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Seismic Performance Evaluation of Rocking Plasterboard Partition Walls Via Full-Scale Shake Table Tests

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ABSTRACT

This paper presents the seismic performance of rocking plasterboard partition walls installed on the first storey of a full-scale, three-storey steel-frame building subjected to unidirectional and bidirectional horizontal ground motions. Three configurations (planar, L-shaped, and T-shaped) of rocking partition walls with dual-slot tracks were tested to assess their response under various seismic inputs. Peak rocking displacements were measured and compared with theoretical predictions. The observed damage was classified into different damage states, which were correlated with inter-storey drift demands and non-structural performance levels defined in ASCE/SEI 41-23 (2023). The planar walls accommodated peak in-plane inter-storey drift ratios of up to 1.50% without any visible damage. In contrast, the L-shaped and T-shaped walls accommodated peak in-plane inter-storey drift ratios of up to 2.12%, sustaining moderate but easily repairable damage. The partition walls experienced out-of-plane acceleration demands higher than the corresponding peak floor accelerations, with median amplification factors ranging from 1.5 to 3.5. Additionally, dynamic characteristics of the wall system, including natural frequency and damping ratio, were derived from the experimental results. The out-of-plane natural frequencies were found to range from 9.51 Hz to 7.79 Hz for the T-shaped walls and from 13.18 Hz to 11.90 Hz for the planar walls. Both wall configurations exhibited similar damping ratios, with values falling within the 1–6% range.

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Rocking partition walls; low-damage design; shake table test; dual-slot track; acceleration amplification; dynamic characterization

1. Introduction

Partition walls are key elements in modern buildings, used to divide large interior spaces into smaller, functional areas and to provide separation within or between occupancies. In commercial buildings, partition walls are typically designed as non-structural elements (NSEs), meaning they are not intended to be part of the primary gravity or lateral load-resisting system of a structure (AWCI 2018; Baird et al. 2014). Partition walls span between two consecutive floors and can, therefore, be classified as drift-sensitive components in their own plane, with their damage primarily governed by inter-story drift (Taghavi and Miranda 2003). However, during seismic events, they may also fail in the out-of-plane direction (Davies et al. 2011), where the response is predominantly controlled by the acceleration experienced by the partition wall about its center of mass (Magliulo et al. 2025). For this

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reason, partition walls have been recognized as elements sensitive to both drift and acceleration demands (Zito et al. 2022).

Commonly used partition systems include masonry infills, glass-aluminum partition walls, lightweight hollow partition walls, and plasterboard-sheathed partition walls (also known as drywall). Among these, this paper focuses on plasterboard partition walls, which are generally constructed using lightweight steel or timber frames lined with plasterboard on one or both sides. The plasterboard may be installed vertically or horizontally and attached to the studs using adhesive, self-drilling screws, or specially designed nails (AWCI 2018), and in some cases, using adhesive fixings alone (Yazdi, Hashemi, and Gad 2022). These walls are widely used in multi-story buildings due to their ease of installation, familiarity, fire resistance, and acoustic performance. Evidence from past major seismic events, including the 1971 San Fernando earthquake (Lew, Leyendecker, and Dikkers 1971), the 1994 Northridge earthquake (FEMA E-74, 2011), the 2001 Nisqually earthquake (Filiatrault et al. 2001), the 2010 Chile earthquake (Miranda et al. 2012), the 2010 Darfield earthquake (Dhakal 2010), the 2011 Christchurch earthquake (Baird et al. 2014), and the 2016 Kaikōura earthquake (Baird and Ferner 2017), has repeatedly shown that partition walls are highly vulnerable to earthquakes. Commonly reported damage includes plaster cracking, screw pull-out at plasterboard-to-stud connections, detachment of studs from tracks, and cracking or dislodgement of plasterboard. While post-earthquake reconnaissance highlights the vulnerability of partition walls, it rarely provides quantitative data that can correlate observed damage states with seismic demand parameters.

Seismic damage to partition walls is not merely an aesthetic issue; it can cause significant economic losses, disruption to building functionality, and occasionally pose risks to life safety (Arifin et al. 2021; AWCI 2018; Bradley et al. 2009; Dhakal, Pourali, and Saha 2016; Magliulo et al. 2022). Bradley et al. (2009) reported that partition walls may contribute up to 20% of a building's total seismic loss. In addition, Magliulo et al. (2022) quantified the seismic losses associated with the damage to traditional and innovative partition wall systems. Furthermore, partition walls are more vulnerable than the primary structural system, often experiencing substantial damage at lower drift levels. For instance, steel-framed and timber-framed partition walls constructed using standard New Zealand construction practices were found to lose serviceability at inter-story drifts of 0.3% and 0.75%, respectively (Tasligedik, Pampanin, and Palermo 2015), whereas reinforced concrete or structural steel buildings can sustain drifts exceeding 1.2% before requiring repair (Priestley, Calvi, and Kowalsky 2007).

Numerous component-level studies have investigated the lateral in-plane behavior of partition walls. Early experimental work primarily focused on identifying damage thresholds and fundamental mechanical properties such as stiffness, strength, and energy dissipation capacity (Adham et al. 1990; Freeman 1971; Rihal 1980). Following the 1994 Northridge and 1995 Kobe earthquakes, the emergence of Performance-Based Earthquake Engineering (PBEE) shifted attention towards designing NSEs to meet seismic design demands. Since then, several experimental studies have examined the seismic performance of partition walls constructed using typical building practices (Davies et al. 2011; Kim and Shin 2021; Kishiki, Fu, and Tatsumi 2022; McMullin and Merrick 2007; Petrone et al. 2015; Restrepo and Bersofsky 2011; Restrepo and Lang 2011; Tasligedik, Pampanin, and Palermo 2015; D. Wang et al. 2021). Davies et al. (2011) conducted both quasi-static and dynamic tests on 50 full-scale partition walls across 22 configurations and reported that full-height walls with commercial slip-track systems were damaged at a mean inter-story drift of 0.26%. Petrone et al. (2015) conducted quasi-static tests on six partition walls representative of typical European industrial and commercial buildings and reported a median inter-story drift of 0.28% at the onset of initial damage. Furthermore, D. Wang et al. (2021) conducted tests on 15 I-shaped partition wall specimens designed in accordance with the Chinese design code and engineering practice and observed that damage initiated at mean inter-story drift levels as low as 0.25%.

