
Seismic Performance of Fire-Rated Partition walls with “No-Fix Zones” details in New Zealand industry practice

J. Bhatta, R. Shrestha, R.P. Dhakal, T.J. Sullivan, & G. Roux

University of Canterbury, Christchurch, New Zealand.

M. Hosseini, & A. Shegay

University of Auckland, Auckland, New Zealand.

M. Browne, S. Oliver, & M. Inwood

Holmes, Christchurch, New Zealand

L. Pledger

Zürich, Switzerland

ABSTRACT

Two full-scale wall specimens, each 3.55 m high, one timber-framed and the other steel-framed, were constructed between reinforced concrete slabs on a 6 m × 3.50 m plan, following standard New Zealand industry practice with fire-rated plasterboard linings. Specimens were subjected to floor motion records representative of a six-storey reinforced concrete building model excited by scaled El Centro ground motion records. Lateral movement at the top of the walls was accommodated in-plane through a deflection head track (DHT), while out-of-plane flexibility at wall junctions was achieved through deformation of the timber top plates in the timber wall and by the DHT in the steel wall. DHT anchors for the top plates were intentionally offset 1.20 m from wall junctions to create “No-Fix zones,” allowing the intended flexibility.

The results showed that the timber-framed walls could preserve their fire integrity up to 0.60 % drift and exhibited top-plate damage at 1.00 % drift. The steel-framed wall with “fixed” ends outperformed timber-framed walls, sustaining minor damage (leading to partial loss of fire integrity) at 1.80 % drift and critical damage (separation of walls at junctions) at 2.50 %, far exceeding the 0.60 % median capacity of traditional steel walls. Displacement readings also confirmed notable out-of-plane flexibility at “No-Fix zones” junctions. These results advocate for flexible top-track detailing to enhance drift capacity of walls. However, they also reveal potential deformation incompatibilities

with other non-structural elements, indicating a need for further investigation into interaction effects for improved seismic resilience.

1 INTRODUCTION

In New Zealand, current industry practice constrained by limited evidence regarding the seismic performance of partition wall detailing, often results in the overly conservative design of very stiff structures. This approach increases construction costs and embodied carbon without necessarily delivering proportionate performance benefits. The New Zealand building industry currently lacks sufficient empirical data on the performance of plasterboard lined partition walls under seismic loading conditions, especially for displacements exceeding the suggested 0.33 % inter-storey drift for serviceability limit state in Table C1 of AS/NZS 1170.0 (2002). As a result, design teams face uncertainty when specifying these systems in buildings.

Utilizing University of Canterbury's Structural Engineering Lab facility our research team was able to gather critical performance data on existing and novel practices for partition walls under seismic conditions for multi-unit residential and commercial buildings to address this knowledge gap. Such data has the potential to inform code revisions, support more cost-effective and sustainable building designs, provide suitable design options for and reduce unnecessary material use in Low-Damage Seismic Design (LDSD) projects (MBIE and NHC, 2024).

As a part of this experimental campaign, three phases of tests were conducted (Bhatta, 2026):

- **Phase I:** Evaluate the seismic performance of typical partition walls with “No-Fix Zones”. This phase investigates the seismic performance of these partition walls under simulated seismic demands of displacement and acceleration.
- **Phase II:** Evaluate the combined seismic performance and interaction of typical partition walls (with “No-Fix Zones” details) with perimeter-restrained concealed plasterboard ceilings and mechanical pipes, under simulated seismic demands of displacement and acceleration.
- **Phase III:** Evaluate the seismic performance and interaction of novel “rocking” partition walls with fully floating plasterboard ceilings and mechanical pipes, under simulated seismic demands of displacement and acceleration.

This paper presents the Phase I experimental results. In this detailing, connections between the deflection head track (DHT) and the floor above are prohibited at wall junctions. This allows the system to accommodate inter-storey drifts: out-of-plane drift through the bowing of the DHT and top plates, and in-plane drift through the sliding of the wall framing and lining relative to the DHT or floor. Further details can be found in Bhatta (2026).

2 BACKGROUND

Non-structural elements (NSEs), such as partition walls, suspended ceilings, mechanical and electrical systems, cladding, and windows, are essential to a building's functionality and operation. Although these elements are secondary and are either attached to or placed near structural members, they can interact with the structure or with one another during an earthquake, leading to significant damage (Baird et al., 2014; Dhakal, 2010; Dhakal et al., 2011). Such damage often results in costly repairs, operational downtime, evacuations, and even full replacements (Bachman, 2004; FEMA, 1994; Filiatrault, 2002). Suspended ceilings, piping, and partition walls have been identified as major sources of economic loss following seismic events (FEMA, 1994; Filiatrault, 2002).

Lightweight internal partition walls are common in multi-unit residential and multi-story commercial buildings. Typically built with a steel or timber frame, comprising vertical studs, horizontal plates, and

noggings, they are lined on one or both sides with gypsum plasterboard fastened with screws and/or adhesive. The walls are then finished by plastering joints, sanding, and painting.

Experimental research from various regions consistently shows that such walls begin to sustain seismic damage at very low drift levels. The onset of damage, typically occurs between 0.26 % and 0.30 % inter-story drift (Davies et al., 2011; Mulligan et al., 2022; Petrone et al., 2015; Tasligedik et al., 2015; Yeow et al., 2018) with some studies reporting initial cracking as low as 0.05 % (Restrepo and Bersofsky, 2011) or at 0.38 % (Kim and Shin, 2021) underscoring their high vulnerability to even minor shaking.

The 2010-11 Canterbury Earthquake Sequence in New Zealand demonstrated this vulnerability, causing widespread partition wall damage such as screw failures, cracked or crushed plasterboard corners, and dislodged panels (AWCI, 2018; Baird et al., 2014; Dhakal, 2010; Tasligedik, 2014). Damage also occurred during aftershocks that exceeded serviceability limit demands (Arifin et al., 2021). Such early failures often lead to significant economic losses in minor to moderate earthquakes, due to repair costs, replacements, and operational downtime (Bachman, 2004; Bradley et al., 2008). In response, seismic-resilient partition systems have been developed to improve performance. Innovations like seismic relief joints (Tasligedik et al., 2015), flexible tracks (Davies et al., 2011; Mulligan et al., 2020), slotted and bracketed head tracks (Bhatta, Dhakal, Sullivan et al., 2023), and “rocking” walls (Bhatta, Dhakal and Sullivan, 2023a; Bhatta, Dhakal and Sullivan, 2023b; Bhatta and Dhakal, 2026).

In New Zealand, “No-Fix Zone” detailing for partition walls is typically specified, in buildings with specific post-earthquake functionality requirements, when inter-storey drifts are expected to exceed the 0.33 % serviceability limit in the AS/NZS 1170.0 (2002). However, a lack of performance data for these details has led to overly conservative and stiff structural designs. This study investigates the seismic performance of these walls to inform industry practice and future code revisions, aiming to support more cost-effective, sustainable, and materially efficient LDS (MBIE and NHC, 2024) projects.

3 EXPERIMENTAL PROGRAMME AND METHODOLOGY

The methodology involved constructing full-scale timber and steel-framed wall specimens following industry best practices, then subjecting them to incrementally increasing seismic demands (displacements, velocities, accelerations) in laboratory testing. These tests simulated realistic building drift levels, both within and beyond serviceability limit demands, to evaluate damage thresholds, failure modes, and fire integrity. Data was collected via displacement/acceleration measurements, Motion Capture (MoCap) Camera and photographic documentation.

