Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 58, No. 3, September 2025

SEISMIC PERFORMANCE AND LOSS ASSESSMENT OF LIGHT TIMBER FRAME RESIDENTIAL HOUSES IN NEW ZEALAND: STATE OF THE ART

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(Submitted *June 2024*; Reviewed *August 2024*; Accepted *October 2024*)

ABSTRACT

Past earthquake experiences in New Zealand indicate that light timber-frame (LTF) residential housing stock in New Zealand could suffer significant damage in major earthquakes, leading to significant downtime and economic losses for the community. It is necessary to develop a rigorous approach to predict seismic damage on LTF residential houses and estimate the subsequent economic losses. This paper provides an overview of recent research advances in the fields of seismic performance assessment and seismic loss models for LTF residential houses in New Zealand. It systematically reviews the evolution of residential houses in New Zealand, experimental and simulation studies of plasterboard bracing walls and LTF buildings, numerical modelling methods currently used for wood shear walls, and prevailing building seismic loss estimation models. In addition, recent technological advancements and current design recommendations relevant to such LTF houses and bracing walls are highlighted. Possible future research directions are recommended to better understand the seismic performance and develop a loss estimation framework for LTF residential houses in New Zealand.

https://doi.org/10.5459/bnzsee.1701

INTRODUCTION

Generally, in light timber-frame (LTF) residential houses, walls provide stiffness and resistance to lateral wind and seismic forces [1]. In North America, LTF shear walls are commonly sheathed with plywood or oriented strand board (OSB) and fastened with nails. However, in New Zealand, plasterboard bracing walls are widely used as the gravity and lateral load-resisting systems for LTF residential houses [2]. Most residential houses in New Zealand are low-rise (single or double storied) houses, over 90% of which include LTF proprietary bracing wall systems made of plasterboards [3]. A plasterboard bracing wall has plates and studs made of timber, to which plasterboard panels are sheathed on one side or both sides. Optional fixing methods for sheathing plasterboards to timber framing include adhesive, screws, and nails, with screws being the more commonly used fasteners than nails. Edges of the sheathing panels can be blocked or unblocked. The plasterboard for walls is also known as gypsum wallboard (GWB) or drywall.

Low-rise LTF structures normally have a low probability of collapse under earthquakes. Because wood is a material with a high strength-to-weight ratio, LTF residential houses are lighter than concrete and steel buildings of similar sizes, thus attracting lower seismic forces. The LTF shear walls have intrinsic redundancy which makes the whole structure very robust against collapse. That is why LTF residential houses can easily avoid structural failures and achieve the life safety objective [4]. According to previous research [5,6], low-rise LTF houses could sustain a storey drift of 6% before reaching the collapse limit state.

However, collapse avoidance is not the only target of seismic design. LTF residential houses may still suffer severe damage under high-intensity earthquakes even if they do not collapse. For example, in the 2010 Darfield earthquake and the 2011 Christchurch earthquake, unprecedented damage to LTF residential houses with plasterboard walls was recorded. The estimated total economic losses to residential houses caused by the 2010-11 Canterbury earthquake sequence was around $12B, about 30% of the total losses [7].

Over the years, many studies have investigated the seismic performance of plasterboard bracing wall systems and explored empirical seismic loss models for typical New Zealand residential houses. However, no studies have systematically scrutinised the progress made on these topics and reviewed the current state of art to identify the knowledge gaps and needs for further research to enhance the seismic design and performance of LTF residential houses. Moreover, the relationships between earthquake intensity, seismic damage, and economic loss for New Zealand LTF residential houses are not well understood. This paper revisits and summarises the development of bracing wall systems in New Zealand residential houses, and reports the characteristics of plasterboard bracing walls. It also reviews experimental and numerical as well as analytical investigations on plasterboard bracing walls and LTF houses, and explores current literature to understand the seismic loss assessment models used for LTF residential houses in New Zealand. While the scope of this paper is primarily limited to conventional construction materials and methods used in New Zealand residential houses, some overseas studies are included to demonstrate the current state of knowledge and research on these topics in other countries.

RESIDENTIAL HOUSES AND DEVELOPMENT OF BRACING WALLS IN NEW ZEALAND

In New Zealand, there are three predominant residential housing typologies: the typical 1930s timber frame bungalows, the 1940-1960 timber frame houses, and the post-1980s brick veneer timber frame houses (as shown in Figure 1). They make up over 95% of the residential houses stock (by value and quantity) [7]. Aside from these three types, other typologies include the pre-1940 unreinforced masonry houses (which were

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phased out of construction following the 1931 Napier Earthquake), the post-1980s houses with reinforced concrete tilt-up slabs, and houses made of reinforced hollow concrete blocks [7].

The lath and plaster system is the earliest internal lining used for timber walls in New Zealand [8]. In this system, wood laths are nailed across the wall studs, and plaster is forced into the gaps between the laths and covers the full wall [9]. Lath and plaster can only provide little lateral capacity and fail in a brittle mode at low loads. The main lateral load resistance comes from the diagonal braces. As shown in Figure 2, some braces are cut between studs, and some are fitted into slots cut into the studs. According to the Canterbury earthquake survey [5], the use of lath and plaster on the exterior of houses was common in the early 1900s houses. Some cases were observed where sheets of the plaster were detached from the lath, and both the plaster and the lath broke away from the wall (Figure 3).

Figure 1: Predominant residential housing typologies: (a) typical 1930s timber frame bungalow; (b) 1940-1960 timber frame house; (c) post-1980s brick veneer timber frame house [7].

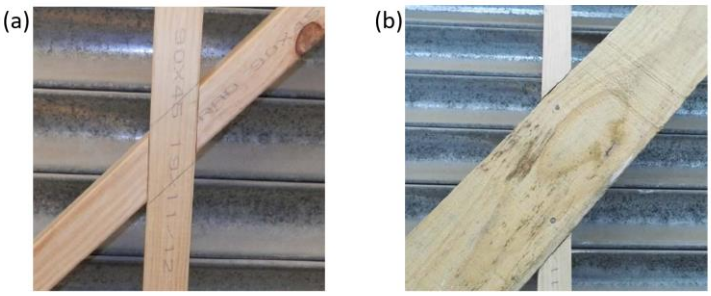

Figure 2: Diagonal timber braces: (a) cut between studs; (b) a let-in brace [10].

Figure 3: A failure example of lath and plaster wall [5].

Weatherboard is commonly used in the exterior walls of the pre-1940s timber frame bungalows. As shown in Figure 4, the weatherboard is fixed to wall studs with nails at some distance from the bottom of each weatherboard. The resistance of this kind of wall is expected to be provided by the moment couples between the horizontal lines of nails and the friction of one board against the next. Figure 5 illustrates the hysteresis loops of a 2.4 m long weatherboard wall tested by BRANZ [9]. Although the hysteretic load-drift curves of the weatherboard wall were fat and stable, the maximum load was very low, around 1kN. Therefore, weatherboards cannot be considered as bracing materials. The interior side of the wall needs to be lined with much stiffer panels in order to provide bracing capacity.

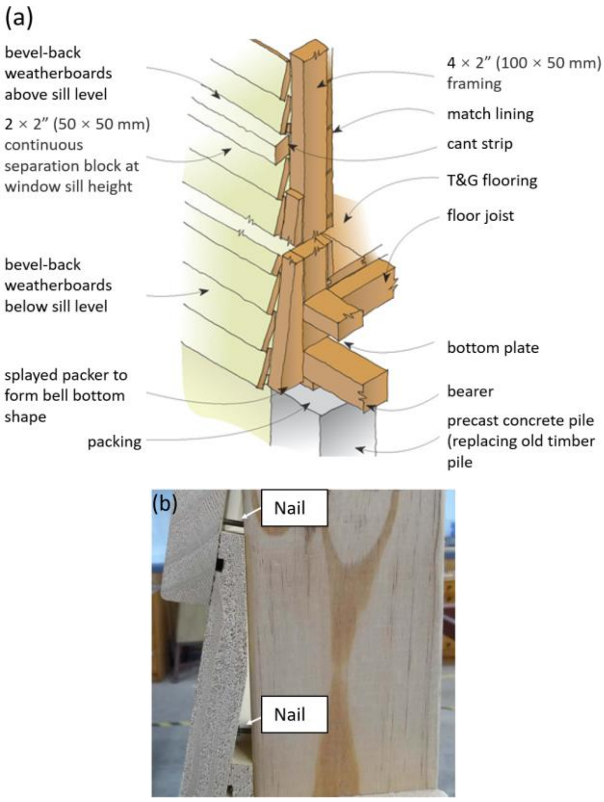

Figure 4: Bell-cast horizontal weatherboard: (a) overall construction [13]; (b) nails detail [9].

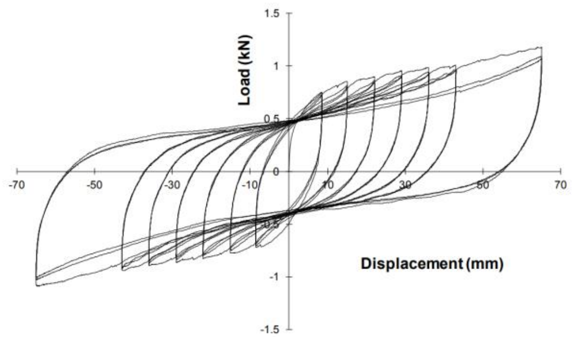

Figure 5: Hysteresis loops of a 2.4 m long weatherboard wall [9].

Fibrous plaster sheets were first introduced during the 1920s and 1930s in New Zealand [11], and developed to replace the lath and plaster system [8]. Fibrous plaster, also known as Hessian fibre-reinforced gypsum, is a type of plasterboard sheet reinforced with a mixture of fibres. Figure 6 illustrates the construction of fibrous plaster. Beattie et al. [8] stated that the product fit within the description of a generic bracing system in the early versions of NZS3604 [12] and was expected to act as a bracing element. However, the bracing capacity of fibrous plaster sheets is very low, and the diagonal braces mainly provide the lateral capacity.

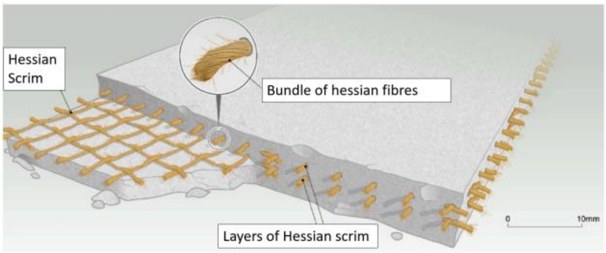

Figure 6: Section of fibrous plaster with exposed scrim layer [14].

Softboards and hardboards were commonly used for lining in the middle of the 20th century [5]. Softboards and hardboards are wood fibreboard with low and high densities. Softboards are usually used for lining living rooms and bedrooms, whereas hardboards are used for lining utility rooms, i.e. bathrooms, kitchens and laundries. Softboards are generally fixed with steel clouts or glue, while hardboards are fastened with brads. They also provide little bracing capacity.

Plasterboards have been commonly used in New Zealand since the 1920s and domestic manufacturing began in 1925. Plasterboards and wood-based panels became the predominant wall-lining materials in the 1930s [11]. Using plasterboards has several advantages including lower material cost, and fire protection [15]. Compared with the earlier versions of bracing wall systems, plasterboards can meet greater bracing demand in modern LTF houses [8]. Therefore, diagonal bracing is no longer needed in plasterboard bracing walls. A typical plasterboard bracing wall is shown in Figure 7. The plasterboards are fixed to the timber frame by fasteners (normally by screws). The bottom plates of the walls are bolted or coach-screwed to the foundation beam. Sometimes hold-downs are used at the wall ends. Plasterboards used in New Zealand LTF walls include the standard plasterboard, the bracing plasterboard with a higher density core or fibreglass reinforcing in its core, the fire-resistant plasterboard and the water-resistant plasterboard. Plasterboard products could be shown to be compliant through conformance with the manufacturing and performance specifications with AS/NZS 2588 [16].

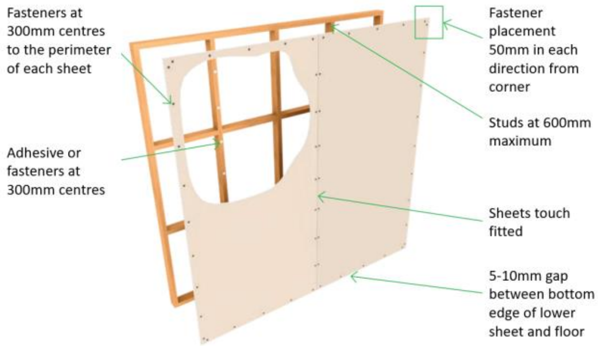

Figure 7: A typical plasterboard bracing wall [17].