In recent years, significant research efforts have been directed towards developing improved connection details for partition walls using various strategies, including sliding or friction connections (Araya-Letelier, Miranda, and Deierlein 2019; Jenkins et al. 2016), seismic gaps (Fiorino, Bucciero, and

Landolfo 2019; Lee et al. 2007; Magliulo et al. 2014; Mulligan, Sullivan, and Dhakal 2020; Pali et al. 2018; Tasligedik, Pampanin, and Palermo 2015), flexible track systems (Davies et al. 2011; Mulligan, Sullivan, and Dhakal 2022; Retamales et al. 2013), double deflection track systems or telescopic details (Hasani and Ryan 2021), slotted and bracketed connections (Bhatta et al. 2023; Menegon, Hashemi, and Gad 2023), no-fix zone details (Bhatta, Oliver, et al. 2026), and viscoelastic dissipative devices (Ditomaso et al. 2026; Scozzese, Zona, and Dall'asta 2024). Partition walls incorporating these improved construction details have demonstrated superior seismic performance compared to traditional fully connected partition wall systems, achieving higher drift capacities before damage onset and reducing economic losses. For instance, walls with friction/sliding details sustained negligible damage up to drifts of 1.5%, compared to 0.1–0.3% for conventional partition walls. The expected annual loss of conventional partition wall systems was eight times larger than that of the improved system (Araya-Letelier and Miranda 2012; Araya-Letelier, Miranda, and Deierlein 2019). Recently, to reduce the required horizontal seismic gaps while maintaining architectural functionality and constructability, Bhatta, Dhakal, and Sullivan (2023b) introduced a novel rocking partition wall system with dual-slot tracks (DSTs) that successfully accommodated the intended inter-story drifts.

Despite these advances, low-damage partition wall systems have been experimentally validated at the component level, primarily through tests on individual partition walls with various configurations and boundary conditions. Such tests, while informative, cannot fully replicate the combined effects of realistic earthquake inputs, floor vibrations, and boundary conditions. Moreover, only a limited number of system-level shake table tests have investigated the seismic performance of partition walls in either reduced-scale buildings (Bianchi et al. 2021; Pampanin et al. 2023; Shegay et al. 2026) or full-scale buildings (Jenkins et al. 2016; Soroushian et al. 2016; X. Wang et al. 2015) equipped with multiple NSEs subjected to ground motion. X. Wang et al. (2015) performed shake table tests on a full-scale, five-story reinforced concrete building with partition walls common in the United States, observing minor damage in the base-isolated condition and moderate damage at drifts of 0.66–1.09% in the fixed-base condition. Additionally, Soroushian et al. (2016) tested an integrated ceiling–piping–partition system installed in a five-story steel moment frame building. In their study, partition walls showed no drift-related damage; however, large diagonal and vertical cracks were observed in the plasterboard, caused by relative vertical acceleration between the floors. Lateral dislodgement of steel studs from the top slip-track connection also occurred due to the large differential movement between the top track and the studs. These findings highlight that the performance of partition walls is governed not only by drift demands but also by acceleration-driven responses. Recently, Magliulo, Zito, and D'angela (2024) conducted shake table tests on a full-scale innovative cleanroom system comprising partition walls, ceilings, ventilation, and electrical systems. The specimen was subjected to artificial floor motions generated according to the international certification protocol AC156 (ICC-ES 2020), demonstrating the excellent seismic performance of integrated non-structural systems under representative service conditions.

Partition walls have been observed to experience amplified floor accelerations (Fiorino, Bucciero, and Landolfo 2019; Jenkins et al. 2016; Landolfo et al. 2025; Petrone, Magliulo, and Manfredi 2017), in some cases exceeding the amplification factor prescribed by ASCE/SEI 7-10 (2010) for flexible non-structural components (Soroushian et al. 2016). It is known that dynamic amplification is affected by the proximity of the component period to the structural period as well as the damping of the structure and the component (Haymes, Sullivan, and Hare 2025; X. Wang and Hutchinson 2024; Welch and Sullivan 2017). Although a few studies have examined the dynamic properties of partition wall systems such as fundamental frequency and damping ratio (Landolfo et al. 2025; Magliulo et al. 2014; Petrone, Magliulo, and Manfredi 2017; Zhong et al. 2025), the available data are still insufficient. Magliulo et al. (2014) evaluated out-of-plane natural periods ranging from 0.06 s to 0.08 s, while Landolfo et al. (2025) reported mean fundamental periods of 0.11 s (in-plane) and 0.08 s (out-of-plane). Petrone, Magliulo, and Manfredi (2017) found damping ratios ranging from 5% to 10% for different partition walls.