3.1 TEST SETUP

A photo of the test setup and wall specimens are shown in Figure 1. The test setup comprised two 100 mm thick reinforced concrete slabs (3.50 m × 6 m) positioned at the top and bottom of steel columns (200 UC 46.2). The clear vertical spacing between the slabs was 3.55 m. The columns were pin-connected to the slabs, allowing free rotation during lateral movement in the X-direction. The bottom slab was elevated 500 mm above the laboratory floor using concrete blocks. The blocks were mounted on a linear ball bearing and guideway assembly that permitted sliding of the bottom slab in the X-direction. Lateral movement of the slabs was applied through 250 kN actuators with ±500 mm stroke capacity. Both actuators could be operated simultaneously, enabling coordinated movement of the slabs under the prescribed loading protocols. Stability in the Y-direction (perpendicular to loading) was provided by tension-only braces. In the X-direction (loading direction), these braces were removed during testing and reattached once the tests are complete. Refer to Pledger (2026) and Bhatta (2026) for additional information regarding the test setup components.

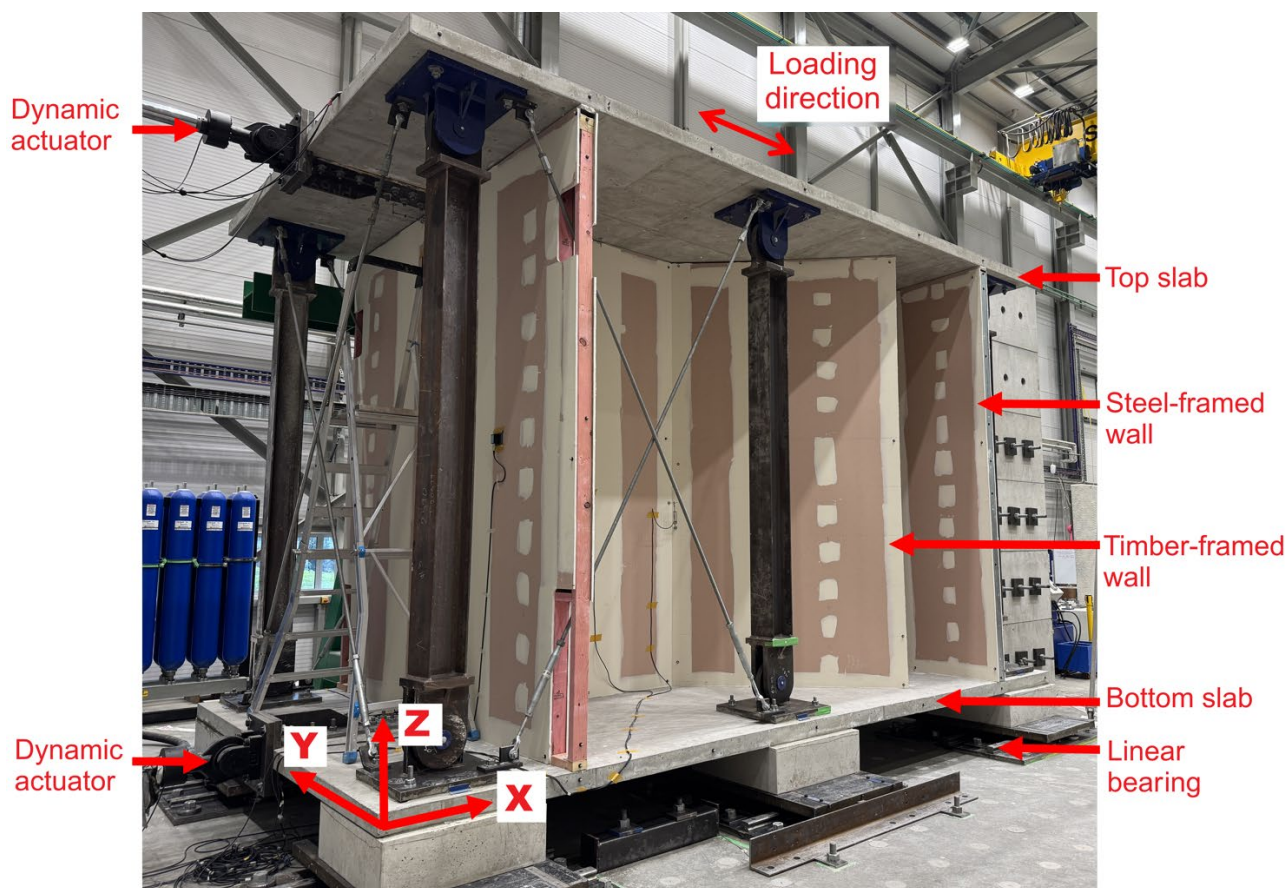


Figure 1: Photo of the test setup showing partition wall specimens

3.2 Wall specimen design, configurations and details

3.2.1 Walls with “unrestrained” ends (“unrestrained” specimens)

The two wall specimens, one timber-framed and one steel-framed, were constructed between the concrete slabs of the test setup as shown in Figure 2, Figure 3, and Figure 4. Their key design details were as follows:

- The walls were permitted to slide in-plane along the top deflection head track (DHT).
- Anchors connecting the DHTs to the top slab were omitted within 1200 mm of the wall T-junctions, defining the “No-Fix Zones” as shown in Figure 3.
- Out-of-plane flexibility at the junction was achieved through the bending of the top timber plate (in timber wall) and the DHT (in steel wall).
- At the wall junctions (with inclined walls and return walls), DHT flanges were cut approximately 40 mm short to prevent premature interaction with adjacent DHTs.
- Frames were lined with 13 mm GIB FYRELINE plasterboard per GIB EzyBrace® System specification GS2-N (GIB, 2016), but the plasterboard linings were not fastened to the top DHTs, as shown in Figure 4.
- The frames meeting at the wall T-junctions were connected. The top plates of the timber wall at these junctions were secured with a steel strap. The return intersection details in timber and steel framed walls are shown in Figure 4.