CHARACTERISTICS OF PLASTERBOARD BRACING WALLS

General Performance of Plasterboard Bracing Walls

Plasterboard bracing walls in New Zealand residential houses are the main structural element that resists in-plane shear forces. In other countries where LTF houses are widely used, such as the United States and Canada, LTF walls are usually sheathed with plywood sheets or oriented strand boards (OSB) on one side only, or on both sides with plasterboards on the other side [1,15,18]. Regardless of the material of panels used, these walls can all be categorised as light wood-frame shear walls sheathed by panels. They have similar mechanisms in resisting the racking loads. However, because plasterboards are weaker and more brittle than wood-based panels, plasterboard bracing walls have different racking responses compared to bracing walls sheathed with mainly wood-based panels [10,19].

Chen et al. [1] tested and compared the performance of OSB sheathed walls and Type X plasterboard sheathed (on one side) walls used in Canada. Type X plasterboard is a special fire-resistant plasterboard popular in North America. Special glass fibres are intermixed with gypsum to reduce the size of the cracks that form as the crystalline water is driven off during fire, thus extending the length of time the panels maintain their structural integrity. The common thicknesses of Type X plasterboard include 12.7mm, 15.9mm and 25.4mm. In New Zealand market, there are some similar plasterboards which have fire ratings longer than the standard plasterboards. Available thicknesses include 10mm, 13mm, 16mm and 19mm. Figure 8 illustrates the load-displacement hysteretic responses of two walls of the same size and made of Canadian Spruce-Pine-Fire framing members. The only difference was that SW-01 was sheathed by 12.5mm thick OSB on one side using 8d ($\phi3.5 \times 63.5$ mm) common wire nails while SW-03 was sheathed by 15.9mm Type X plasterboard using drywall screws #6 ($\phi2.87 \times 50.8$ mm). The spacing scheme of the nails and screws is the same, i.e. spacing at 152mm on centre along the panel edges and 305mm along intermediate studs. The results showed that the plasterboard walls had much lower strength, less energy dissipation, and lower deformation capacity and ductility.

Wang et al. [19] collected a series of P21 test data (introduced in the next section) of bracing walls used in New Zealand LTF houses and analysed the effect of the sheathing material on the walls' lateral performance. In these wall specimens, plasterboards were fixed to timber framings by screws, while plywood panels were fixed by nails. The average maximum loads and the average drift ratios at the maximum loads of the walls sheathed by different materials are shown in Figure 9(a) and 9(b), respectively. Each pair of two adjacent columns represents values of two wall types with the same construction details but different sheathing materials. The maximum loads of plasterboard bracing walls were lower than that of plywood sheathed walls, and the drifts of plasterboard bracing walls at the maximum loads were also lower than those of the plywood sheathed walls. The test results indicated that, compared to plywood sheathed walls, plasterboard bracing walls are less ductile with lower energy dissipation capacity.

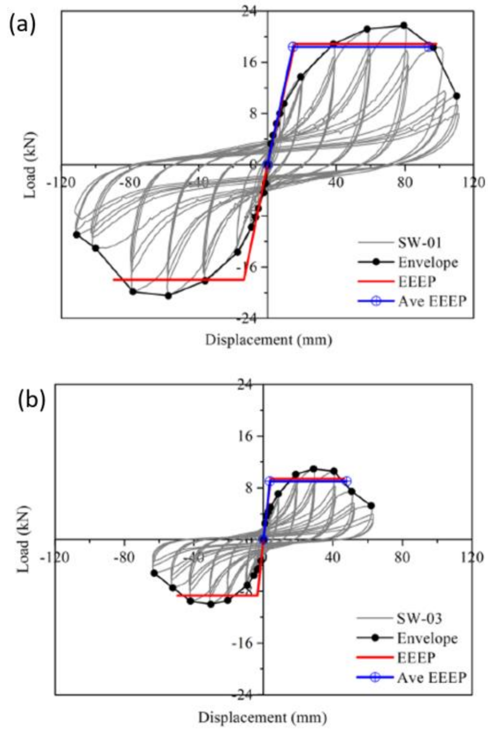

Figure 8: Load-displacement curves of shear walls under reverse cyclic loading: (a) the shear wall sheathed by OSB; (b) the shear wall sheathed by plasterboard [1].

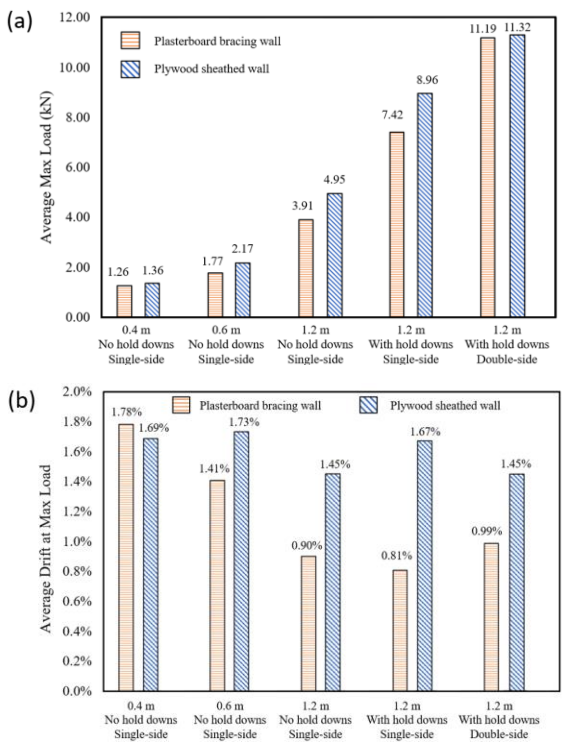

Figure 9: Effect of sheathing material on: (a) maximum loads; and (b) drifts at the maximum loads [19].

Plasterboards are also used as linings for other wall systems. For example, they can be installed as interior linings of light (cold-formed) steel frame load-bearing walls. Previous studies [20,21] found that plasterboards can increase initial stiffness and modestly increase the strength of the walls. However, plasterboards are not considered the main bracing material in this system, and the performance of light steel frame walls is different from that of LTF plasterboard bracing walls. Plasterboard sheathed timber walls are also used as infill walls in reinforced concrete (RC) frames and steel frames [22,23]. In such systems, plasterboard infill walls are not designed to be lateral load-resisting elements, but rather non-structural elements. A series of quasi-static tests on plasterboard infill walls within RC frames were performed by Tasligedik et al [24]. The cyclic performance of the plasterboard infill walls showed a higher peak load and lower drift ratio at the peak load compared to structural plasterboard walls.

In summary, the performance of plasterboard bracing walls used in New Zealand is different from that of walls sheathed by wood-based panels, the light steel frame walls with interior plasterboards, and the infilled plasterboard walls in RC/stell frames. Many overseas researchers studied the effects of plasterboard on the seismic performance of LTF shear walls braced by wood-based panel products [18,25,26], and the performance of the whole LTF houses [27–30]. However, there are only few New Zealand-based studies on plasterboard bracing walls. As plasterboard bracing walls are the main lateral load-resisting systems of typical New Zealand LTF houses, developing a good understanding of their seismic behaviour is the key to a reliable assessment of the seismic performance of New Zealand LTF residential housing stocks.

Timber-frame Bracing Design in NZ Standards

In New Zealand, buildings are designed to resist structural design actions, the general principles of which are outlined in AS/NZS 1170.0 [31]. AS/NZS1170 Parts 1, 2, 3 and NZS1170.5 [32] specify the permanent, wind, snow and ice, and earthquake design actions, respectively. For timber-framed residential houses, NZS 3604 [12] is referenced as an Acceptable Solution for Building Code clause B1 Structure (buildings will withstand likely loads, including wind, earthquake, live and dead loads). It provides methods and details for NZ timber-framed houses and small buildings for the code compliance.

Current seismic design standards are generally developed to achieve life safety at ultimate limit state (ULS) events and control deflections at the serviceability limit state (SLS) events. The inter-storey deflection limit at ULS in NZS1170.5 is 2.5% of the corresponding storey height or lesser as may be prescribed in the appropriate material standard. For SLS, the drift limit is specified as 0.33%. The LTF houses designed per NZS3604 could easily achieve life safety performance target at design-level earthquakes [2].

In NZS3604, the earthquake bracing demand is determined by the building location, subsoil type, the building size, roofing and cladding weights, and floor live loads. The demand is developed based on the equivalent static method which is a force-based approach according to NZS1170.5. The design base shear force, V, is determined by the following equation:

$$\mathrm{V} = C_d(T_1)W_t \tag{1}$$

where $C_d(T_1)$ is the horizontal design action coefficient derived by assuming a ductility of μ and a fundamental period of T_1, and W_t is the seismic weight. The equivalent static horizontal force (F_i) at each level (i) is obtained from the following equation:

$$F_i = F_t + 0.92V\frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i} \tag{2}$$

where $F_t = 0.08V$ at the top level and zero elsewhere, W_i is the seismic weight of level i, and h_i is the height of level i. The earthquake forces (bracing demand) are also presented in "bracing units" (BUs) where 1 kN equals 20 BUs.

NZS3604 [12] specifies so-called P21 tests to evaluate the bracing ratings of bracing wall elements. The P21 test method, developed by BRANZ [33], aims to determine the seismic bracing capacity of proprietary LTF shear walls and ensure that these walls have adequate strength, stiffness, elastic recovery, and resistance under cyclic loads. Figure 10 shows the P21 test setup. The P21 test is a slow cyclic racking test performed by applying a lateral load at the top of the test specimen. The bracing rating of a specified bracing wall system is determined by experimentally subjecting three nominally identical full-scale specimens to an incremental series of cyclic lateral in-plane displacement sets and measuring the force that the wall resists within a defined displacement range. P21 tests are often conducted on a standard wall length of 1.2 m. For longer walls up to 2.4 m in length, the seismic rating per meter length is assumed to be the same as for 1.2 m long walls [4]. Overall, the P21 test method is similar to other overseas test methods for lateral force resisting systems such as the ASTM E2126 standard. One difference is that in the ASTM E2126 standard, the racking load is applied to the test specimen through a load beam which is fixed to the top plate of the test wall, whereas the P21 test specifies that the horizontal load is applied in the middle of the test wall, as shown in Figure 10(a). In addition, the P21 test uses supplementary uplift restraints at each end of the test specimen. Construction details of the restraint are shown in Figure 10(b). A bolt or coach screw providing a sliding attachment between the angle and the end of the specimen through a slotted hole is also acceptable.

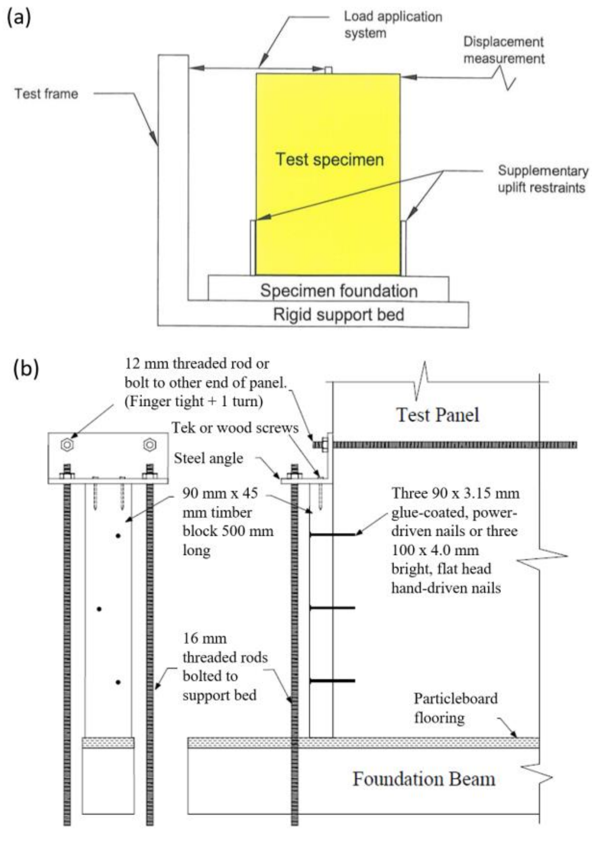

Figure 10: P21 Test arrangement [12]: (a) setup; (b) supplementary uplift restraint.

Apart from satisfying the bracing demand, the bracing elements are required to be evenly distributed along notional "Bracing Lines" in each direction (along and across the ridge) of the building. It is specified that the bracing lines in any storey shall be placed at not more than 6 m centres apart. On each bracing line, the minimum bracing provision is the greater of 100 BUs or 50 % of the total bracing demand divided by the number of bracing lines in the direction being considered. Besides, the minimum bracing resistance for each external wall in any storey shall be no less than 15 BUs/m of external wall length. As a result, LTF residential houses constructed to NZS3604 have an 'egg-crate' structural form and are considered to be reasonably regular both in plan and elevation [4].