In summary, while significant progress has been made in the development of low-damage partition wall systems, system-level experimental data are still insufficient (Cook et al. 2026),

particularly regarding visual damage, acceleration amplification, and dynamic characteristics of partition walls installed in full-scale buildings under realistic earthquake shaking. Although the seismic performance of rocking partition walls with DSTs has been experimentally validated through component-level quasi-static cyclic tests (Bhatta and Dhakal 2026; Bhatta, Dhakal, and Sullivan 2023b, 2023c) and unidirectional dynamic tests (Bhatta, Shrestha, et al. 2026), their system-level performance when installed in a full-scale building under both unidirectional and bidirectional dynamic loading has not yet been investigated. To address these gaps, shake table tests were conducted on a full-scale steel-frame–composite floor building at the International Joint Research Laboratory of Earthquake Engineering (ILEE), Tongji University, Shanghai, China. These tests were carried out as part of a New Zealand–China collaborative research initiative under the RObust BUilding SysTem (ROBUST) project. The primary objective of the ROBUST project is to introduce and validate various low-damage concepts for both structural (MacRae et al. 2020; Zhang et al. 2026) and non-structural components (Dhakal et al. 2020; Shrestha et al. 2026; Tiwari et al. 2026). The novelty of the present study lies in the first full-scale shake table validation of the previously developed rocking partition wall system under unidirectional and bidirectional horizontal ground motions. The specific objectives of this study are to (i) evaluate the seismic performance of rocking partition walls with different configurations and document their associated damage mechanisms; (ii) compare the experimental rocking responses with theoretical predictions; (iii) quantify acceleration amplification factors; and (iv) identify the dynamic properties, including natural frequency and damping ratio.

2. Experimental Program

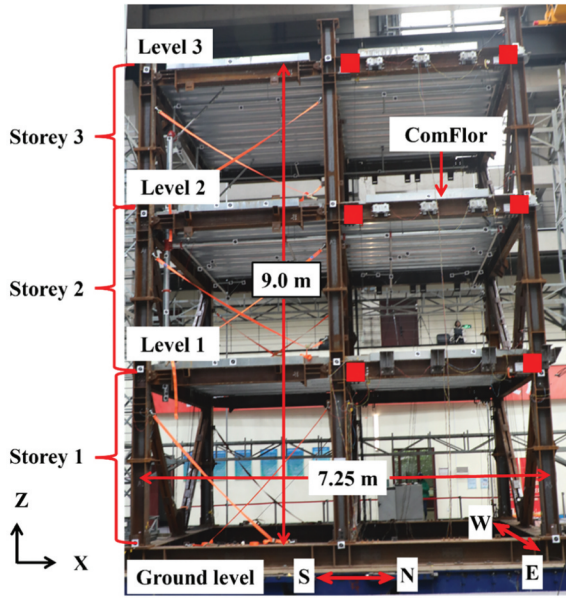
2.1. Primary Structural System

The test structure in the ROBUST project was a three-story steel-frame building with plan dimensions of 7.25 m × 4.75 m and an inter-story height of 3 m, as shown in Fig. 1. The structural layout of the building comprised two bays in the longitudinal (NS/X) direction and one bay in the transverse (EW/Y) direction. The building was designed as an office building (Importance Level 2) in accordance with NZS 1170.0 (2002), located on shallow soil, i.e. Subsoil Class C as per NZS 1170.5 (2016), in Wellington, New Zealand.

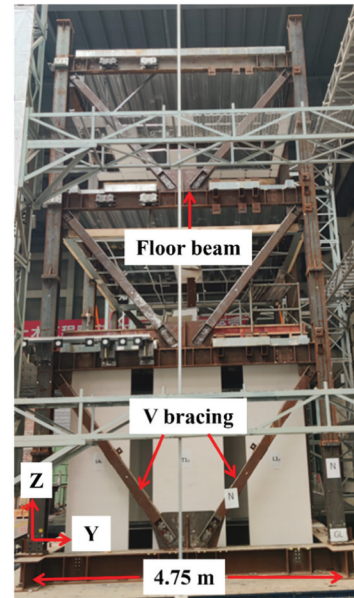
During partition wall testing, the structural configuration consisted of a lateral load-resisting system formed by a moment-resisting steel-frame (MRSF) with Optimized Sliding Hinge Joints (OSHJs) (Ramhormozian et al. 2021) in one bay, while another bay had pinned beam-to-column connections in the longitudinal (NS/X) direction. The lateral load-resisting system in the transverse (EW/Y) direction was a V-braced concentrically braced frame (CBF) with Symmetric Friction Connections (SFC) incorporating partially deflected Belleville Springs (SFCBeSs) (Ramhormozian et al. 2017), as shown in Fig. 1. The central column was designed to carry only gravity loads while remaining capable of accommodating lateral drifts under the rocking configuration. A composite flooring system, ComFlor® 80 (Steel & Tube Ltd. 2016), consisting of a trapezoidal steel deck with a concrete topping, was used. Additional mass blocks were placed at each floor level to simulate the live load, using uniform loads of 3.5 kPa at Levels 1 and 2 and 4.7 kPa at Level 3 (Assadi et al. 2024). Further details of the different structural configurations and the experimental setup are available in Yan et al. (2021), Yan et al. (2023), and MacRae et al. (2024).