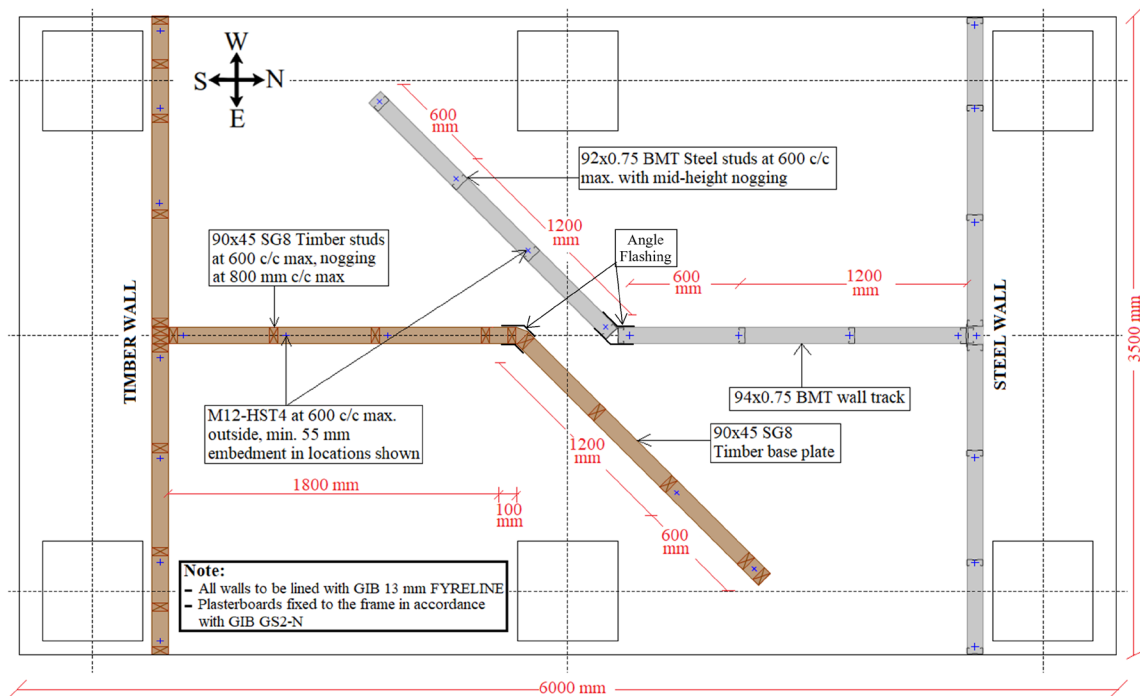


Figure 2: Timber and steel stud framing plan – “unrestrained” specimens

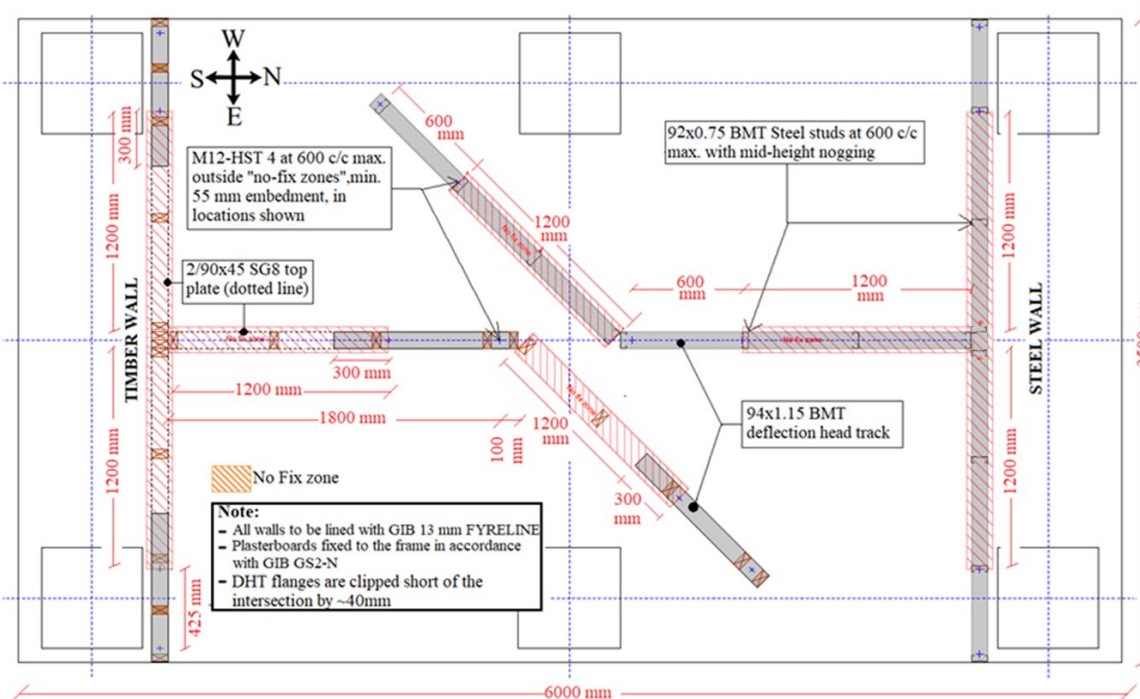


Figure 3: Timber and steel stud reflected ceiling plan – “unrestrained” specimens

3.2.2 Walls with “restrained” ends (“restrained” specimens)

The wall ends of the “unrestrained” wall specimens, were later, prevented from rotating (to represent an approximately fixed boundary condition) by placing $2/45 \times 90$ timber elements tightly against the wall linings as shown in Figure 5. These timber elements were secured in position using threaded rods anchored into the top slab.

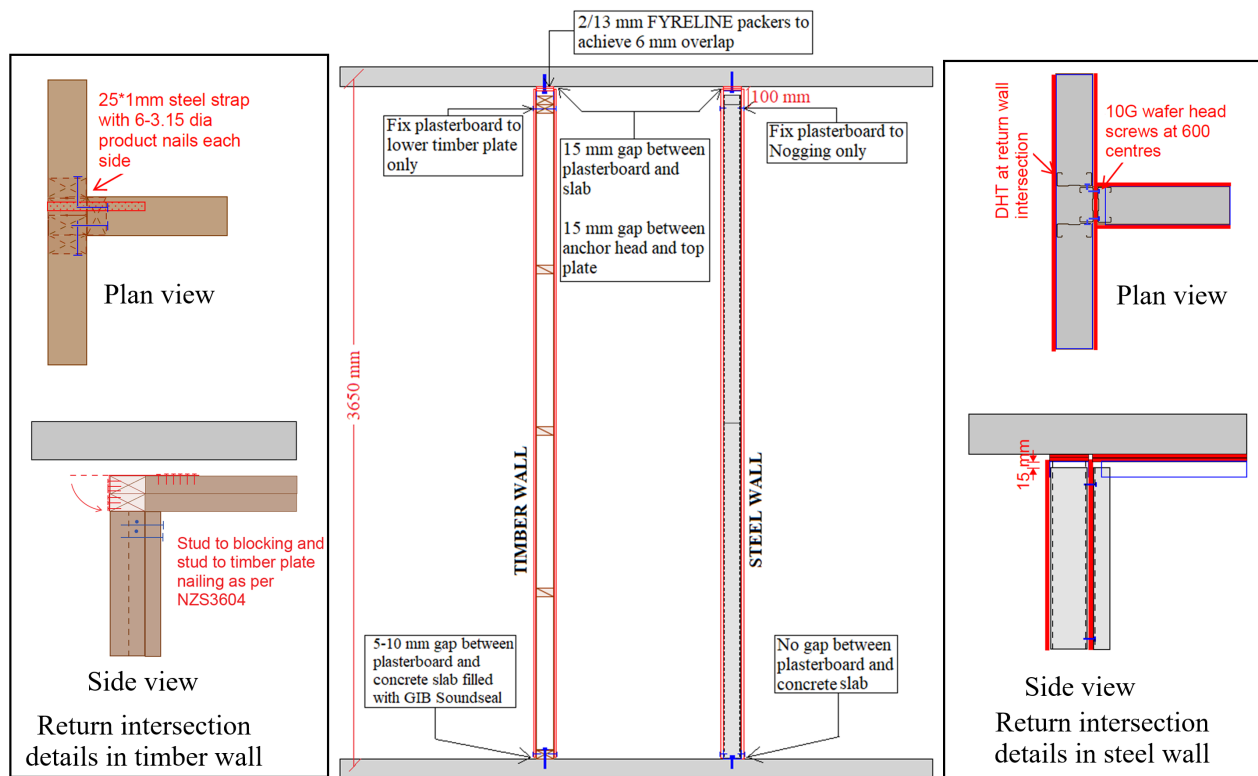
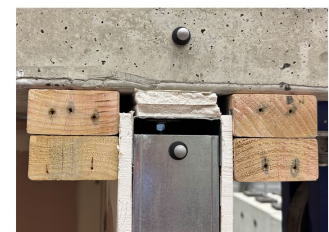
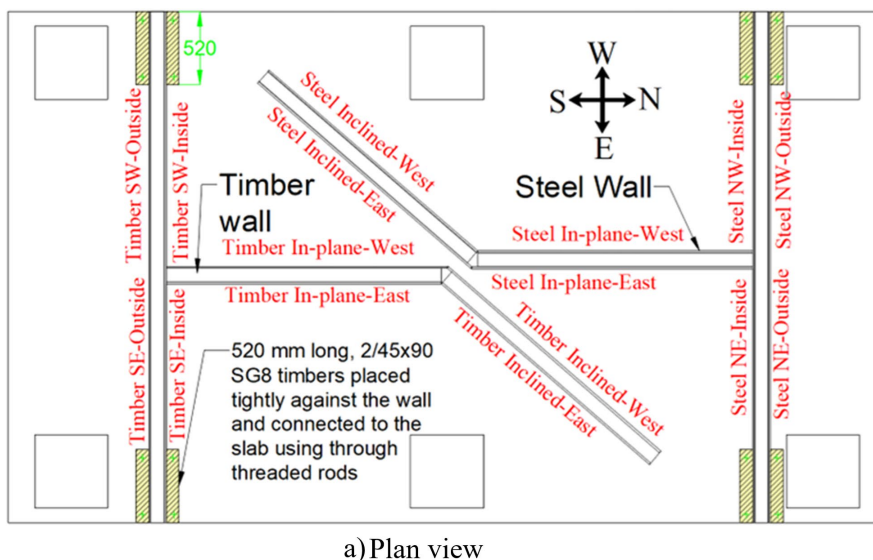


