seismic performance of such braced systems and provided proper connection solutions to enhance the response of the systems under seismic loads.

This study focused more on wood structures with lateral resisting systems made of wood shear walls. For more information on the braced timber frame system see Popovski et al. (2002).

4 A REVIEW ON NUMERICAL MODELING

Different numerical models have been developed to simulate the complex behaviour of timber structures during seismic events. There are several challenges in performing a structural analysis for timber buildings subjected to the seismic load. Wood is a hygroscopic (ability to attract moisture from the air), anisotropic material (its structure and properties vary in different directions) of biological origin. The biological origin of wood means its diversity and variation, between and within different species of trees. These characteristics of wood make the prediction of its material properties very complex and difficult for analysis and numerical modeling. Woods are considered among the most highly nonlinear materials. In addition, the strength of wood members depends on moisture content, direction of grain, and defects (i.e. splits); wood strength is also affected by heat and water exposure, and fungus. Consequently, for the analysis of a structure subjected to a seismic load, a nonlinear analysis is required since the linear analysis may not be sufficient.

Under the application of earthquake loadings, the overall response of a structure is controlled by the interaction of several factors. When a wood-frame building is exposed to earthquake, it is expected that the structure will experience significant degradation in stiffness, with major changes in dynamic characteristics due to cracking of gypsum wall-boards.

For many years, considerable research efforts were devoted to establish satisfactory analytical tools for analysis and design of wood-based structures. Most of the work has been limited to investigate performance of 2D structures or individual components, such as shear walls, diaphragms, fasteners, and connections under monotonic and/or cyclic loadings. Chehab (1982) presented an analytical model to represent the linear behaviour of a typical wood frame house under seismic loadings. The results illustrated the capability of the model to simulate the effects observed in earthquake-damaged houses, including torsional sensitivity as a result of the asymmetric arrangement of shear walls.

Cheung and Itani (1983) proposed a finite element model to study the performance of light frame shear walls subjected to static and dynamic loadings. The model utilizes load-displacement characteristics to simulate nonlinear behaviour of nail elements. Itani and Cheung (1984) developed a finite element model which represents nail connections, frame members, and sheathing panels using joints, beam elements, and two-dimensional plane elements, respectively. The model incorporates load-slip properties to model the nonlinear behaviour of nail elements. The model was limited to capture different arrangements of sheathing panels, loading conditions, and asymmetric geometry. The numerical results were compared and found to be in good agreement with experimental results.

Itani and Robledo (1984) presented a finite element model that uses joint elements, beam elements, and constant strain triangles. The model simulates nonlinear behaviour of joints, and considers both continuous panels and panels with openings.

Gupta and Kuo (1985) developed a simple mathematical model to analyze the behaviour of shear walls. The researchers implemented a generalized coordinate approach to establish equilibrium equations for walls, and then extended the concept for multiple panels. Suitability of the approach was performed through finite element modelling of a tested shear wall. Two years later, Gupta and Kuo (1987) developed a numerical model to analyze linear elastic performance of a tested building. The model had nine degrees of freedom, and incorporated seven “superelements”. The superelement was based on a shear wall model previously developed by Gupta and Kuo (1985). The superelement concept was used to represent shear walls, diaphragms, and ceilings. As for the nine degrees of freedom, five of them were defined for horizontal displacement at the top of walls, two for vertical displacement, and two for uplift behaviour of studs along walls.

Patton-Mallory and McCutcheon (1987) formulated a model to predict rocking resistance of shear walls. The model addressed the contribution of different sheathing materials, and the effect of installing sheathing panels on both sides of the wall. Further, the model incorporated load-slip relations for fasteners, which were defined based on satisfying asymptotic curves. The model predicted well the rocking response of tested shear walls.

Kasal et al. (1994) developed a FE model of a single storey shear wall using ANSYS (Kohnke 1989). The researchers assumed linear behaviour and used superelements to represent floor and roof systems. Shear wall systems were represented using quasi-superelements containing a truss and diagonal spring elements, which were treated as nonlinear elements. The nonlinear parameters of the spring elements were defined based on experimental results. The assessment of the model was carried out by testing a full-scale house subjected to cyclic loadings.

White and Dolan (1995) formulated a finite element model to examine monotonic and dynamic behaviours of timber shear walls. The model uses beam element, plate element, nonlinear spring element, and bilinear spring element to represent framing members, sheathing panels, nail connections, and bearings between adjacent sheathings, respectively. The suitability of the model was compared with experimental results, and the model predicted well the load bearing capacity of the tested shear wall.

Davenne et al. (1998) proposed a finite element model to analyze a shear wall subjected to cyclic loadings. The model simulated the strength degradation associated with the cyclic response based on satisfying test curves for the first and subsequent cycles.