2.2. Design Philosophy of the Rocking Partition Wall System

The rocking partition wall system incorporating DST connections was initially proposed by Bhatta, Dhakal, and Sullivan (2023b). The DSTs are constructed by screwing two steel tracks of different widths together, creating separate slots of sufficient width to accommodate the



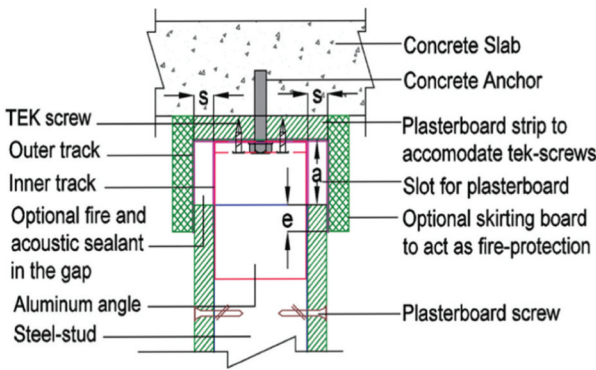
(a) Longitudinal (NS/X) direction (MRSF – OSHJ system–OSHJ locations highlighted in red)



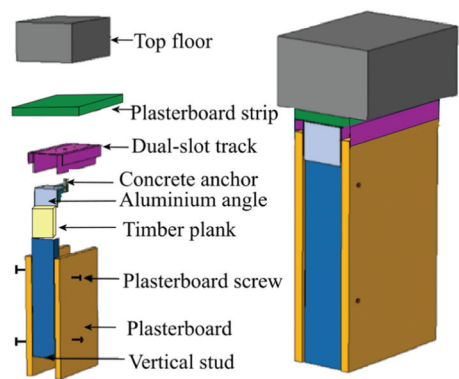
(b) Transverse (EW/Y) direction (CBF–V–SFCBeSs system)

Figure 1. Three-storey steel-frame test structure (Shrestha et al., 2026).

studs and plasterboard on both sides (Figs. 2 and 3). A vertical gap is provided between the DST web attached beneath the upper floor slab and the top of the wall (i.e. top of the studs and plasterboard), as depicted in Fig. 2. For this wall system, the in-plane drift capacity “ δ ” depends on this vertical gap as well as the length of the wall and can be expressed as $\delta = a/L$ (Bhatta, Dhakal, and Sullivan 2023b), where “ a ” is the top vertical gap, and “ L ” is the total length of the wall. Aluminum angles are screwed to the ends of the DSTs (at the four corners



(a) Schematic diagram



(b) Isometric view: exploded (left) and combined (right)

Figure 2. Top rocking connection with dual-slot track (DST) (adapted from Bhatta, Dhakal, & Sullivan, 2023c).

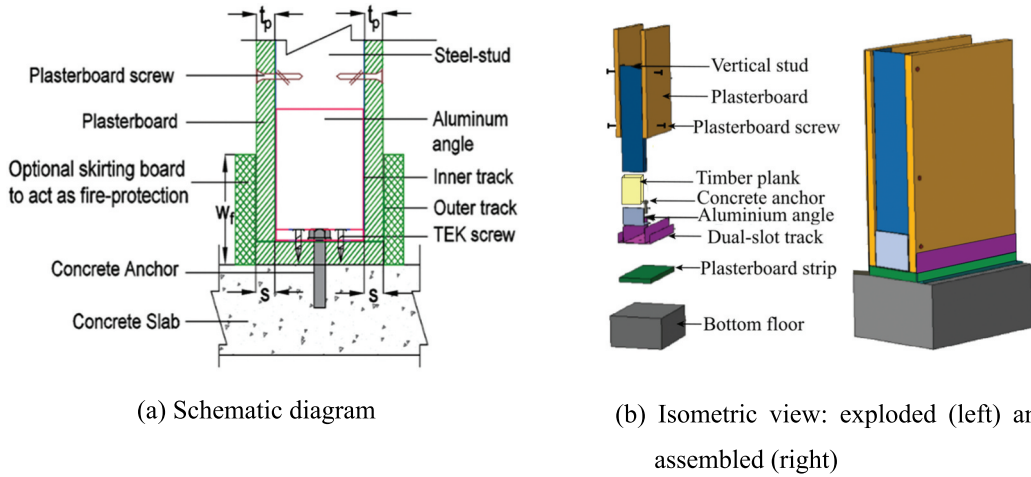


Figure 3. Bottom rocking connection with dual-slot track (DST) (adapted from Bhatta, Dhakal, & Sullivan, 2023c).

of each wall) to enable the walls to rock and prevent direct interaction between the plasterboard and adjacent boundary elements, such as columns and return walls. During lateral displacement of the top slab, the aluminum angles transfer the loads to the wall while allowing the plasterboard and internal frames to rotate freely until the vertical gap closes.

2.3. Specimen Design, Configurations, and Construction Details

Three configurations of the rocking partition walls, namely L-shaped, T-shaped (hereafter referred to as L-wall and T-wall, respectively), and planar, were installed at Story 1, between the ground level and Level 1 of the tested structure, as shown in Fig. 4. Each wall was 100 mm thick, with 10 mm thick plasterboard on each side. The wall heights varied such that the L- and T-walls were 3.5 m tall, while the planar walls were 2.7 m tall, depending on their placement. Since the ground level was the top of the shake table, 140 mm × 80 mm × 4 mm rectangular hollow sections (RHS) were placed underneath the bottom DST of the L- and T-walls to facilitate their attachment to the shake table, as shown in Fig. 5a. These RHS were placed with their 140 mm side resting on the shake table and bolted using M24 bolts spaced at 500 mm centers (c/c). The top DST of these walls was attached beneath the troughs of the ComFlor at Level 1. Where the wall could not be aligned along the troughs, 300 mm × 100 mm × 5 mm steel plates were used to bridge the gaps between troughs and secure the top DST, as shown in Fig. 5b. However, the planar walls were constructed between 125 mm × 75 mm × 5 mm RHS sections mounted on the 400 mm deep ring beam at the ground level and the 310 mm deep floor beam at Level 1 using two M12 bolts spaced at 500 mm c/c.