Figure 4: Wall section elevation and return intersection details (typical) – “unrestrained” specimens



b) Side view of restrained ends

Figure 5: Wall specimens with “restrained” ends with timber members

3.3 Loading protocols

Displacement time histories (referred to as “Motions”) for the second and third floors were extracted from a numerical model of a five-storey reinforced concrete building subjected to varying levels of the Imperial Valley earthquake (El Centro, 1940, Array #9) using different peak ground velocities (Pledger, 2026). These histories were then scaled to achieve the target inter-storey drifts specified in Table 1. Finally, the scaled motions for the third and second floors were applied to the top and bottom slabs of the test specimen, respectively. Figure 6 shows the scaled displacement input for Motion 14 as an example.

Testing of the rotationally “unrestrained” wall specimens (tested twice) concluded at Motion 10 (1.00 % inter-storey drift), while the “restrained” specimens were subjected to the full suite of motions in Table 1. Unless specified otherwise, the results and discussion in this paper focus on the “restrained” specimen.

Table 1: Input motions to the test setup

Motion	PFA ¹ Bottom slab (g)	PFV ² Bottom slab (m/s)	PFD ³ Bottom slab (mm)	PFA Top slab (g)	PFV Top slab (m/s)	PFD Top slab (mm)	Target ISD ⁴ (%)
0	0.01	0.01	2.70	0.02	0.02	3.40	0.03
1	0.04	0.04	9.64	0.05	0.05	12.14	0.10
2	0.08	0.08	19.29	0.11	0.11	24.29	0.20
3	0.12	0.12	28.93	0.16	0.16	36.43	0.30
4	0.16	0.15	36.55	0.21	0.21	46.21	0.40
5	0.20	0.19	45.69	0.27	0.26	57.76	0.50
6	0.24	0.23	54.83	0.32	0.31	69.31	0.60
7	0.28	0.27	56.00	0.32	0.37	72.47	0.70
8	0.32	0.31	64.00	0.37	0.42	82.82	0.80
9	0.36	0.35	72.00	0.41	0.48	93.18	0.90
10 ⁵	0.37	0.37	77.88	0.44	0.50	100.00	1.00
11	0.48	0.46	106.34	0.59	0.59	121.27	1.25
12	0.71	0.60	152.90	0.85	0.74	157.74	1.50
13	0.90	0.77	184.56	1.08	0.82	187.43	1.75
14	1.17	0.94	228.43	1.34	0.95	242.16	2.00
15 ⁶	0	0	0	0.36	0.42	82.13	2.25
16 ⁶	0	0	0	0.40	0.47	91.25	2.50
17 ⁶	0	0	0	0.44	0.51	100.38	2.75
18 ⁶	0	0	0	0.48	0.54	109.50	3.00

¹Peak Floor Acceleration, ²Peak Floor Velocity, ³Peak Floor Displacement, ⁴Inter Storey Drift,

⁵Final test for “unrestrained” wall specimens, ⁶Bottom slab is locked into position

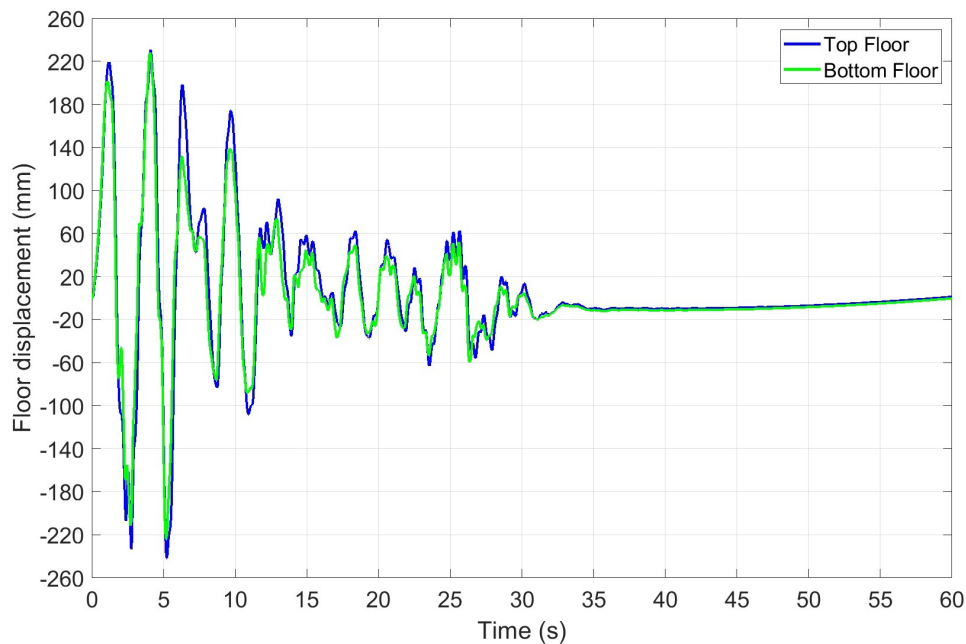


Figure 6. Floor displacement command time histories of top and bottom slabs for Motion 14

3.4 Instrumentation

The test structure was instrumented with accelerometers placed on the top slab and beneath the bottom slab to record accelerations along the loading direction (the North-South, or X-axis). To measure structural deformations, the setup was equipped with Motion Capture reflector points, monitored by five cameras positioned facing the east elevation during testing. Additionally, the wall specimens themselves were instrumented with displacement potentiometers, accelerometers as shown in Figure 7 and MoCap reflectors at key locations to capture their detailed response.