The wall bracing demand in the current version of NZS3604 (revised in 2011) has been re-examined by several studies. Liu [2] analysed the expected earthquake performance of a case study LTF residential house with the minimum standard seismic bracing provision by using the direct displacement-based approach. The deflection requirement at ULS for plasterboard bracing walls was determined to be 1% storey drift based on the available P21 test results. Figure 11 shows the relationship between the response acceleration (Sa) and response displacement (Sd) for the site of the case study building, which was calculated by the direct displacement-based method. The bracing capacity of this building is equivalent to Sa=0.4g. As shown in Figure 11, the bracing walls need to deflect to 70mm (3.0% drift) in a 500-year event even if the bracing system could maintain strength and 20% equivalent viscous damping beyond 22mm deflection (i.e., 1% drift). It was concluded that the expected seismic deflection of the conventional LTF house (with plasterboard bracing walls) designed per NZS3604 would be larger than the specified deflection limit of 2.5% storey drift at ULS. The author suggested the seismic bracing demand in NZS3604 potentially needs to be increased by 40% at ULS.

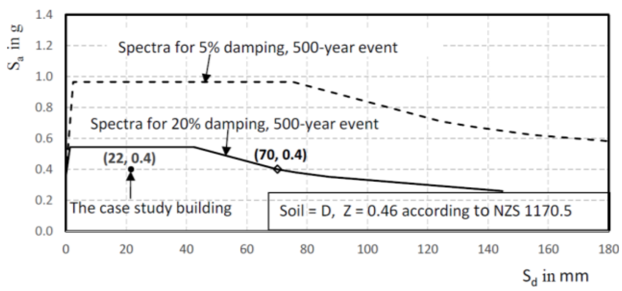

Figure 11: Relationship between spectral acceleration and spectral displacement [2].

The design guidance for LTF bracing systems, suggested by BRANZ [4], is based on the extension of the above research. It was reported that many new LTF houses use specifically designed bracing elements that are out of the scope of NZS3604. The guidance highlighted that potential stiffness incompatibility between conventional LTF bracing walls and specifically designed bracing elements could lead to significant earthquake damage to LTF houses. A step-by-step seismic design procedure for specifically designed bracing elements was suggested, in which the storey drift limit at the ultimate limit state was set at 1%. Several methods for enhancing the racking performance of plasterboard walls were suggested including improving the connection details between timber frame members and improving the hold-down details at wall bases.

In terms of dynamic characteristics, there are no specific recommendations for timber-frame structures in the design standards. When applying time history analyses, NZS1170.5 requires 5% viscous damping for all modes whose period is less than the incremental time step included in the analysis. If Rayleigh damping is used, there shall be no more than 5% of critical damping in the two first translational modes.

EXPERIMENT STUDIES ON PLASTERBOARD BRACING WALLS AND LTF RESIDENTIAL STRUCTURES

Bracing Wall Elements

Wolfe [35] tested 30 plasterboard sheathed walls under monotonic loads to determine the plasterboard's contribution to the wall racking resistance. Some walls had diagonal wood braces or metal strap braces while others did not, and the wall-length range was 8, 16, and 24 feet (2.44, 4.88, and 7.32 m). The typical test setup is shown in Figure 12. For plasterboard sheathed walls without braces, one had nail failure initially in the tension corners and the other two exhibited nail failure distributed along the top or bottom plates rather than concentrated at the corners. The results showed the total bracing capacity of the wall with a diagonal brace and sheathed with plasterboards was equal to the sum of these elements' resistance tested independently. The ultimate shear strength of the tested walls showed an approximately linear relationship with the wall length, but the wall initial stiffness showed a nonlinear relationship with wall length (approximately a power function). It was also found the walls with plasterboard oriented horizontally showed over 40% higher strength and stiffness than those with vertically oriented plasterboards. Thurston [36] explained that the plasterboards in this study had an unconfined edge at the ends which could crack more easily by nails. This weakness was mitigated by taping and stopping when plasterboards were sheathed horizontally. It was mentioned that New Zealand plasterboards have the same unprotected edges when oriented vertically, and some strength gain with horizontal construction was also observed in some unpublished BRANZ tests.

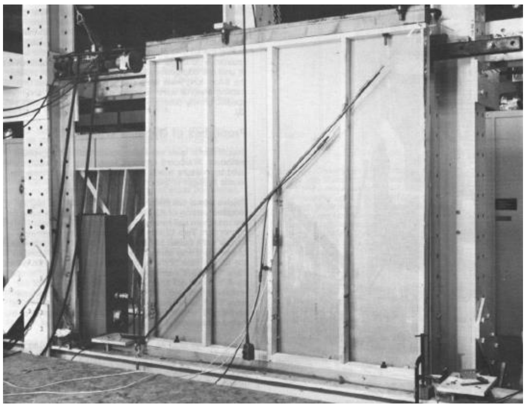

Figure 12: Wall test setup [35].

The openings in walls can also significantly affect the performance of plasterboard bracing walls. Dishongh and Fowler [37] compared the performance of plasterboard sheathed (both sides) walls with and without openings. Eight tests were conducted including three continuous diaphragm walls, three walls with door openings, and two walls with window openings. It was concluded that a wall with a central window opening could be treated as two separate full height bracing walls.

Thurston [36] conducted P21 tests on 10 long plasterboard walls with openings under pseudo-static reverse-cyclic loads. It was found that the performance of walls with large window openings or door openings (with hold-down straps on the edges bounding the door) could be obtained by adding the performance of the two separate walls between the openings. But, for a wall with a door opening in which straps were not used, adding the two segments' performance would overestimate the wall performance. The racking strength appeared to be very close between the walls where plasterboards joined at the window openings (as shown in Figure 13(a), the nearest joint is 300mm or more away from the vertical opening edge) and those where plasterboards were cut at the opening (as shown in Figure 13(b), sheathing sheet edges coincided with the window or door trimmer studs). The racking deformations were analysed and scrutinised. The conclusion was the two most dominant components were the rocking of the entire panel and sheet rotation relative to the frame due to fastener slips. The former one contributed 60% to 100% of the total deformation and the second one contributed 10-50%.

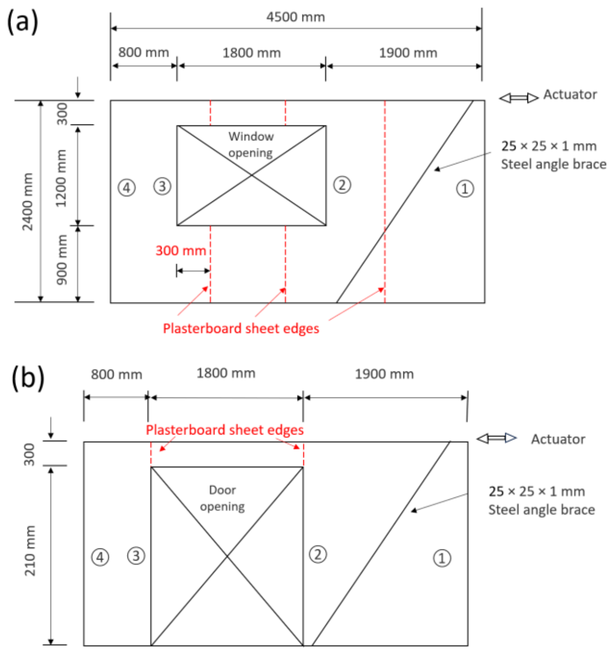

Figure 13: The wall case where plasterboards joined at the window openings [36].

Liu and Carradine [3] analysed P21 test results of 12 plasterboard walls in terms of stiffness/strength degradation, displacement capacity, superposition applicability, and failure mechanisms. It was found that the plasterboard walls showed significant strength degradations under cyclic loading. For the same displacement level, the racking strength in the third cycle was 15% to 25% lower than that in the first cycle. The maximum equivalent damping ratio of these walls was around 15%, which means the plasterboard bracing walls had limited energy-dissipating capacity.

Experimental studies were also conducted overseas to evaluate the performance of plasterboard sheathed walls. Although different design standards and product standards were followed, these studies also provided insightful knowledge to understand the performance of plasterboard bracing walls typical of New Zealand constructions. Chen et al. [1] conducted tests on 12 shear walls sheathed with OSB alone, Type X plasterboard only, and a combination of OSB and plasterboard under monotonic and reversed cyclic lateral loads. The specimens followed construction practice in Canada, where Type X plasterboard (gypsum wallboard, GWB) is commonly used for fire rated wood-frame walls and can be used for shear wall applications [38]. The test setup is shown in Figure 14. It is noted that the end studs and the bottom plates of the test specimens were firmly bolted down to the rigid foundation beams. The test program concluded that the racking resistance of shear walls sheathed with OSB and plasterboard on opposite sides can be estimated by summing those of shear walls with OSB or plasterboard alone (the direct superposition rule). Using joint tapings and the second layer of plasterboard could increase the strength and decrease the ductility ratio, and the walls with the panels placed vertically provided higher strength and energy dissipation than the walls with the panels placed horizontally.

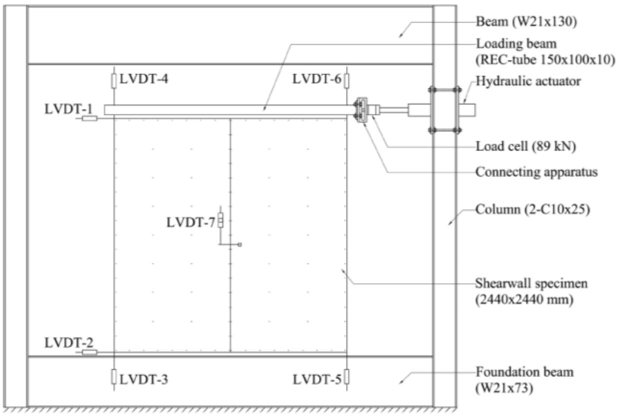

Figure 14: The wall test setup [1].

Lafontaine et al. [15] tested eight full-scale Type-X plasterboard sheathed shear walls under reversed cyclic loading to investigate the effect of fastening parameters. Typical failure modes included fastener bending, panel edge tear out, plasterboard fissure failure, end nails yielding and withdrawal, hold-down deflection, and wood crushed by anchor bolts (as shown in Figure 15). Common parameter variations in LTF plasterboard shear wall constructions that affect the response were found to be fastener type, panel orientation, shear wall length, joint compound type, and loading type.

Figure 15: Typical plasterboard sheathed shear wall (overseas cases) failure modes: (a) fastener bending; (b) panel edge tear out; (c) plasterboard fissure failure; (d) end nails yielding and withdrawal; (e) hold-down deflection; (f) wood crushed by anchor bolt [15].

Building Systems

Several filed tests have been conducted on LTF residential houses. A single-storey LTF house was tested by BRANZ [39]. The test house was a standard Fletcher Homes house, typical of those at the low-cost end of the market available around 1990, having plasterboard linings and fibre-cement weatherboard claddings. The bracing walls were 2.4m high, sheathed by 10mm thick plasterboards, and had no hold-downs. The house plan and wall cross-section details are shown in Figure 16(a). Free vibration tests and cyclic racking tests were conducted. The test setup is shown in Figure 16(b). The load was applied using two hydraulic jacks to the ceiling plane at four locations along the house length. Four timber load beams located in the ceiling cavity were used to spread the applied force along the adjacent house walls. The two hydraulic jacks were fixed to separate reaction frames which were bolted to an existing concrete pad. According to the free vibration test results, this house had a natural frequency of 20.8Hz (fundamental period of 0.05s) and an average critical damping of 8.2%. The test results showed that the averaged cyclic strength of the whole house was 50% greater than that predicted based on the summing of all walls' strengths derived from P21 test results.

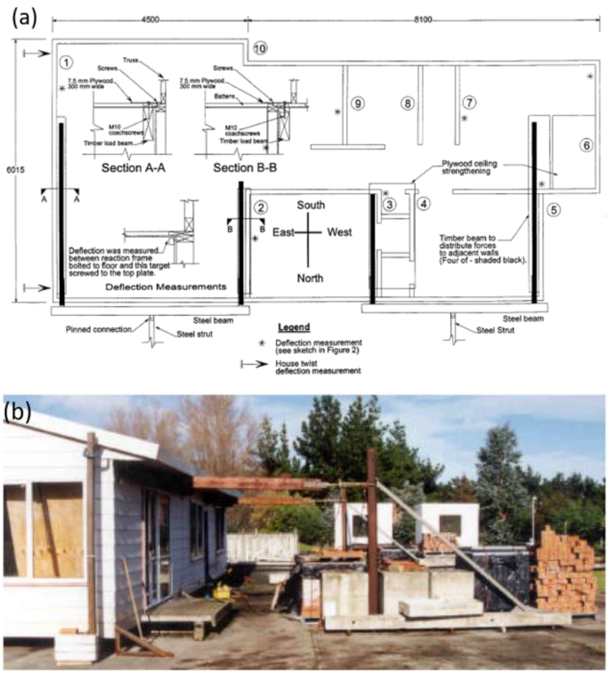

Figure 16: Test house [39]: (a) house plan; (b) general view of test in progress.