The DSTs were fabricated by fastening a 75 mm × 60 mm × 0.6 mm steel track to a 100 mm × 45 mm × 0.6 mm steel track using tek screws, as shown in Fig. 6a. The inner track had extended flanges to enable the smooth insertion and sliding of the plasterboard into the slots during installation. At the ends of the DSTs, 73 mm × 48 mm × 3 mm aluminum angles were attached using four tek screws per angle, as illustrated in Fig. 6b. At some locations, 6 mm-thick steel angles were added externally to stiffen the 3 mm-thick aluminum angles. This intervention was intended to simulate stiffer end conditions as well as to prevent premature yielding of the aluminum angles. These DSTs were anchored to the RHS sections using tek screws and to the ComFlor deck using concrete anchors spaced at 500 mm c/c.

The studs at the ends of the partition walls were constructed as boxed studs by clipping two 75 mm × 35 mm × 0.6 mm studs together. The ends of the boxed studs were reinforced with 150 mm ×

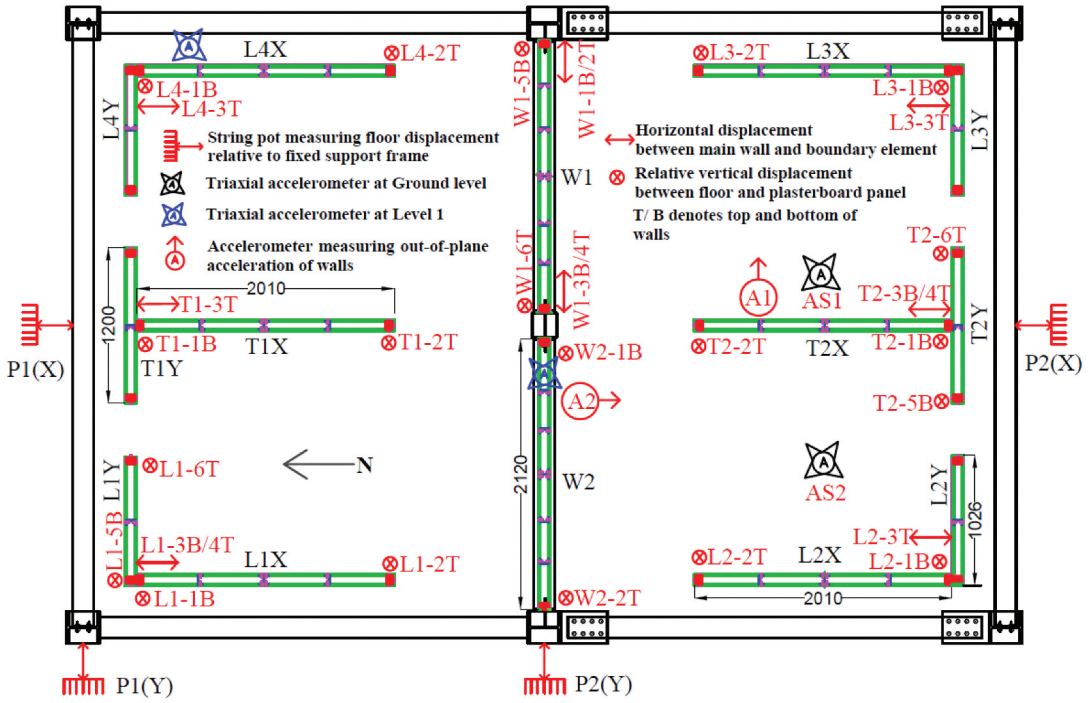


Figure 4. Layout of partition walls on the structure, showing instrumentation for both the structural system and the partition walls.

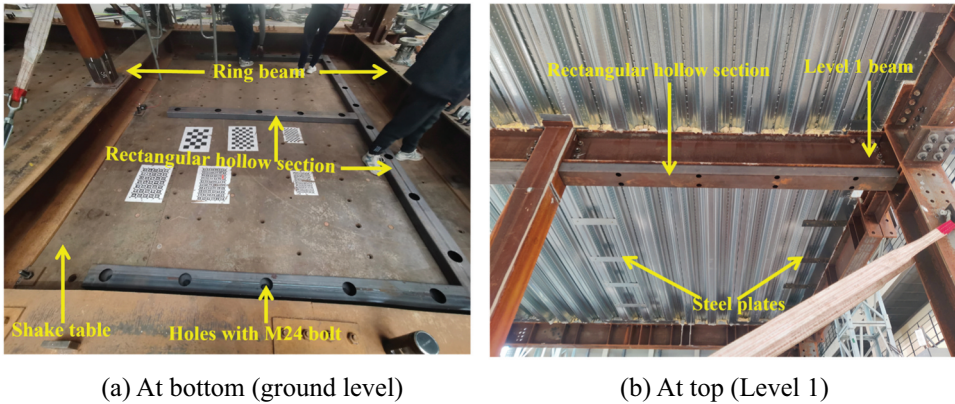


Figure 5. Attachment methods for partition walls.

72 mm × 32 mm timber planks, as depicted in Fig. 7, to prevent their localized deformation as the aluminum angle pushes against them. The remaining intermediate studs were 75 mm × 35 mm × 0.6 mm, spaced at 500 mm intervals. The studs were not mechanically fixed to the top and bottom DSTs, while the plasterboard was screwed only to the studs at 200 mm c/c spacing. The first row of screws at the top was placed 100 mm below the upper slab, whereas the first row at the bottom was positioned 75 mm above the bottom of the wall. The rocking partition wall specimen during installation is shown in Fig. 8, illustrating the top and bottom rocking connection details, and the major components such as the DSTs, aluminum angles, and boxed studs. A vertical gap of approximately 35 mm was maintained between the DST web and the partition wall at the top connection, with minor variations due to construction tolerances, as shown in Fig. 8b.