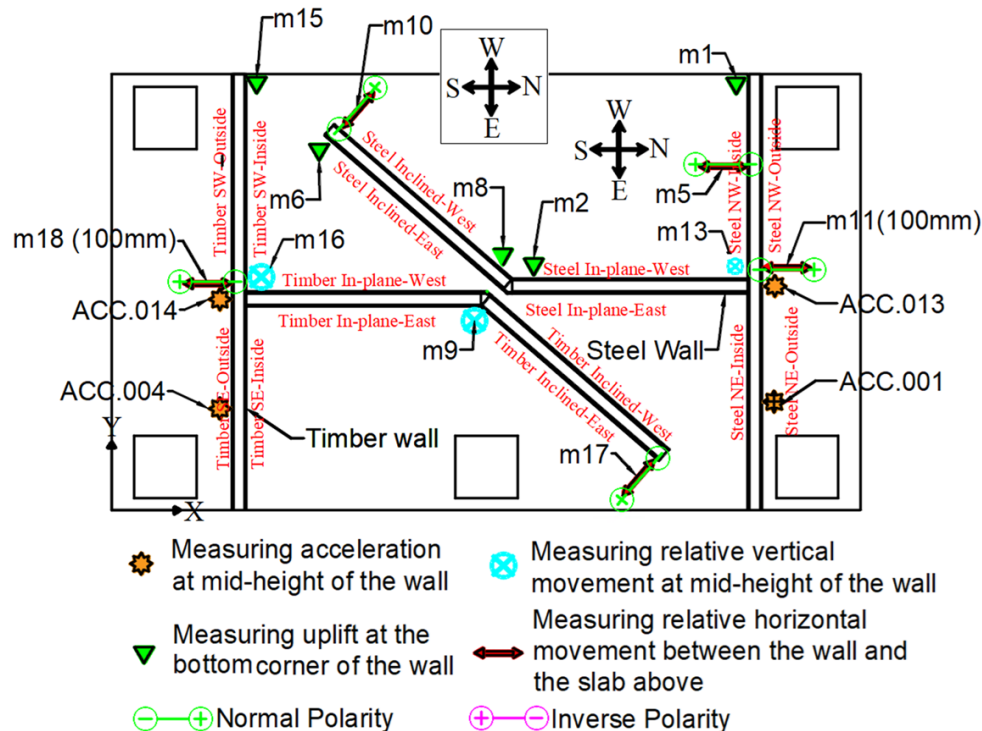


Figure 7: Instrumentation plan

3.5 Data processing

Recorded accelerations were filtered to reduce noise. A 4th-order low-pass Butterworth filter (50 Hz cutoff, 180 Hz sampling) (Butterworth, 1930) was applied to capture the specimens' local high-frequency response. A 4th-order 1-D median filter (window size 4) then smoothed impulse peaks by replacing each point with the local median, preserving signal edges with zero-padding. Displacement data were used unprocessed. For analysis, acceleration and drift data from Motion 0 were excluded due to negligible magnitude. In displacement plots, Motion 0 data are shown for completeness but omitted from discussion, as the measured values were insignificant.

4 EXPERIMENTAL RESULTS

4.1 Response of the test structure

The recorded inter-story drifts (ISDs) and peak floor accelerations (PFAs) of the test-structure are shown in Figure 8. The measured peak inter-storey drifts derived from actuator data reached 0.98 % during two initial tests of the “unrestrained” specimen. Following the addition of out-of-plane restraints, the “restrained” specimen was tested up to a drift of 3.08 %. Acceleration data, averaged from post-processed floor-mounted accelerometers, showed that the top floor consistently experienced higher peak horizontal accelerations than

the bottom floor. For instance, during Test #34, the top and bottom floor accelerations reached 1.28 g and 0.95 g in the loading direction (X), respectively.

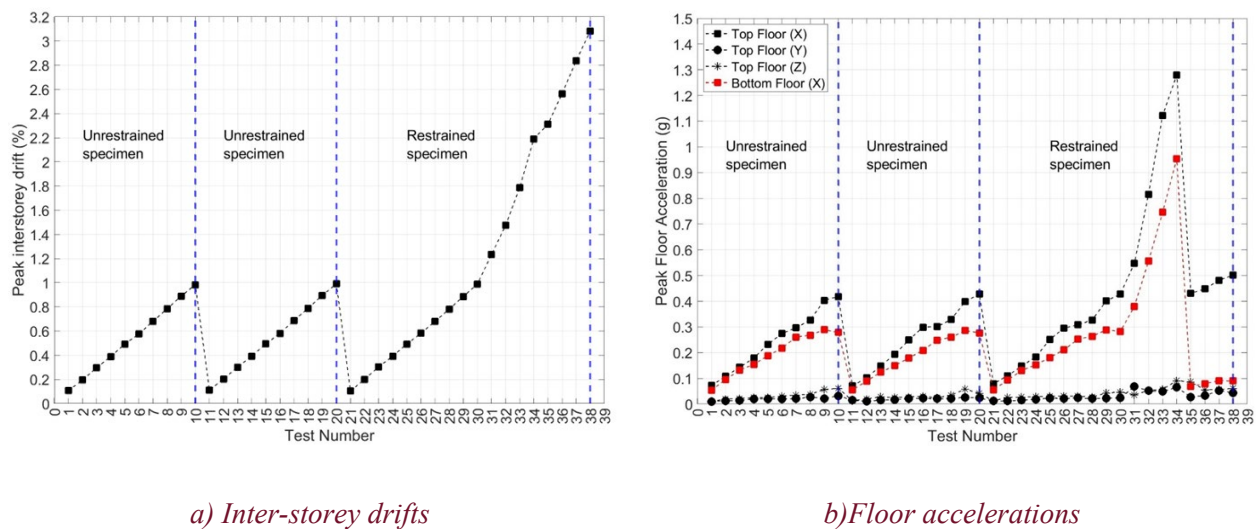


Figure 8: Recorded test structure response

4.2 Component response

4.2.1 Damage observation to specimens and their damage state classification

4.2.1.1 Timber Specimens

Damage in the “unrestrained” timber specimen, which was tested up to 1.00 % drift demand, initiated at a very low inter-storey drift of 0.11 % with the formation of hairline cracks at the wall T-junction. As drift increased, screw impressions became apparent at 0.39 %, followed by minor cracking in the plasterboard of the out-of-plane wall at 0.78 % drift; the cracking occurred where the DHT terminates running from the wall end - see Figure 3). This progression indicates that the specimen experienced gradual damage, with cosmetic and minor issues emerging early in the loading protocol.

The “restrained” timber specimen, which was tested to a much higher drift of 3.00 %, exhibited a different damage pattern influenced by prior testing. While hairline cracks in the plasterboard in the out-of-plane wall (where the DHT terminates running from the wall end) were noted at 0.30 % drift, these were considered progressive from the earlier “unrestrained” test. But more definitive new damage at that location, in the form of a crack in the plasterboard that could adversely affect the fire seal, was observed at 0.58 % drift (see Figure 9 (a)). This onset of damage may have been influenced by prior loading cycles associated with the “unrestrained” wall configuration i.e., flanges of DHT may have sustained some plastic deformation during the earlier loading cycles. Consequently, it is postulated the deformation capacity of 0.60 % could be considered a conservative estimate. The most significant damage occurred at 0.99 % drift, with the breaking of the top timber plates at the T-junction, a failure confirmed by loud sounds during testing and post-test inspection (see Figure 9 (b)). However, its effect on the wall response was not noticeable until afterwards. This sequence of damage shows that while restraint slightly delayed the onset of cosmetic damage, it ultimately led to a breaking of timber plate at a drift level comparable to where the “unrestrained” test ended.