Morris et al. [40] summarised an in-situ testing program on post-quake houses after the 2010-11 Canterbury earthquake sequence. This program conducted quasi-static cyclic tests and snap-back tests on 5 houses. Two houses were built before the 1970s. One house, built in 1923, had walls lined by plaster on lath, and the other house, built in 1947, had walls lined by fibrous plaster and light timber panelling. The remaining three houses were constructed after 1970, braced by plasterboard sheathed walls. The measured structural properties of the houses, including lateral strength, stiffness, fundamental period, and damping ratio are listed in Table 1. It was found that the periods of two newer houses (built in the 1980s and 1990s) were all 0.14s.

A full-scale one-storey simple building with long plasterboard bracing walls was tested at BRANZ [10] by applying cyclic loading to determine the bracing performance of long plasterboard-lined walls. Along the loading direction, the building had 2.4 m and 3.6 m long plasterboard walls, as shown in Figure 17. The test results showed the wall bracing strength of the test building degraded more slowly than that of the isolated long walls in the P21 test. The walls were more ductile than the isolated walls and the strength was around twice of combined isolated walls with the same total length. This matches the findings by Thurston [9], i.e. LTF houses appear to have higher capacity than the simple summation of individual bracing walls. Furthermore, it was found that the test building had a systems overstrength factor of approximately 2.0 and the author thought this was mainly attributed to the plasterboard tapes between the orthogonal walls. The value of this overstrength factor was the same as that found in [39].

Table 1: Information and structural features of the test house [40].

Address	Built year	Wall lining	Stiffness	Period	Damping
Retreat Road	1923	Plaster on lath	3.8 kN/mm	0.29s	12% (snapback linear)
Bexley Road	1947	Fibrous plaster and light timbe panelling	9.0 kN/mm	0.23s	
Cardrona Street	1970+	Plasterboard	7.5-8 kN/mm	0.20s	>6%
Wairoa Street	1983	Plasterboard	18 kN/mm	0.14s	
Norcross Street	1993	Plasterboard	27 kN/mm	0.14s	6% (by a hammer blow)

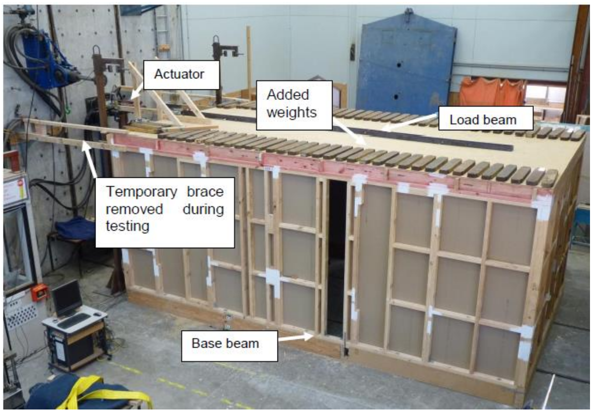

Figure 17: Test building and test setup [10].

BRANZ tested another single-room one-storey building to determine stiffness degradation of LTF houses after earthquake shaking and the effectiveness of different repair methods [41]. As shown in Figure 18(a), the building was nominally 2.4 m high and incorporated windows and doors and two short internal walls. It had plasterboard bracing walls and a plasterboard-lined ceiling. Most of the plasterboard bracing walls were sheathed by standard plasterboards and did not have hold-downs except one wall labelled "BP10" sheathed by bracing plasterboards and with hold-downs. The test setup is shown in Figure 18(b). The structure was first loaded with three cycles of displacement amplitude of 1.65mm, 3.92mm and 7.29mm. Then it was repaired using one method and was retested. Following another repair method, the structure was tested again. The repair methods used in the different phases are listed in Table 2. The comparison between backbone curves from different test phases is illustrated in Figure 19. The results showed that the cosmetic repair was moderately effective at reinstating the initial building stiffness, and adding additional screws showed little improvement compared to the cosmetic repair only. The most effective repair method was fully overlaying plasterboard sheets and adding hold-downs to the ends of the bracing walls. It should be noted that the repair methods suggested in this study were based on the racking tests in which the walls only reached early plastic phases instead of complete failures.

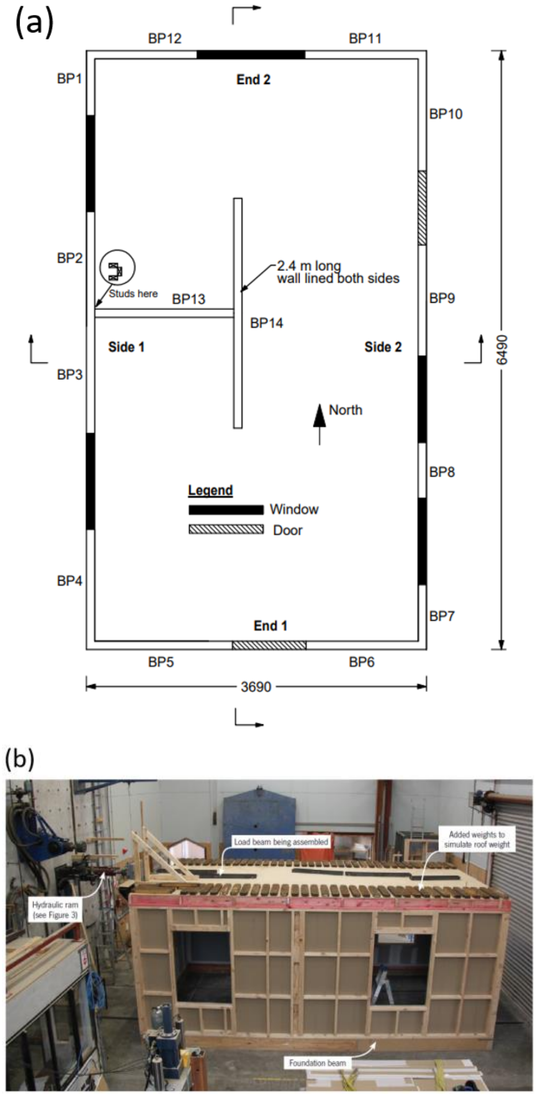

Figure 18: Test building [41]: (a) plan view; (b) test setup.

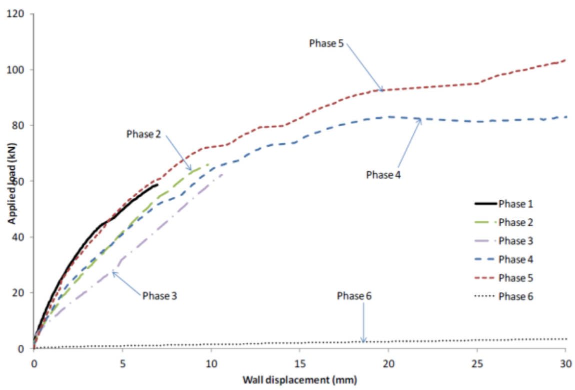

Figure 19: Comparison of backbone curves from each test phase [41].

Table 2: Construction used in the various test phases [41].

Test Phases	Building Condition
1	As-built
2	Cosmetic repair
3	All plasterboard to timber framing glued joints broken (after test 2, hammered a wooden block placed over the plasterboard inward from the outside of the building at all glue joint locations)
4	Cosmetic repair plus strengthening by adding drywall screws between all adjacent existing screws
5	A complete overlay of plasterboard added. Wall hold-down anchors added at ends of bracing elements.
6	All lining on Side 1 and 2 walls removed and replaced. Internal walls removed.

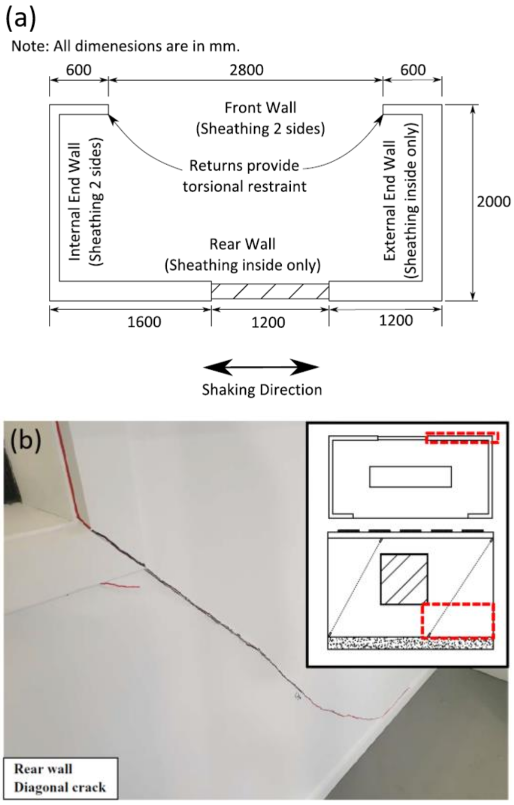

Figure 20: Test structure [42]: (a) floor plan; (b) diagonal shear cracking around window opening.

A full-scale LTF building with plasterboard sheathed walls was designed and tested on the University of Canterbury shake table by Francis et al. [42]. The test structure represented a corner room of a common one-storey house conforming to NZS3604. The floor plan is shown in Figure 20(a). The rear wall and right wall represented external walls, sheathed by 10mm plasterboards as internal linings. The front wall and the left wall represented internal walls, sheathed by 10mm plasterboards on both sides. Hold-downs were used at each end of front and rear walls. Nominal screw spacing for non-bracing walls was used for all walls. The authors explained that if the common screw spacing for bracing walls was followed, the fundamental period would be too short, or an unrealistic quantity of weight needed to be added to the roof to alter the structural period. The total seismic weight of this structure was 11.35 kN after adding additional 920kg of mass to the roof. The test structure was excited in the longitudinal direction only by three New Zealand earthquake records. The fundamental period of the test structure was found to be 0.1s. During low-intensity tests, only minor damage was observed with small cracks initiating from the large opening's corner on the front wall. During the full intensity run of the Darfield earthquake record, the peak inter-storey drift ratio reached 0.23% and more structural damage was observed, including cracks around the corners of the front wall and diagonal shear cracks around the window opening (as shown in Figure 20(b)). Meanwhile, the hold-downs and sill plates were found to be undamaged.

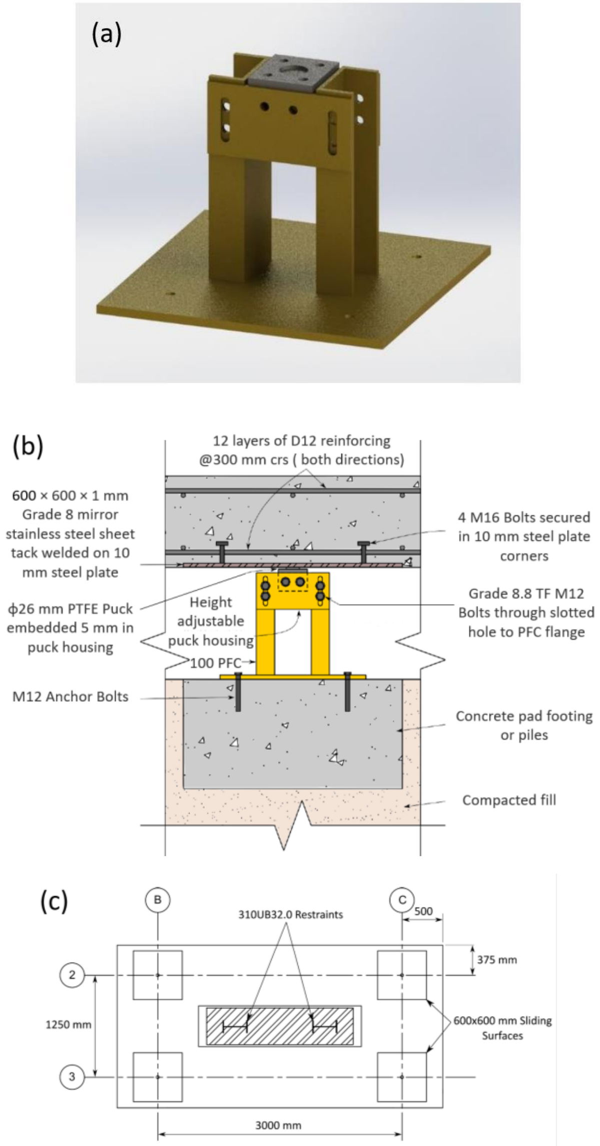

Figure 21: (a) Low-cost base isolation device [43]; (b) and (c) component section details.