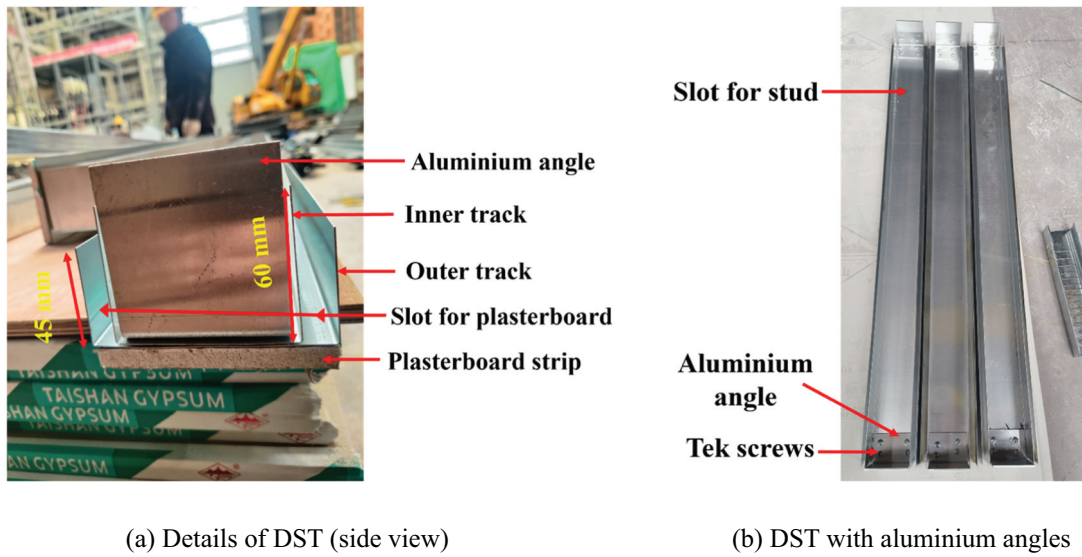


Figure 6. Preparation of the dual-slot tracks (DSTs).

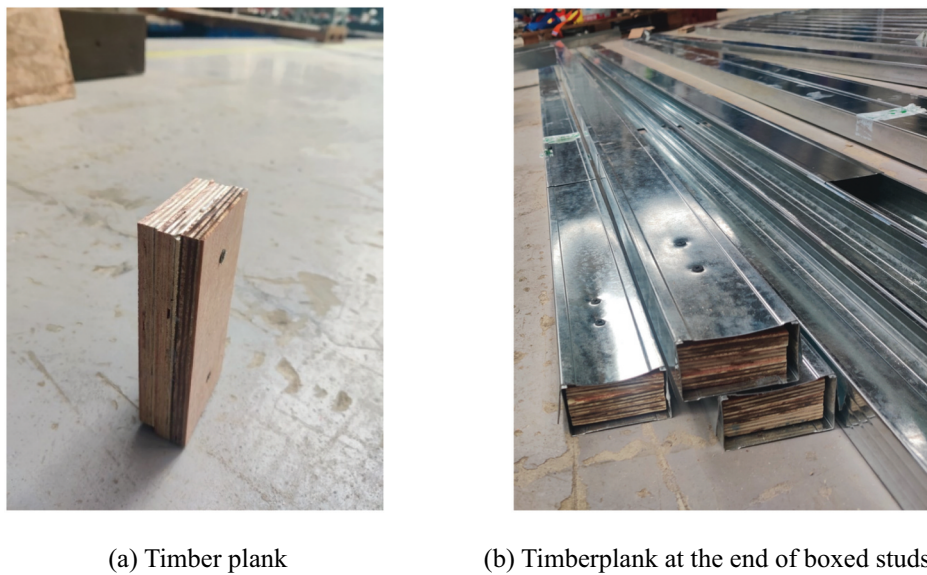


Figure 7. Boxed studs.

A vertical joint, 6–9 mm wide, was provided at the junction of the main wall and return wall in the L-walls (Fig. 9a) and T-walls (Fig. 9b), and at the junction of the planar walls with the central gravity column (Fig. 9c). All joints were sealed with silicone sealant and left to cure for 6 days, except those between the planar walls and the external moment-frame columns, which were left open. Additionally, a steel T-trim was used to fill the intermediate joint gap that was provided between adjacent plasterboard panels, as well as between the steel studs, as shown in Fig. 9d. These intermediate joints were finished with self-adhesive mesh tape, two layers of plaster, and a final topping coat to achieve a construction quality consistent with practical applications (Fig. 10).