4.2.1.2 Steel Specimens

For the “unrestrained” steel specimen, tested up to 1.00 % drift demand, initial damage appeared simultaneously at 0.20 % drift in the form of hairline cracks in the plasterboard (at T-junction) and screw impressions (the latter attributed to a construction error). The damage remained minor, with only a small tear in the joint paper observed at 0.89 % drift. The damage largely confined to finishes up to 1.00 % drift.

The “restrained” steel specimen, tested up to 3.08 % drift demand, demonstrated a significant delay in initial damage. Screw impressions were not recorded until 0.99 % drift (occurred in the out-of-plane wall near T-junction), and a minor joint paper tear (at the T-junction) followed at 1.23 % drift. Substantial damage began at 1.79 % drift, with cracks in the plasterboard at the top of the out-of-plane wall (opposite to the T-junction) and tears in the plaster paper at T-junction, both noted as potential compromises to the fire seal. Further cracking occurred at 2.19 % drift. The defining failure event was the separation of the steel frames at the wall junctions (can compromise occupancy) which occurred abruptly at 2.56 % drift (see Figure 9 (c)). This indicates that the restraint system suppressed early, minor damage but resulted in the accumulation of internal forces, leading to a sudden and brittle connection failure at high drift.

Davies et al. (2011) reports that in commercial H-shaped walls with slip tracks, such damage occurred at lower median drifts: screw impressions at 0.26 %, plasterboard cracking at 0.68 %, and frame damage/connection failure at 0.75 %. This benchmark highlights the significant improvement in the seismic resilience offered by the “No-Fix zones” detail in partition walls.

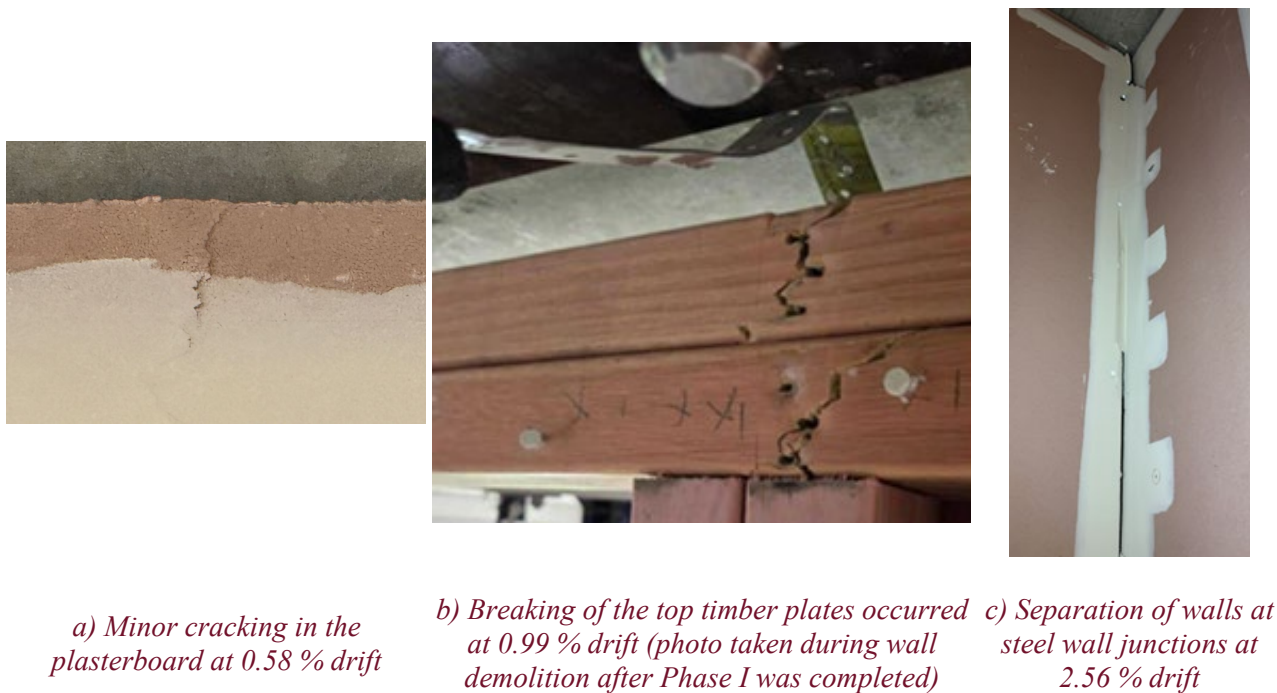


Figure 9: Photos of damage observed in the specimens

4.2.2 Peak component accelerations and dynamic amplification factors

Peak component accelerations (PCA) in the out-of-plane (X) direction at wall junctions and at mid-length of the overhang are plotted against the average peak floor acceleration (PFA) (of the top and bottom slabs) for all 38 tests of the timber-and steel-framed walls in Figure 10 (a). Accelerometers were located at the mid-height of the out-of-plane walls at the junctions and at a return overhang for both specimens. The box plot of all the PCA/PFA ratio for the walls, calculated by dividing the PCA by the average PFA of both floors is shown in Figure 10 (b) .

Figure 10 also shows PCA values calculated by multiplying the average PFA from both top and bottom floors by: (1) the ratio $C_i(T_p)/C_{ph}$ for various ductility levels (μ) as per SNZ TS 1170.5 (2025), where $C_i(T_p)$ is the component spectral-shape coefficient and C_{ph} is the component horizontal-response factor, and (2) the Component Resonance Ductility Factor (C_{AR}) from ASCE/SEI 7-22 (2022). A linear fit through the origin is included alongside the experimental data in Figure 10 (a). The experimental PCA/PFA ratio (Dynamic

Amplification Factor, including ductility μ) for out-of-plane light-frame walls is approximately 1.03 ($R^2=0.82$). This result aligns with the SNZ TS 1170.5 (2025) design cases for a Rigid component ($\mu = 1$) or a high-ductility Flexible component ($\mu \geq 2.5$). Both standards appear to provide conservative estimates for these test results: SNZ TS 1170.5 (2025) for $\mu = 2$ exceeds the measured ratio by a factor of 1.39, while ASCE/SEI 7-22 (2022) does so by a factor of 1.36. The median experimental PCA/PFA value for light-framed walls was found to be 1.05, as shown in Figure 10 (b).