A further shake table test was conducted by Francis [43] for a base-isolated LTF building. The super structure was the same as the test structure in [42]. A low-cost base isolation system was proposed which used custom built bi-directional friction slider devices (Figure 21(a)) with pucks made of Polytetrafluoroethylene (PTFE) sliding against a grade 8 mirror finish stainless steel surface embedded in a concrete slab. The construction details of the base isolation devices are shown in Figure 21(b), and four devices were arranged on the shake table following Figure 21(c). The shake table test results showed that the isolation system can provide excellent protection to the superstructure and contents resulting in no observable damage throughout 31 tests under full-intensity ground motions.

NUMERICAL MODELLING APPROACH OF WOOD-FRAMED SHEAR WALLS

Since the bracing walls are the most important components of LTF structures to resist seismic loads, accurate wall modelling is the essential part of their seismic simulation. The racking model of timber shear walls can be broadly classified into two main categories, analytical models, and numerical models. Analytical models are mathematical models that have a closed-form solution. This kind of model of timber shear walls is adopted by many design standards to build up the relationship between the bracing capacity and deformation. As the Engineering Basis of NZS3604 [44], the lower bound of elastic modulus from NZS3603 [45] is used for calculating the deflections and limiting plate loads of bracing walls. This originated from the plastic lower bound model, a classic analytical wall model developed by Neal [46]. Numerical methods with hysteresis wall models are more suitable for full-structure nonlinear dynamic analysis to simulate seismic responses. There are two types of numerical modes for LTF structures: detailed finite element models and macro element models.

Detailed Finite Element Models

The detailed models consider almost all structural components by modelling most parts of the timber shear walls, normally including beam elements for frame members, shell/plane elements for sheathing panels, and spring elements for connections. The spring models for connections are the key parts of the detailed model. Specific hysteretic models of these springs are required for nonlinear dynamic analysis, accounting for hysteretic damping and strength/stiffness degradation. There are various types of hysteretic models for timber connection: mechanics-based models, empirical models, and mathematical models.

Mechanics-based models rely on the basic material properties of the fastener and the embedment characteristics to model the connection's hysteresis [47,48]. Empirical models fit the connection's hysteresis using a combination of linear segments or curves for loading and unloading paths obtained from testing. Several empirical models for timber connections have been proposed, including the bilinear model [49], the trilinear model [50], and the damage-considered model by Wen [51]. Mathematical models do not directly rely on mechanical properties but on physical understanding of the hysteretic system. The Bouc-Wen model [52] is one widely used mathematical model for the hysteretic behaviour in civil and mechanical engineering, and Foliente [53] modified this model to characterize the general features of the hysteretic behaviour of wood connections and structural systems, which is known as the Bouc-Wen-Baber-Noori (BWBN) model.

The following four detailed wood shear wall models represent different detailed modelling techniques. A finite element computer program named WANELS was developed by Gutkowski and Castillo [54–56] for the analysis of single- and double-sheathed wood shear walls under static loads. In this program, the sheathing panels are modelled by two-dimensional orthotropic-plane stress elements. The nailed connections between the frame and sheathing are modelled by nonlinear nondimensional spring elements, and the joints between the frame members are modelled by linear spring elements. This program can also examine the nail forces and their distribution, and failed nails can be removed automatically to trace progressive failure. For the lateral nail resistance, this program uses a solution combining the advantages of the variable stiffness method and the load correction method. The related parameters are determined by a stepwise approximation to either nonlinear lateral nail resistance data or a chosen empirical relationship. This model can predict the load-displacement relationship of target walls with a high degree of accuracy well into the nonlinear range. Figure 22 shows the comparison between the simulation results of the program WANELS and ten experimental results on a wall only sheathed by plasterboards on one side.

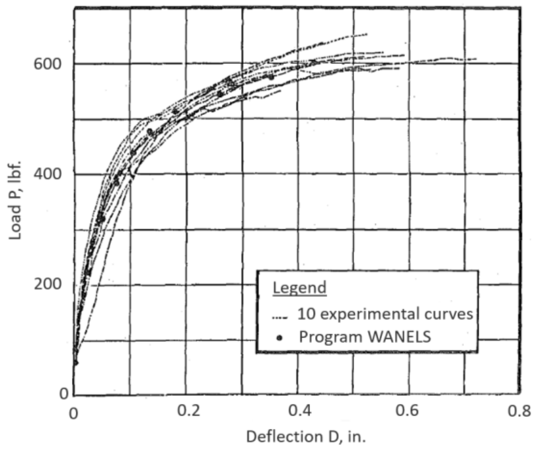

Figure 22: Comparison between the simulation results of program WANELS and ten experimental results on a wall only sheathed by plasterboards on one side [56].

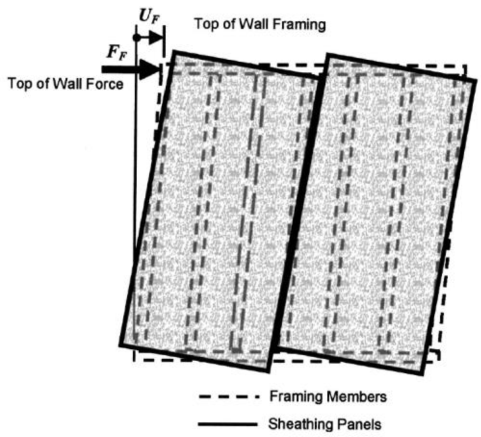

Figure 23: Assumption of racking mode for wood-framed shear walls [27].

A simpler numerical model was developed by Filiatrault and Folz [27,57] to predict the response of timber shear walls under cyclic loads. The model was composed of three types of structural components: rigid framing members, linear elastic sheathing panels, and nonlinear sheathing-to-framing connections. The racking mode of wood-framed shear walls was assumed to be as shown in Figure 23. The framing members were assumed to be rigid with pinned connections, so the wall frame alone has no lateral stiffness. The out-of-plane deformation of sheathing panels was ignored as it is a two-dimensional wall model. Each rectangular sheathing panel developed a uniform in-plane shear deformation, superimposed on horizontal and vertical rigid-body translations and rotations. The relative displacements between sheathing and framing resulted in inelastic deformations at the sheathing-to-framing connections. Previous studies support these assumptions [58,59]. An empirical hysteretic model for sheathing-to-framing connections, originally proposed by Foschi [60], was modified to minimize the path-dependent rules. The force-displacement response of connections under monotonic and cyclic loading is shown in Figure 24. The wall model was verified against tests of wood-framed shear walls and has been incorporated into the computer program CASHEW (Cyclic Analysis of Shear Walls).

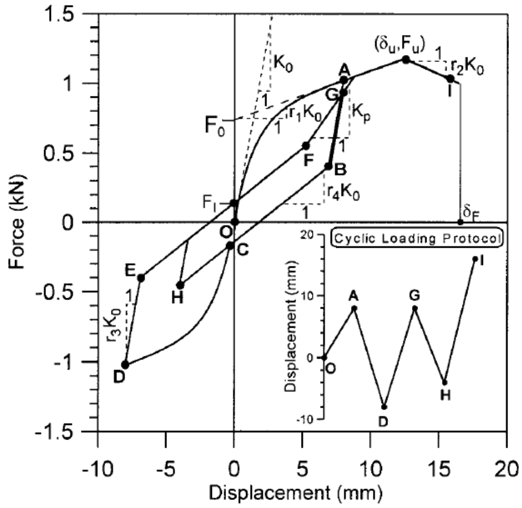

Figure 24: Force-displacement model of sheathing-to-framing connections under monotonic and cyclic loading [57].

Pang and Hassanzadeh [61] claimed that the simplified assumptions in the CASHEW program limit its applications to modelling engineered and fully anchored shear walls only. A new detailed model using the nodal condensation technique was developed [61] and coded into a computer program named M-CASHEW. As shown in Figure 25, this model employs three types of connection elements to model the partial composite action between the frame and the panel, including panel-to-frame, frame-to-frame, and panel-to-panel connections. Each connection is represented by a 2-node 3-DOF (degrees of freedom) connection element with three orthogonal uncoupled springs. Each of the orthogonal springs can be assigned the properties of one of the seven elastic and hysteretic spring models available in M-CASHEW. By combining the M-CASHEW wall and diaphragm element models, a global 3D platform named Timber3D was developed by Pang et al. [62] for nonlinear time history analyses of light-frame wood buildings. In Timber3D, the frame elements are modelled individually as 12-DOF elastic elements with pinned ends. Horizontal loads are resisted by shear springs that span between floors and represent the behaviour of the wall elements.

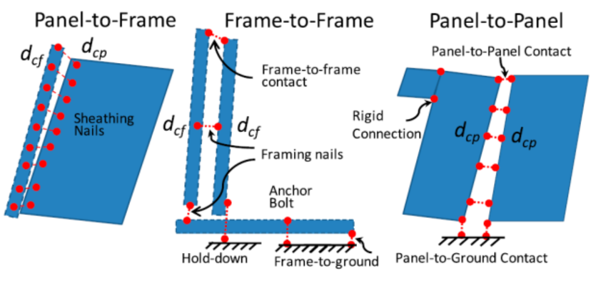

Figure 25: Connection elements of the detailed model [61].

Christovasilis and Filiatrault [63] pointed out that most of the previous wall models did not consider the rocking and uplifting deformation among frame members and connections to the diaphragms, which could reduce the wall strength and stiffness. They developed a shear wall sub-structure model to predict the lateral stiffness, strength, and energy dissipation capacity. Their model is configured to require a smaller number of DOFs to make the wall model more efficient for the global analysis of complete buildings. In this model, the sheathing panels are described with 4 DOFs and sheathing-to-framing connections are described with two orthogonal independent phenomenological springs. Similarly, the frame members are represented with 2-noded elastic beam elements. Figure 26 illustrates the numerical model of the framing domain. As can be seen in this figure, contact springs are introduced to model the framing-to-framing and framing-to-floor connections, and the horizontal forces between the top plates and diaphragms are transferred through the master nodes (normally at the centre of the top plate). It enables modelling of the uplifting response without consideration of geometric nonlinearity.

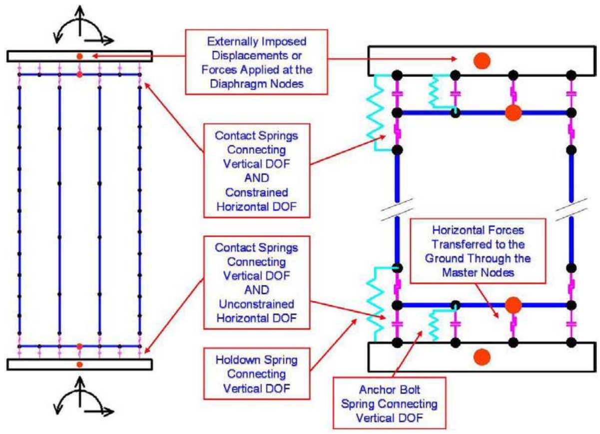

Figure 26: Detailed numerical model of the framing domain [63].

Macro Element Models

Although the finite elements based numerical models can capture the detailed behaviour of framing members, nails, and sheathing panels, they are often not computationally efficient to simulate the entire building. For the whole building simulations, the focus is on the overall wall performance or storey drift rather than the specific responses of the individual components. For this purpose, macro element models have been developed to simplify the wall models in whole building analyses.

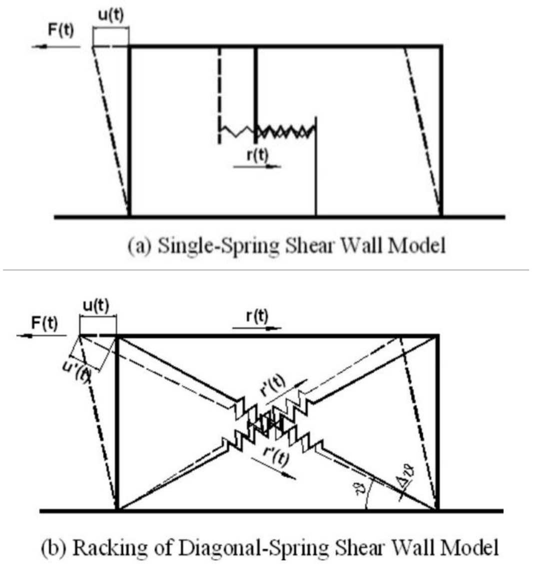

Figure 27: Examples of the macro element model [64].