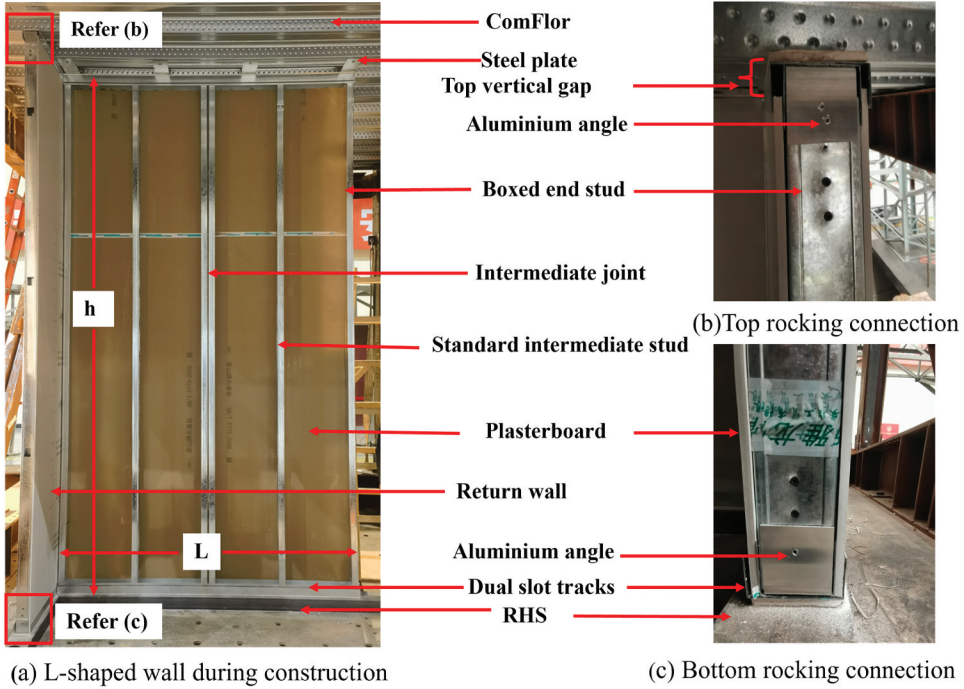


Figure 8. Partition wall specimen showing rocking connection details (RHS = rectangular hollow section; h = wall height; L = wall length).

Structural access panels (SAPs) were installed near the corners of selected walls, at 450 mm from the bottom of the L1X and L2X walls and 150 mm from the top of the L3X and L4X walls. Some of the installed SAPs are shown in Fig. 11. The SAPs provide physical access to critical structural elements and other damage-prone parts of the NSEs (such as cladding connections and partition wall aluminum angle connections), facilitating inspection of potential damage and repair following seismic events without damaging the NSEs (Luo et al. 2024, 2025). A total of eight SAPs were provided at four inspection locations, with one panel installed on each side of the wall. Each SAP was created by cutting a 400 mm × 400 mm opening in the plasterboard and inserting a commercially available access panel consisting of a framed opening fitted with a door and removable hinges.

2.4. Drift Capacity of Walls

2.4.1. In-Plane Drift Capacity

Since the wall heights differed from the inter-story height of the building and varied across different wall configurations, the in-plane drift capacity expression presented in Sec. 2.2 can be modified as follows to account for the effect of wall height:

$$\delta_{in} = \frac{a}{L} \times \frac{h}{H} \quad (1)$$

where “a” and “L” are defined in Sec. 2.2, “h” is the height of the wall (Fig. 8) and “H” is the inter-story height of the structure (3 m).

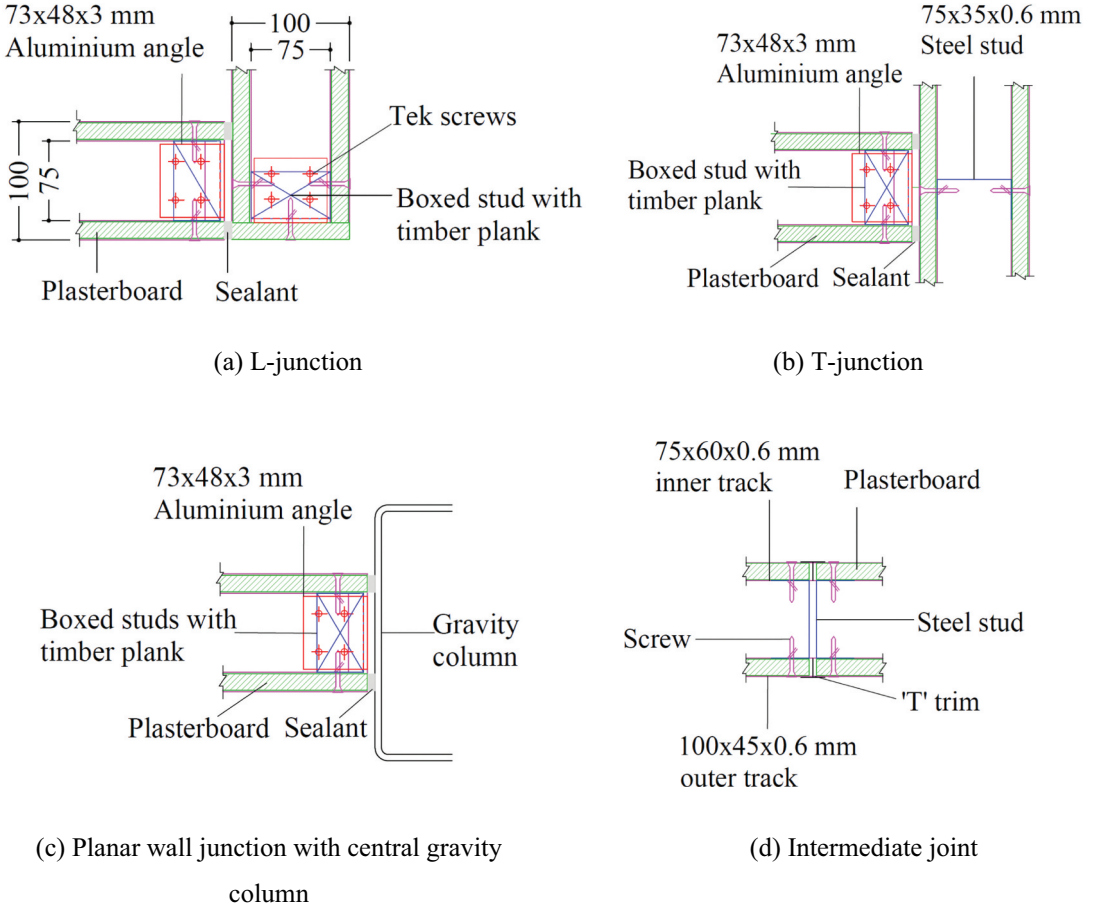


Figure 9. Schematic diagram showing the connection details of the wall specimens (all dimensions are in mm). Sealant width varies from 6 mm to 9 mm.