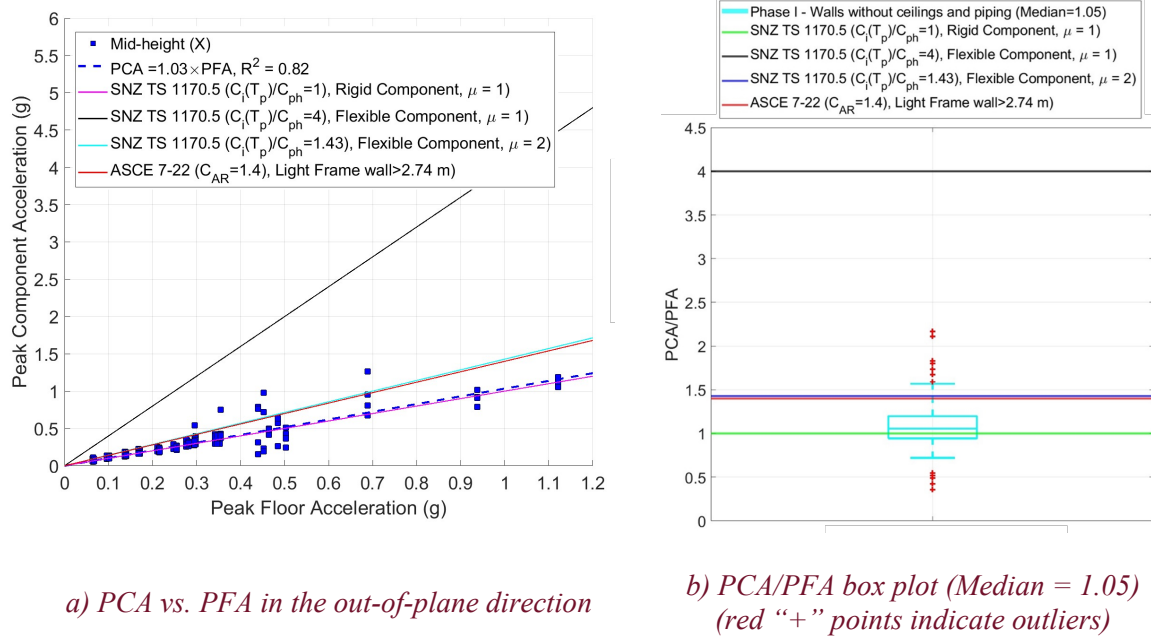


Figure 10: Peak component accelerations for wall specimens in out-of-plane direction (X)

4.2.3 Peak component lateral displacement

Figure 11 displays the absolute maximum lateral displacements of the timber wall specimen relative to the top slab, measured at various locations during the “restrained” test series.

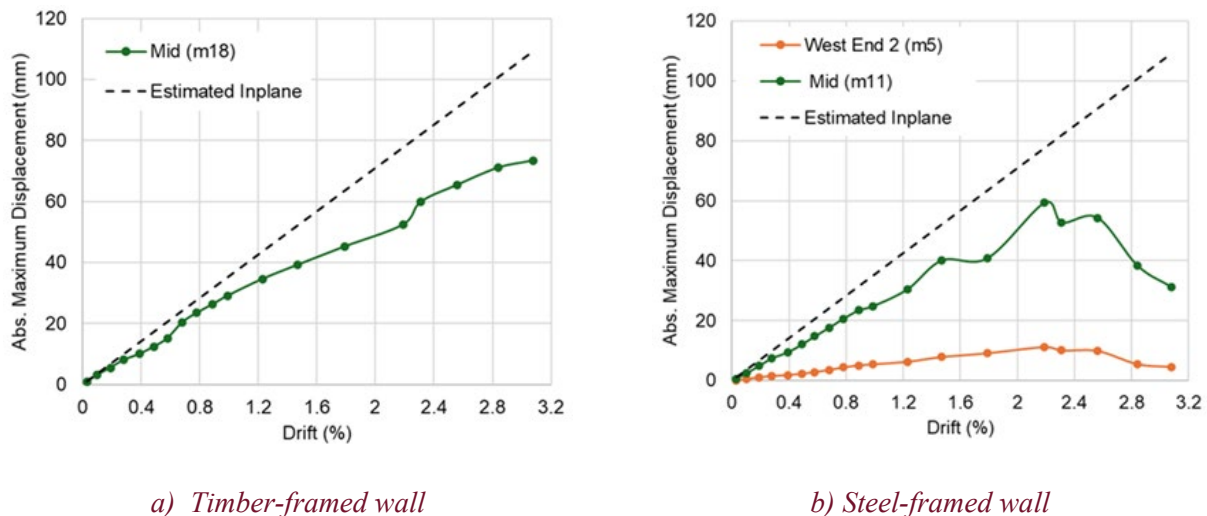


Figure 11: Absolute maximum lateral displacements relative to top slab for wall specimens (approximate measurement locations shown and instrument numbers in brackets – refer to Figure 7)

As shown in Figure 11 (a), the maximum displacements at the wall junction (mid-height of the out-of-plane wall) remain below the estimated in-plane displacements between the top slab and the deflection head track (calculated by multiplying the height of the wall and the drift), with a gradual divergence suggesting accumulating specimen damage. In Figure 11 (b), junction displacements similarly remain below the estimated values until exceeding 2.20 % drift, where a sharp divergence occurs. This shift is attributed to separation between the in-plane and out-of-plane walls, allowing the out-of-plane wall to move with the top slab.

5 CONCLUSIONS

This report presents the preliminary findings from partition wall tests on the wall designs currently adopted by the New Zealand construction industry. Two 3.55 m-high wall specimens, one with timber framing and the other with steel framing, were constructed with fire-rated plasterboard lining. Both walls were tested under dynamic loading conditions. Lateral sliding was enabled through a deflection head track (DHT) at the top, while out-of-plane flexibility at the wall junctions was provided by deformation of the timber top plates (in timber walls) and the DHTs (in steel walls). To facilitate this deformation, anchors at the junctions were set back 1.2 m from the wall ends, creating designated “No-Fix zones.”

The experimental results indicated that walls with (“unrestrained”/“pinned”) ends deformed as intended and exhibited no damage that compromised fire rating when tested up to a maximum drift of 1.00 %, while the cosmetic damage in the form of hairline crack at wall junctions occurred between 0.11 % (in timber wall) and 0.20 % drift (in steel wall). When the wall ends were rotationally “restrained”, effectively converting them to “fixed” ends, plasterboard crack damage compromising the fire-rating was observed at 0.60 % drift in the timber wall and at 1.80 % drift in the steel wall. Damage requiring major repair (with high priority) was deemed to occur at 1.00 % drift in the timber wall due to cracking of the top plate, and at 2.50 % drift in the steel wall due to separation at the junctions which can compromise occupancy. It should be noted that damage progression in the “restrained” walls is influenced by prior “unrestrained” testing, meaning the drift values at which damage appears are likely lower-bound estimates. When compared with traditional commercial walls, significant improvement in the seismic resilience was offered by the “No-Fix zones” detail.

Moreover, the median dynamic amplification factor for the tested wall specimens was found to be about 1.05, which corresponds to the design cases for a Rigid component ($\mu=1$) or a Flexible component with high ductility ($\mu\geq 2.5$) as per SNZ TS 1170.5 (2025). The SNZ TS 1170.5 (2025) provision for a Flexible component with $\mu=2$ and ASCE/SEI 7-22 (2022) provide a conservative estimate, exceeding the measured ratio by a factor of 1.30.

The displacement potentiometer recordings of the walls demonstrated significant out-of-plane flexibility around the junction regions (No-Fix zones). As a result, potential deformation incompatibilities are anticipated with attached or adjacent elements such as suspended ceilings, façades, doors, windows, service penetrations, and nearby structural members (e.g., columns or structural walls), requiring further investigation. A detailed dynamic characterization and displacement analysis of wall deformations are to be carried out using Motion Capture Camera recordings in future publications.

6 ACKNOWLEDGMENTS

This research was supported by funding from the New Zealand Natural Hazards Commission - Toka Tū Ake and QuakeCoRE. The authors gratefully acknowledge the constructive discussions and feedback provided by Andrew Baird (from Beca) and Jan Stanway (from WSP). Finally, valuable assistance in the SEL lab is acknowledged from SEL staff Alan Thirlwell, and Matthew Robinson. The QuakeCoRE publication number is 1136.