A typical macro element model consists of three rigid truss elements (acting as a frame) and one or two nonlinear springs, where the springs represent the nonlinear behaviour of sheathing-to-framing connections, which mostly govern the nonlinear wall behaviour. Figure 27 shows two examples of the macro element model [64]. The wood-frame shear wall is simplified to single horizontal shear-springs or diagonal-spring elements. Chen et al. [65] developed a modified macro element model that also accounts for the wall rotations. As shown in Figure 28, in this model two vertical springs are added to the bottom of the rigid truss element and pinned to the ground.

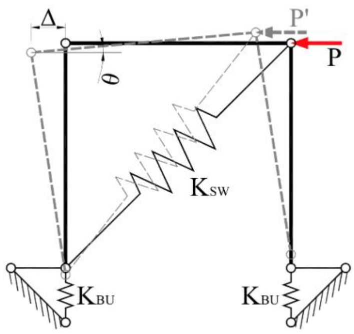

Figure 28: A modified macro element model [65].

The definition of the hysteretic rule of these springs is the key to the reliability of macro element models in predicting nonlinear dynamic response. The three types of hysteretic models introduced above (mechanics-based models, empirical models, and mathematical models) are also applied to the macro springs. The hysteretic parameters of the springs can be obtained from the detailed shear wall models or shear wall test data. As an example, Filiatrault and Folz [27] proposed a three-dimensional nonlinear pancake model for LTF buildings. In this system, each wall is modelled by a single zero-height nonlinear in-plane shear spring using Stewart's empirical hysteresis model [50], as shown in Figure 29.

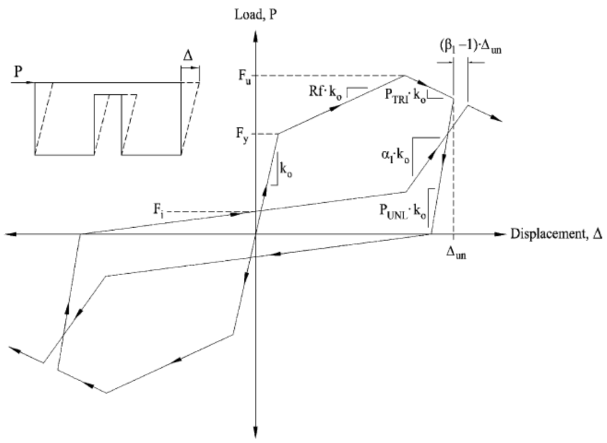

Figure 29: Stewart's hysteresis model [50].

A macro spring model for shear walls has also been derived by modifying the individual nail connection model [64,66]. This approach was based on the fact that the global hysteretic behaviour of LTF shear walls is similar to that of nail-to-wood connections, including strength/stiffness degradation and pinching effect. The "pseudo-nail" model is a typical macro wall model developed by Li et al. [66], revised from a nail connection model named HYST [47]. HYST is a common panel-frame nail connection model used in wood shear walls. Figure 30(a) illustrates the schematics of HYST. A modified HYST algorithm, developed by Li et al. [48], improved the computational efficiency and addressed the stiffness degradation effect. Figure 30(b) shows the loading and unloading of wood medium in the modified HYST algorithm. The parameters in this model include the nail length L, nail diameter D, and six parameters to describe the compressive properties of the surrounding embedment medium. These parameters can be calibrated by shear wall test data or detailed wall models. The "pseudo-nail" wall model was incorporated into a computer-based structural analysis tool called ''PB3D" developed by Li et al. [66]. ''PB3D" is an efficient three-dimensional analysis platform for nonlinear time history analysis of residential post and beam timber buildings under seismic loads. In this platform, the diaphragms are modelled by beam elements and diagonal truss elements considering the in-plane stiffness, and beams and posts are modelled by elastic beam elements. The uplifting is simply prevented by wall post elements which are fully end-restrained onto the foundation or stories. Figure 31 shows the schematics of a PB3D model.

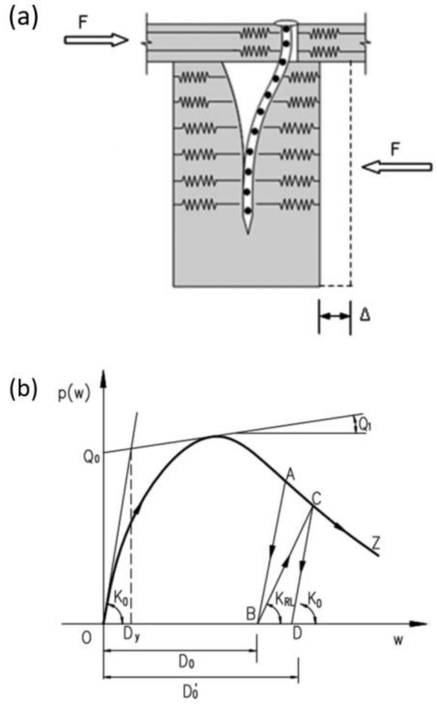

Figure 30: HYST algorithm: (a) schematics of HYST panel-frame nailed connection; (b) loading and unloading of wood medium in modified HYST algorithm [48].

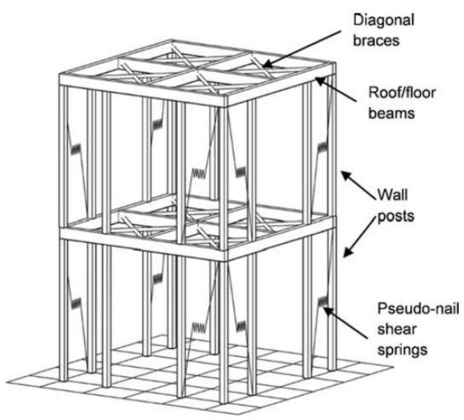

Figure 31: Schematics of a "PB3D" model [66].

SIMULATION STUDIES ON SEISMIC PERFORMANCE OF LTF RESIDENTIAL STRUCTURES

A closed-form racking model was developed by Liu and Carradine [3] for NZ plasterboard walls based on the racking test results of 12 plasterboard walls. It was assumed that the total lateral deformation of the wall is the sum of the flexural deformation and the equivalent shear deformation that accounts for all other sources of deformations including sheathing panel shear, connection slip, and hold-down uplift. The expressions are as follows:

$$\Delta_{total} = \Delta_{flexural} + \Delta_{uplift} \tag{3}$$

$$\Delta_{flexural} = \frac{2VH^3}{3EA_C L^2} \tag{4}$$

$$\Delta_{uplift} = \frac{VH}{G_e L t_e} \tag{5}$$

where V is the racking load at the top of the wall, H is the height of the wall, E is the modulus of elasticity of timber chords, A_c is the area of chords, L is the length of the wall, t_e is the total thickness of plasterboard sheathing, and G_e is the equivalent shear modulus. The key parameter was the equivalent shear modulus at different deflection levels, and it was determined based on P21 wall test results. This racking model can reasonably capture the skeleton curve of the plasterboard walls. However, the model was calibrated based on limited wall configurations of the test specimens and did not consider the effect of strength degradation.

Based on the in-situ tests on post-quake houses after the 2010-11 Canterbury earthquake sequence, a single-degree-of-freedom nonlinear model was developed by Morris et al. [40] for the 1923 tested house as a case study. The load-deflection hysteresis of this numerical model was calculated by the HYST model and calibrated based on the push and pull test results. Figure 32(a) illustrates the hysteresis curves of test results and the calibrated HYST model. After that, time history analyses were conducted using 19 ground motions from the 2011 Christchurch earthquake. The original records were scaled to two design seismic levels (500- and 2500-year return periods). Figure 32(b) illustrates the cumulative distribution of peak displacement responses. It showed the average peak displacement at the 500-year return period design level was only 9mm (0.4% drift ratio), which meant this post-quake structure would still perform well at this seismic level.

A series of building models were developed in 3D ETABS by Liu and Shelton [67] to evaluate seismic effects of permissible irregular distribution of bracing resistance within the scope of NZS3604. Six case study single-storey LTF houses braced by plasterboard bracing walls were designed according to NZS3604 and modelled. The first three houses had the same rectangular identical floor plan but different irregularity levels (0, 50%, and 100% respectively). The structural irregularity was caused by irregular arrangements of bracing walls. In the first building (i.e. 0% irregularity), the bracing arrangements were perfectly regular and there was no irregularity. “100% irregularity” meant that the bracing arrangements were very irregular and reached the specified limits in NZS3604. Similarly, “50% irregularity” meant the bracing arrangements were between “100% irregularity” and perfectly regular, where the minimum bracing capacity of each bracing line is 75% of the total bracing demand divided by the number of bracing lines. The floor plans of rectangular case study houses are illustrated in Figure 33. The other three houses were L-shaped in plan. They also shared the same outline but different irregularities, 0%, 50% and 100% respectively. In the 3D ETABS models of these houses, plasterboard bracing walls and plasterboard ceiling diaphragms were modelled as shell elements. Then equivalent static push-over analyses were conducted in the Y direction. The results showed that the extremely irregular bracing arrangements as allowed by NZS3604 could result in a significant increase in the maximum lateral deflections compared to the houses with regular bracing wall arrangements, approximately 5 times for rectangular cases and 3 times for L-shaped cases. Taking the rectangular case study houses as an example, the max drift of the regular arrangement case was 0.31% while that of the 100% irregularity case was 1.58%. Besides, it was found that the effect of irregularity on the fundamental periods (T_1) was not significant. T_1 of rectangular case study houses was about 0.2 to 0.3s, and T_1 of L-shaped houses was about 0.45 to 0.55s.

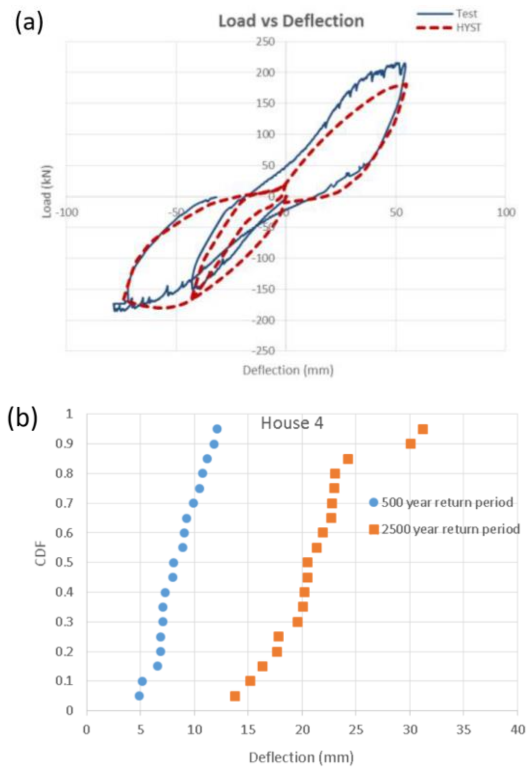

Figure 32: Single-degree of freedom nonlinear model based on HYST model for a case study building [40]: (a) hysteresis curves; and (b) cumulative distribution of peak displacement responses.

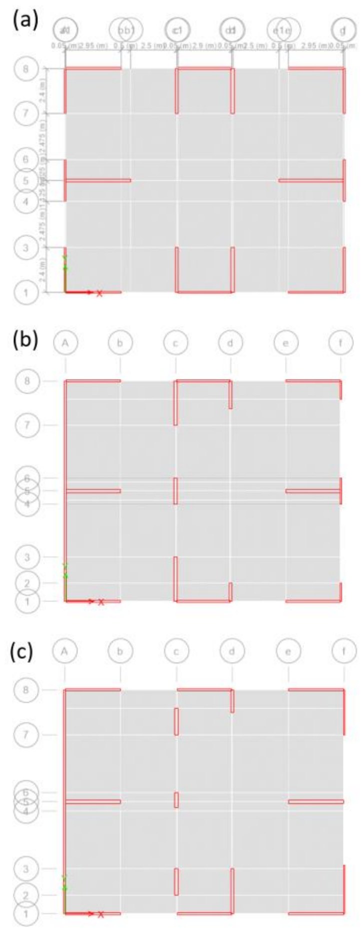

Figure 33: Floor plans of single-storey rectangular case study houses [67]: (a) regular; (b) 50% irregularity; and (c) 100% irregularity.