He et al. (2001) developed a computer program called LightFrame3D to model the performance of 3D timber frame structures subjected to static loading. The software library offers a thin plate element to model a plywood panel, beam element to model a stud member, and nonlinear spring element to model a nail and the surrounding wood medium. The software also incorporates a mechanic-based representation of load-displacement laws to capture shear and pullout behaviour of nail connections. Further, the software recognizes various materials for structural components, analyzes structural systems under combined loading scenarios and torsional response of eccentric facilities.

Folz and Filiatrault (2001) developed a numerical model to predict the shear wall performance under a quasi-static loading. The model incorporates rigid framing members, linear elastic panels, and nonlinear nailed fastener connections. The model captures strength and stiffness degradations, as well as hysteretic pinching.

Follesa et al. (2010) performed a blind prediction using the DRAIN 3-D program of the response of a 6-storey wood frame building, tested on a shake table in Japan, reported by van de Lindt et al. (2010). The results of the analysis were in reasonable agreement with the ones obtained from the shake table tests (Follesa et al. 2010). C. Ni et al. (2009) carried out numerical simulations of a 6-storey building, in comparison to a 4-storey building, located in Vancouver BC subjected to seismic loads using DRAIN 3-DX and SAPWOOD programs. The results of this study were used to compare the seismic response between a typical 4-storey and 6-storey wood-frame structure. It was found that the response of the 6-storey structure was the same or slightly better compared to that of the 4-storey one. The results from the study were partially used by the Association of Professional Engineers and Geoscientists of B.C (APEGBC) when developing the new design guidelines for 5- and 6-storey residential buildings in the BC Building Code.

Richard et al. (2002) presented and validated a FE model formulation to investigate the seismic behaviour of shear walls with openings. The nonlinear load-displacement law of nails was programmed to account for the resistance of the nail type connection under cyclic loading, and to represent the consequential global nonlinear behaviour of the shear wall. Nonlinear rules were also implemented to capture the cyclic performance of hold down devices and its implication on the response of shear walls. The numerical results verified the FE model as a useful tool to predict hysteretic response and monotonic response of the shear wall specimen with relation to the reported experimental data.

Results from most of the above studies, as well as experimental results, indicate the importance of the nails behaviour, at the connection between the sheathing and the wood frame, in the response simulation of the light-frame timber shear walls/structures. Therefore, using a proper model for nails in the numerical analysis could result in a more accurate response prediction of such elements/structures under the seismic load.

5 CONCLUSION

Based on the performed literature review the following conclusions could be made:

- The main failure mode of a single storey light-frame shear wall under lateral loads occurred along the sheathing nails and at the connections of the wood frame components.
- Nail failure is either pull out, mainly under monotonic loadings, or nail fatigue fracture under cyclic load.
- Higher ultimate loads (in some cases, less or more the same) were observed for shear walls under dynamic loads than those under static loading. However, less ductility was observed under a dynamic load.
- Lower ultimate capacity, load/deformation, was observed for shear walls under cyclic loads than those under monotonic loads.
- The governing failure mode of light-frame shear walls changes from one-storey, being connection/nail failure (shear failure) to a flexure failure (compressive/tensile stud failure) for a 6-storey building. Studies are required to assess the effect of the change in the failure mode in the seismic design of multi-storey buildings.

- Increase of openings in walls could reduce the resistance capacity of the shear walls.
- Non-structural finishing materials contribute to (or enhance) lateral-load bearing capacity, and reduce seismic demand of timber assemblies. It is commonly agreed that sheathing materials (e.g. gypsum board) enhanced stiffness and strength, as well as torsional resistance capacity of timber structures.
- The behaviour of wall diaphragms varied between flexible and rigid elements based on the structural system, floor level, and amount of shear deformation experienced by the diaphragms. Studies are needed to develop more reliable tools (e.g. finite element model) for nonlinear analyses and design of mid-rise wood structures with flexible diaphragms.
- The size, material, and type of sheathing panels impact structural characteristics, such as energy dissipation capacity, stiffness, and strength of shear walls. More studies are required to better quantify such an impact in the design.
- Installing hold-down devices reduce uplift deformations at wall ends, and thus enhance the initial lateral stiffness, overall deformation, and energy dissipation capability of shear walls.
- More studies are needed to obtain more reasonable and consistent method to determine the natural period of mid-rise wood buildings.
- Structural dynamic response characteristics of multi-storey wood buildings, such as effects of higher modes, e.g. higher acceleration at the higher storeys, torsion modes, and failure modes, play major roles in determining the seismic load capacity of the buildings.
- More reliable numerical models could be developed to improve structural analysis of wood buildings, e.g. nail connection models.

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