7 REFERENCES

- Arifin FA, Sullivan TJ, MacRae G, Kurata M and Takeda T (2021). "Lessons for loss assessment from the Canterbury earthquakes: a 22-storey building". *Bulletin of Earthquake Engineering*, **19**(5): 2081–2104. <https://doi.org/https://doi.org/10.1007/s10518-021-01055-7>.
- American Society of Civil Engineers (ASCE)(2022). "*ASCE/SEI 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*". Reston, VA, .
- AWCI (2018). "*Code of practice for the seismic design and installation of non-structural internal walls and partitions*". Association of Wall and Ceiling Industries of New Zealand Inc, <https://awci.org.nz/wp-content/uploads/COP-Internal-Walls-and-Partitions-coloured.pdf>.
- Bachman RE (2004). "The ATC-58 project plan for nonstructural components". *Performance-Based Seismic Design: Concepts and Implementation, Proceedings of the International Workshop, Bled, Slovenia, 28 June–1 July 2004*, .
- Baird A, Tasligedik AS, Palermo A and Pampanin S (2014). "Seismic performance of vertical nonstructural components in the 22 February 2011 Christchurch earthquake". *Earthquake Spectra*, **30**(1): 401–425. <https://doi.org/https://doi.org/10.1193/031013EQS067M>.
- Bhatta J (2026). "*NHC Project #4454: Seismically Resilient Partition Wall Design and Testing*". Natural Hazard Commission, .
- Bhatta J and Dhakal RP (2026). "Seismic Response Assessment of “Rocking” Interior Partition Walls with Dual-Slot Tracks Under Different Construction Details". *Journal of Earthquake Engineering*, : 1–25. <https://doi.org/https://doi.org/10.1080/13632469.2025.2612533>.
- Bhatta J, Dhakal RP and Sullivan TJ (2023a). "Seismic evaluation of rocking internal partition walls with dual-slot track under quasi-static cyclic drifts". *Journal of Earthquake Engineering*, **27**(14): 4161–4181. <https://doi.org/https://doi.org/10.1080/13632469.2022.2162629>.
- Bhatta J, Dhakal RP and Sullivan TJ (2023b). "Seismic performance of ‘rocking’ dual-slot track internal partition walls with concealed joints under lateral cyclic drift demands". *Journal of Earthquake Engineering*, **27**(8): 2198–2223. <https://doi.org/https://doi.org/10.1080/13632469.2022.2112322>.
- Bhatta J, Dhakal RP, Sullivan TJ, Bartlett J and Pring G (2023). "Seismic performance of internal partition walls with slotted and bracketed head-tracks". *Journal of Earthquake Engineering*, **27**(12): 3435–3470. <https://doi.org/https://doi.org/10.1080/13632469.2022.2137709>.
- Bradley B, Dhakal R, Cubrinovski M, MacRae GA and Lee DS (2008). "Seismic loss estimation for efficient decision making". <https://doi.org/https://doi.org/10.5459/bnzsee.42.2.96-110>.
- Butterworth S (1930). "On the theory of filter amplifiers". *Wireless Engineer*, **7**(6): 536–541.
- Davies RD, Retamales R, Mosqueda G and Filiatrault A (2011). "*Experimental Seismic Evaluation, Model Parameterization, and Effects of Cold-Formed Steel-Framed Gypsum Partition Walls on the Seismic Performance of an Essential Facility*". MCEER, .
- Dhakal R (2010). "Damage to non-structural components and contents in 2010 Darfield earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*, **43**(4): 404–411. <https://doi.org/https://doi.org/10.5459/bnzsee.43.4.404-411>.
- Dhakal R, MacRae G and Hogg K (2011). "Performance of ceilings in the February 2011 Christchurch earthquake". <https://doi.org/https://doi.org/10.5459/bnzsee.44.4.377-387>.
- FEMA (1994). "*FEMA 74: Reducing the Risks of Nonstructural Earthquake Damage*". Federal Emergency Management Agency, <https://mitigation.eeri.org/files/fema-74.pdf>.
- Filiatrault A (2002). "*Guidelines, Specifications, and Seismic Performance Characterization of Nonstructural Building Components and Equipment*". Pacific Earthquake Engineering Research Center, .
- GIB (2016). "*GIB-EzyBrace-System-Sheet-GS2-N*". GIB, .

- Kim H and Shin D (2021). "Shake table test program of cold-formed steel in-plane partition walls". *Structures*, **30**: 503–517. <https://doi.org/https://doi.org/10.1016/j.istruc.2021.01.026>.
- MBIE and NHC (2024). "Low Damage Seismic Design, Volume 1: benefits, options and getting started". Ministry of Business, Innovation and Employment (MBIE) and Natural Hazards Commission (NHC), <https://www.building.govt.nz/assets/Uploads/getting-started/seismic-work-programme/low-damage-seismic-design-for-buildings.pdf>.
- Mulligan J, Sullivan TJ and Dhakal RP (2022). "Experimental seismic performance of partly-sliding partition walls". *Journal of Earthquake Engineering*, **26**(3): 1630–1655.
- Mulligan J, Sullivan T and Dhakal R (2020). "Experimental study of the seismic performance of plasterboard partition walls with seismic gaps". *Bulletin of the New Zealand Society for Earthquake Engineering*, **53**(4): 175–188. <https://doi.org/https://doi.org/10.5459/bnzsee.53.4.175-188>.
- Petrone C, Magliulo G, Lopez P and Manfredi G (2015). "Seismic fragility of plasterboard partitions via in-plane quasi-static tests". *Earthquake Engineering & Structural Dynamics*, **44**(14): 2589–2606. <https://doi.org/https://doi.org/10.1002/eqe.2600>.
- Pledger L (2026). "The Effects of Reducing Design Drift Limits for Structures". PhD Dissertation, University of Canterbury
- Restrepo JI and Bersofsky AM (2011). "Performance characteristics of light gage steel stud partition walls". *Thin-Walled Structures*, **49**(2): 317–324. <https://doi.org/https://doi.org/10.1016/j.tws.2010.10.001>.
- Standards New Zealand, Standards Australia(2002). "AS/NZS 1170.0: Structural Design Actions - General principles". Wellington, New Zealand, .
- Standards New Zealand (2025). "SNZ TS 1170.5: Structural Design Actions – Part 5: Earthquake Actions – New Zealand". Wellington, New Zealand, .
- Tasligedik AS (2014). "Damage Mitigation Strategies for Non-Structural Infill Walls.". PhD Dissertation, University of Canterbury, https://ir.canterbury.ac.nz/bitstream/handle/10092/9462/thesis_fulltext.pdf?sequence=1&isAllowed=y.
- Tasligedik AS, Pampanin S and Palermo A (2015). "Low damage seismic solutions for non-structural drywall partitions". *Bulletin of Earthquake Engineering*, **13**(4): 1029–1050. <https://doi.org/https://doi.org/10.1007/s10518-014-9654-5>.
- Yeow TZ, Sullivan TJ and Elwood KJ (2018). "Evaluation of fragility functions with potential relevance for use in New Zealand". *Bulletin of the New Zealand Society for Earthquake Engineering*, **51**(3): 127–144. <https://doi.org/https://doi.org/10.5459/bnzsee.51.3.127-144>.