Ma et al. [68] conducted a parametric study for quantifying the effect of different levels of bracing wall irregularity as well as the rigidity of ceiling diaphragms on the seismic performance of LTF houses. Three groups of single-storey baseline houses with different bracing wall layouts were designed per NZS3604. Within each group, three levels of bracing wall eccentricity were designed including a symmetric layout, 50%, and 100% of the specified irregularity limit (same as the definitions in [67]). The floor plans of the baseline houses are shown in Figure 34. All bracing walls were sheathed with plasterboards on one side and had no hold-downs, and the diaphragm system was the GIB Rondo branded ceiling diaphragm. Rayleigh damping model was used, and the damping ratio was assumed to be 5% according to NZS1170.5. The numerical modelling was conducted in the "PB3D" simulation platform, and the "pseudo-nail" model was used for the bracing walls as introduced in the last section. A suite of historical earthquake ground motions from the 2010–2011 Canterbury earthquake sequence was used for time history analysis. These records were scaled to match their mean 5% damped spectral value over a period range of 0.1–0.56 s with the design spectra: 0.9 g spectral acceleration for the ULS level and 0.225 g spectral acceleration for the SLS level. The results showed that, in the baseline houses with rigid diaphragms and bracing walls of limit irregularity allowed in NZS3604, the maximum drift response was about three times that in houses with symmetric bracing wall layouts. This means that the allowed irregularity of the bracing wall layout in NZS3604 may cause significant torsional effects and excessive damage.

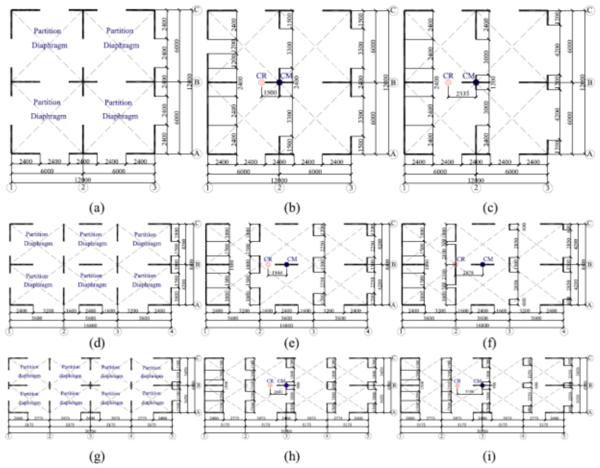

Figure 34: Three groups of single-storey baseline houses with different bracing wall layouts [68].

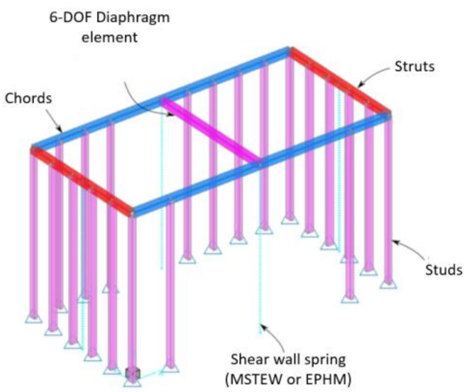

Figure 35: Timer3D model for test building [42].

A numerical model was developed in Timber3D by Francis et al. [42,43] for the LTF building introduced in the experimental studies section. Figure 35 illustrates the framing and wall elements of the Timber3D model. Based on this model, the modal analysis, pushover analysis, and time history analysis were performed. 2% and 5% Rayleigh damping were used in the time history analysis to compare the effect of different damping values. Figure 36 illustrates the displacement response of test results and models using 2% and 5% damping under three New Zealand earthquake records. It was concluded that the model using 5% damping provided a better prediction of the displacement response of the test building. This agrees with the damping approach used by Ma et al. [68].

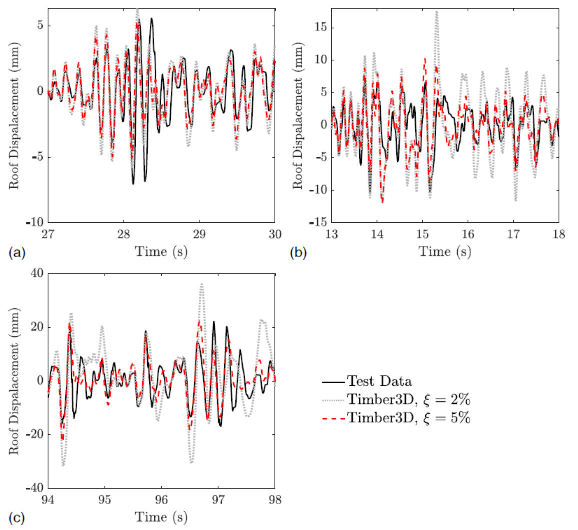

Figure 36: Displacement response of test results and models using 2% and 5% damping [42] under earthquake records of (a) Darfield 2010, (b) Lyttleton 2011, (c) Kaikōura 2016.

SEISMIC DAMAGE AND LOSS MODELS OF NEW ZEALAND RESIDENTIAL HOUSES

Seismic Damage Incurred by LTF Residential Houses

After the Mw 7.1 Darfield earthquake on 4 September 2010, Beattie et al. [8] conducted a post-earthquake damage survey on residential houses near Christchurch. Cracks on plasterboards in lower storey walls were observed in some houses, including diagonal cracks that emanated from the top corners of large door openings (Figure 37(a)) and vertical cracks on the joints at the corners of openings (Figure 37(b)). The survey also found that the "L" and "U" shaped houses suffered greater (no serious) damage at the intersection of the wings.

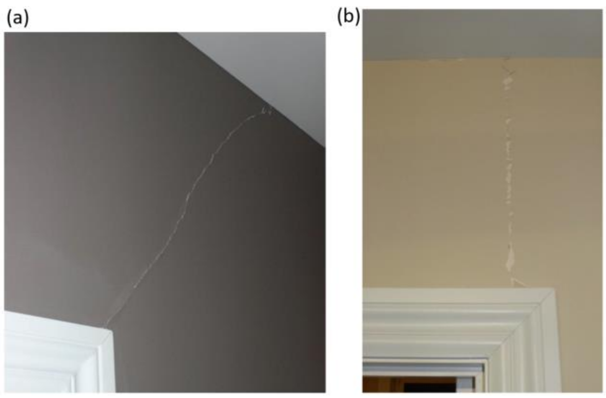

Figure 37: Plasterboard bracing walls damage after the 2010 Darfield earthquake: (a) diagonal cracking of plasterboard; (b) vertical cracking of plasterboard [8].

The 2011 Christchurch earthquake was an Mw 6.3 aftershock of the 2010 Darfield earthquake. Although it had a lower magnitude, the epicentre was closer (approximately 6km southeast) to Christchurch City [69]. Many buildings in the CBD were severely damaged and some were even demolished. Buchanan et al. [5] reviewed the performance of houses after this earthquake and found that LTF houses generally performed well to meet the life safety performance target. Most timber houses envelopes and diaphragms successfully maintained structural integrity. Minor damage such as cracks in plasterboards was typically observed in the houses and some houses suffered more severe damage in bracing walls. Figure 38 illustrates a case where the plasterboard was completely detached from the wall frame.

Figure 38: A severe failure of the plasterboard internal linings after the 2011 Christchurch [5].

Seismic Loss Models

Researchers in New Zealand have also made efforts on developing building seismic loss models for New Zealand earthquakes in the last century. Damage ratios (i.e. ratios of damage repair cost to the building replacement cost) have been evaluated for different building types in several high-intensity earthquakes including the 1931 Hawke's Bay earthquake [70], the 1968 Inangahua earthquake [71], and the1987 Edgecumbe earthquake [72,73]. These loss models can be categorized as empirical models, and the main basis of these studies were insurance claim data. Dowrick and his team [70–74] catalogued and categorised the damage to almost all building types in the higher Modified Mercalli intensity (MMI) zones and were therefore able to relate the distribution of damage ratio to the intensity level for many classes of building and their contents. The damage ratio, Dr, is used to express the degree of damage to any class of property at risk, and it is defined as:

$$D_r = \frac{Cost\ of\ Damge\ to\ Property}{Value\ of\ Property} \tag{6}$$

In this equation, the Value of Property is defined variously in the literature and could be the replacement value, market value, indemnity value, or insured value. The damage ratios are studied as functions of the intensity of ground motion and are related to the MMI isoseismals. The damage ratios for low-rise New Zealand buildings have been estimated by Dowrick et al [71,74] and they are modelled as:

$$\overline{D_r} = A \times 10^{(\frac{B}{MMI-C})} \tag{7}$$

where $\overline{D_r}$ is the mean damage ratio, MMI is the shaking intensity, and A, B, and C are constants. The relationships between the intensity and damage ratios of four common building types are shown in Figure 39. These functions are based primarily on the New Zealand data for intensity zones MM5 to MM7 and a combination of New Zealand and United States data [75] for zones MM8 to MM10. When considering the uncertainty of damage ratios, the shape of the statistical distribution of non-zero damage ratios for various classes of property at each intensity level is found to be well approximated by a lognormal distribution [71].

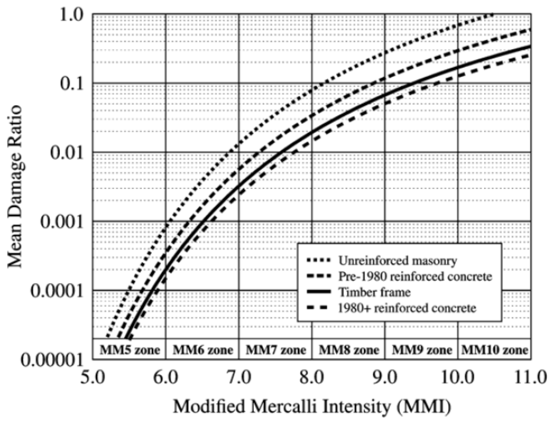

Figure 39: Mean damage ratios of low-rise New Zealand buildings in Dowrick's damage model [76].

Since Dowrick's loss models were developed based on damage data in the last century, these models did not include modern construction types and may not align with current economic conditions. Horspool et al. [7] updated the damage and loss model for residential houses based on the data during the 2010-2011 Canterbury earthquake sequence. The building information comes from the New Zealand Earthquake Commission (EQC) which is a government entity that provides natural disaster insurance to residential properties, covering damage to buildings, contents, and some coverage of land. The damage and loss data were from the EQC Claims database (providing almost total coverage of claims from natural disaster events), and the undamaged buildings' information was from the EQC portfolio database (a national building level database for every residential building in New Zealand). Then, the collected data at each intensity level were fitted to a four-parameter inflated beta distribution [77] to the damage ratios for each typology class. Figure 40 illustrates the mean damage ratio curves for LTF residential houses built pre-1940s (TWL5), 1940-1980 (TWL7), and post-1980s (TWL9). This work has been expanded from the MMI related functions to the peak ground acceleration (PGA) related functions, as shown in Figure 41.

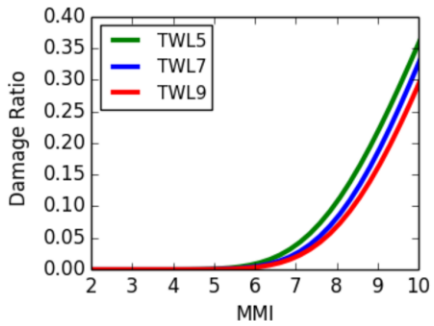

Figure 40: Vulnerability models [7] for mean damage ratios for LTF residential house built pre 1940s (TWL5), 1940-1980 (TWL7), and post-1980s (TWL9).

The Pacific Earthquake Engineering Research (PEER) Centre developed a clearly defined loss estimation framework [79,80]. The PEER framework consistently accounts for uncertainties in the relationships between earthquake hazard, structural response, seismic damage, and economic loss. This process, as shown in Figure 42, is separated into four probabilistic expressions combined using conditional probabilities to account for the uncertainties in the relationships between different parameters. Following the PEER framework, a seismic performance assessment methodology was proposed in FEMA P-58 [81], and a fragility database was established, consisting of damage state definitions, related repair costs, and repair time for most structural and non-structural components.

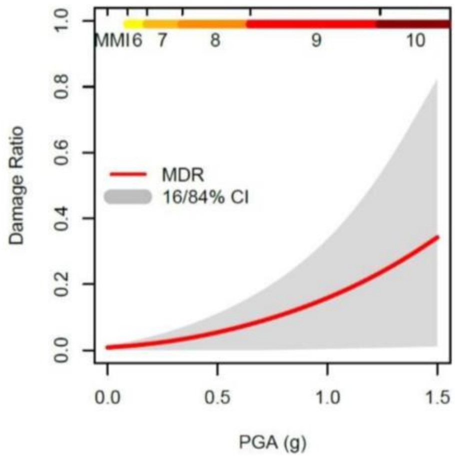

Figure 41: Vulnerability models from [7] for mean damage ratios verse PGA for LTF residential house [78].

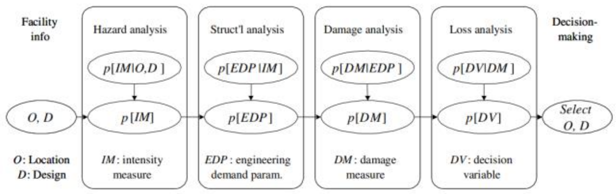

Figure 42: Seismic loss estimation framework proposed by PEER [80].

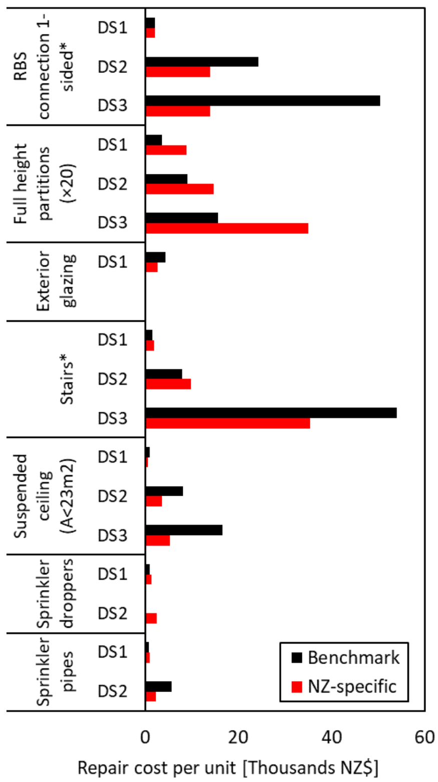

Figure 43: Comparison of the repair cost between FEMA P-58 (referred to as "benchmark") and the NZ-specific database developed in [82]. (*) Damage state numbers refer to the benchmark consequence functions.

However, adapting the overseas data from the FEMA P-58 database to the New Zealand context may be challenging because New Zealand has a different building practice for residential houses, different costs of materials and labour, and different repair methods. A database of New Zealand consequence functions was developed by Fox et al. [82]. In accordance with FEMA P-58, this database included the damage states, repair cost, and repair time of most components in common RC and steel frame buildings. The repair costs were collected by cooperating with a local construction company, and the 10th, 50th, and 90th percentile values of the distribution were estimated to account for the uncertainty. This dataset is freely available on the NEHRI Design Safe website [83]. Figure 43 illustrates the comparison of the repair cost between FEMA P-58 (referred to "benchmark") and the NZ-specific database. It shows that there is a clear difference between them with a ratio of benchmark to NZ-specific repair varying from 0.25 to 2.5. To analyse the impact of the NZ-specific consequence data on the expected annual losses, a case study of a 12-storey steel frame building designed by Yeow et al. [84] was re-examined by using the Seismic Loss Assessment Tool (SLAT) [85]. The hazard model adopted here was a New Zealand-specific rupture forecast model by Stirling et al. [86] and ground motion models by Bradley [87] for spectral acceleration at 2.0s. Figure 44 illustrates the expected annual losses conditional on intensity including and excluding collapse cases. The results show that, at lower intensities, the losses were larger when using NZ-specific consequence functions, but at higher intensities the FEMA P-58 consequence functions result in larger losses.

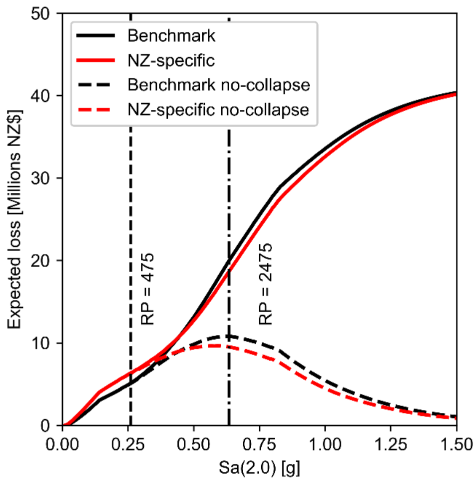

Figure 44: Expected loss conditional on intensity of the case study building (with collapse cases both included and excluded) [82].

It is important to note that the database developed in [82] did not include the common elements in LTF structures, so further contribution needs to be made for adding damage and loss functions of LTF shear walls, ceilings, roofs, etc. An experimental study about damage state quantification of LTF walls was conducted by Liu and Carradine [88]. A quasi-static cyclic test was conducted on a full-scale LTF wall and floor system. The bracing walls were arranged symmetrically in both directions and the loading was applied in the short-side direction, as shown in Figure 45. The walls were sheathed by 10mm thick standard plasterboard on the inside and 9mm thick F8 grade plywood sheets on the exterior and had hold-downs. The following damage was observed. Tearing of tapes between walls and ceiling initiated along the joint lines when storey drift was 0.36%. When the drift reached 0.72%, local plasterboard cracks occurred at the bottom corner of the walls along the loading direction. Noticeable load degradation occurred at 1% drift, when plasterboard damaged locally at the bottom corner (Figure 46(a)) and the vertical sheet joints of plasterboard failed

(Figure 46(b)). When the drift reached 1.45%, out-of-plane bucking of plasterboards occurred on the walls along the loading direction (Figure 46(c)). The testing was terminated following the ±60mm actuator displacement (2.5% drift) cycles because of significant wall damage and reduction of the applied loads. Based on the damage observation on the test structure, the damage state definitions for LTF plasterboard bracing walls were summarised, as reproduced in Table 3. These definitions provide a meaningful linkage among the damage states, storey drifts, and potential repair actions.

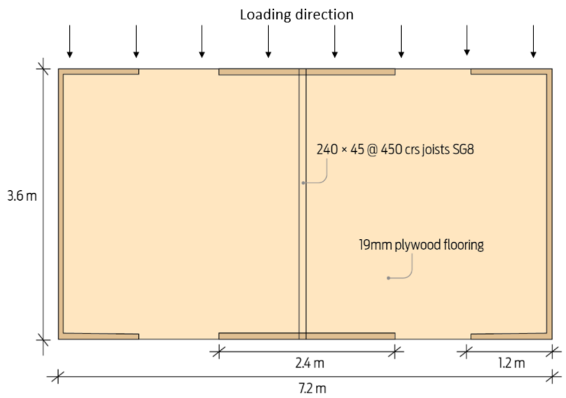

Figure 45: Floor plan of the test structure [88].

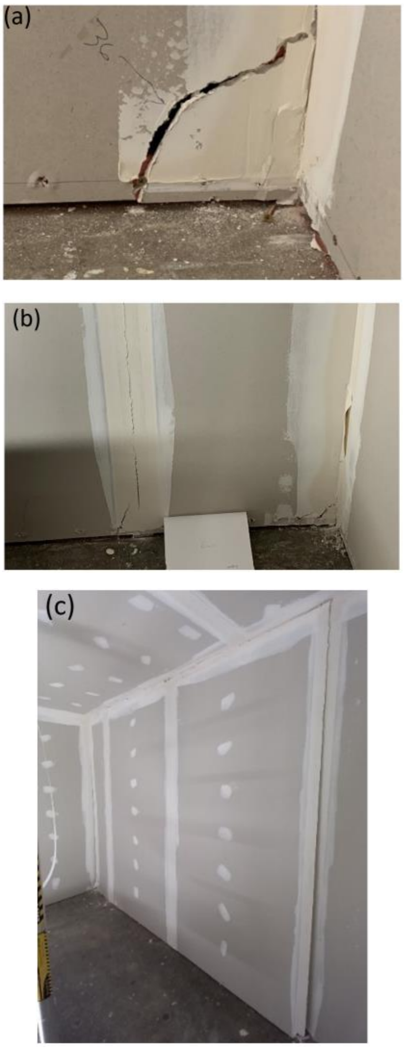

Figure 46: Damage observations of the test structure [88]: (a) local plasterboard damage; (b) failure of vertical sheet joints of plasterboard; (c) out-of-plane bucking of plasterboard.

Table 3: Damage state definitions proposed in [88].

Damage state	Description	Potential repair action	Storey drift
1	Tape wrinkling, screw distress, tape tearing in isolated area	Retape, repair stressed screws and paint	≤ 0.6%
2a	Plasterboard damage - local crushing or cracking within sheets	Replace damaged plasterboard, retape the joints and paint	0.7%
2b	Plasterboard sheet joints opening	Replace the damaged plasterboard, reinstall the screws, retape and paint	1%
2c	Plasterboards detached from the framing, Out-of-plane buckling of plasterboards	Replace the board, refix the boards to frames, retape and paint	1.5%
3	Plasterboard significantly damaged and uneconomical to repair	Repair is uneconomical. Demolition is required	1.8%

SUMMARY AND FUTURE RESEARCH

This paper thoroughly reviewed recent research on seismic performance assessment of New Zealand LTF residential houses and development of seismic loss models for New Zealand houses. It introduced the evolution of New Zealand residential construction with the focus on seismic load-resisting systems, characteristics of plasterboard bracing walls, and experimental as well as simulation studies on the seismic performance of plasterboard bracing walls and LTF houses. The seismic damage incurred by LTF residential houses in the 2010-11 Canterbury Earthquake sequence is summarised and the existing New Zealand seismic loss models are introduced. Based on the review presented herein, the conclusions and future research directions can be summarised as follows:

The plasterboard bracing walls used in modern New Zealand LTF residential houses are unique bracing systems. Overseas and New Zealand research concluded that the seismic performance of plasterboard bracing walls is clearly different from that of the shear walls sheathed by wood-based panels, with lower ductility, lower strength, lower energy dissipation, and smaller ultimate displacement.

According to the experimental results and post-earthquake observations, typical failures of plasterboard bracing walls include disengagement of screws between plasterboard and timber framing around wall corners, plasterboard cracking around screws in the wall corners, diagonal cracking of plasterboards orienting from the openings' corners, bolts of hold-downs pulling out for the walls with hold-downs, and, in some severe cases, out-of-plane bucking even detachment of plasterboards from wall framing. It was also concluded that plasterboard bracing walls are more susceptible to damage such as cracks. However, the relationship between the damage levels and structural responses needs to be further quantified.

There are several well-established numerical simulation methodologies for the hysteretic behaviour of timber shear walls, including detailed finite element methods and macro elements methods. By adapting the existing methods to New Zealand cases, some numerical models have been developed for simulating plasterboard bracing walls and LTF structures in New Zealand. However, greater attention is suggested to give to the up-lift resolution of bracing walls with or without significant axial loads as well as the studies of houses with various irregularities, such as houses with irregular plans and multi-storey houses built on slopes, which are often associated with significant vertical irregularities. The incremental dynamic analysis (IDA) could also be applied to estimate their seismic risks at different earthquake intensity levels.

Experimental tests and numerical simulations showed that the drift limit of 2.5% at ULS specified in NZS1170.5 for general structures is not suitable for New Zealand LTF structures braced by plasterboard bracing walls. It is recommended that drift be limited to 1%, as the plasterboard bracing wall is usually severely damaged and significant strength loss could occur after 1% drift.

Most of the existing seismic loss models for New Zealand are empirical models, developed based on damage data from the post-earthquake surveys. Some attempts were made to build analytical models with NZ-specific loss functions following the PEER seismic assessment framework. However, only a few studies have considered classifying and quantifying seismic damage to plasterboard bracing walls. Further research should focus on developing fragility functions and seismic loss estimations. For this purpose, it is very important to collect NZ-specific information on damage and repair costs.

Lastly, potential avenues for future research and exploration are outlined below:

1. A more conservative design drift limit could be suggested for the plasterboard bracing wall systems to reduce seismic damage in major earthquakes. Reasonable drift limits can be determined based on experimental observations and numerical simulation results. Economic benefits, social impacts and industry acceptance may also be considered when establishing the new drift limits in design standards.
2. A seismic economic loss hazard model could attract wider attention and raise awareness of the economic risk associated with the current bracing system. Developing a comprehensive framework for seismic loss estimation specifically for LTF residential houses in New Zealand would also be highly beneficial for this purpose.
3. Methods to mitigate seismic vulnerability of the plasterboard bracing walls need to be explored to improve their resilience. Such methods could involve the use of different types of sheathing panels, different fastener types, and the addition of hold-downs. Given these measures often lead to higher construction cost, it is essential to clearly articulate the full life-cycle economic benefits to inform decision-making. Calculating the expected annual loss of retrofitted structures, based on seismic loss estimation, can help assess the viability of various mitigation techniques.
4. A deeper study on the effects of bracing irregularities of LTF residential houses is also recommended. In the past major earthquakes, it was often observed that damage could be exacerbated by horizontal irregularities (with irregular floor plan and bracing wall layouts), vertical irregularities (using different bracing system on the first and second storeys), as well as houses built on slopes. More specific design guides could help these irregular structures improve their seismic performance and reduce earthquake damage.

ACKNOWLEDGMENTS

The authors would like to acknowledge the research grant from Resilience to Nature's Challenges. The authors would also like to thank PhD candidate Zhaoyu Ye for recommending literature on this topic.

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