2.4.2. Out-of-Plane Drift Capacity

During out-of-plane loading, the wall tends to incline about its base, causing the plasterboard to push against the DST flange in the out-of-plane direction. Hence, an overlap between the plasterboard and the DST flange (denoted by “e” in Fig. 2a) is provided to accommodate this out-of-plane drift. The out-of-plane drift capacity of the wall, before the outer track flange engages with the plasterboard, is governed by the slot geometry of the DST and can be estimated using the following relation (Bhatta, Dhakal, and Sullivan 2023b):

$$\delta_{\text{out}} = \frac{s - t_p}{w_f} \quad (2)$$

where “s” is the slot width available for the plasterboard = $\frac{100-75}{2} = 12.5$ mm (refer to Figs. 2a and 9a), ‘t_p’ is the thickness of the plasterboard = 10 mm (refer to Fig. 3a) and ‘w_f’ is the height of the DST flange = 45 mm (refer to Figs. 3a and 6a).

2.4.3. Expected Drift Capacity of Walls

Using Eqs. (1) and (2), the expected in-plane and out-of-plane drift capacities of the three wall configurations were calculated and are summarized in Table 1.

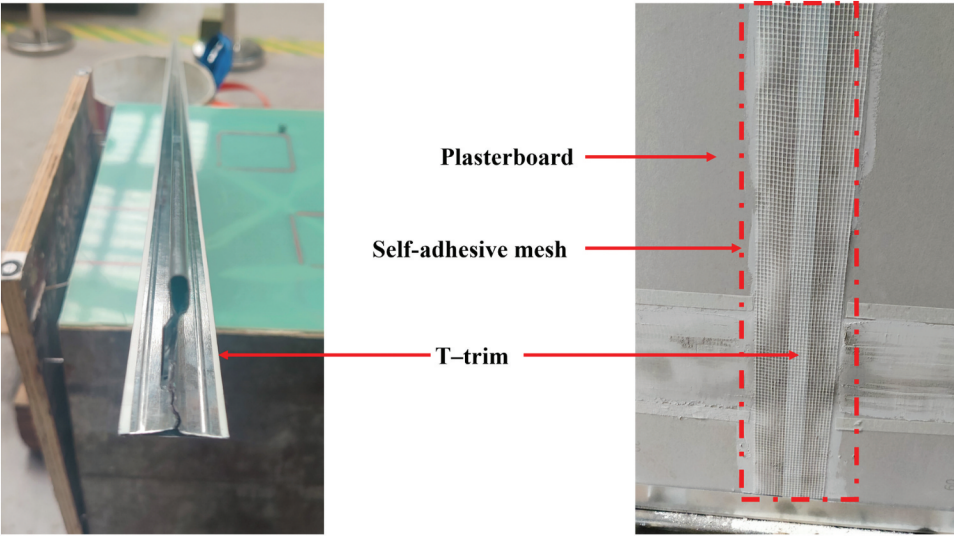
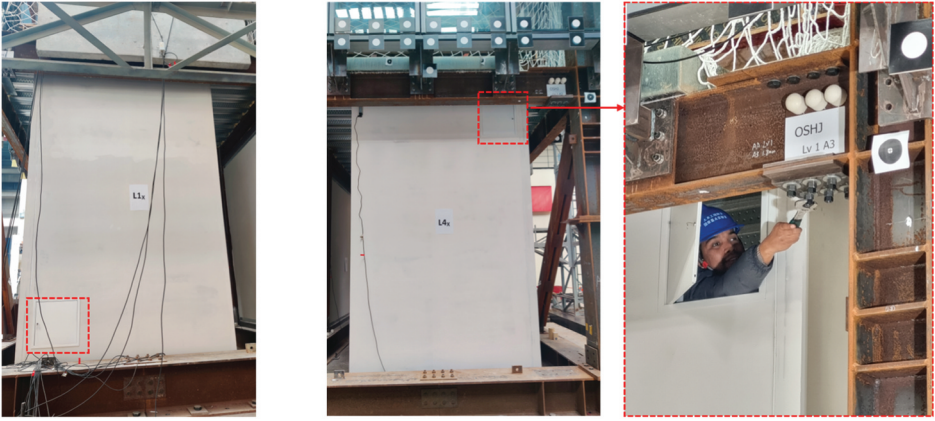


Figure 10. Application of T-trim and self-adhesive mesh at the intermediate joint gap.



(a) Near the base of the wall (b) Near the beam–column joint at the top of the wall

Figure 11. Location of installed structural access panels (SAPs).

Table 1. Expected drift capacity of walls.

Drift capacity	Planar wall	T-wall	L-wall
In-plane	1.49%	2.03%	2.03%
Out-of-plane	5.56%	5.56%	5.56%

Wall height: planar wall = 2.7 m, L- and T-wall = 3.5 m. Refer to Fig. 4 for the length of the different walls.

2.5. Instrumentation Plan

The structural and partition wall responses were recorded using various sensors, as shown in Fig. 4. String potentiometers were installed at different floor levels to measure floor displacements in the longitudinal (NS/X) and transverse (EW/Y) directions, while accelerometers recorded floor accelerations in both horizontal directions and in the vertical direction. The rocking response of the partition

walls was determined from the relative vertical displacement between the wall and the supporting floors, measured by linear potentiometers placed near the top and bottom edges of the plasterboard. Additional potentiometers positioned at the top or bottom of the plasterboard measured relative horizontal displacements between the main wall and the return wall, or between the wall and the boundary column. Out-of-plane wall accelerations were recorded by uniaxial accelerometers mounted at the mid-height of walls W2 and T2X. In addition to these instrumented measurements, horizontal reference lines were marked across the vertical joints between the main wall and the return wall in the L- and T-walls, as well as across the intermediate vertical joints, to visually track relative vertical movement and residual displacements between adjacent plasterboard panels.

2.6. Test Plan

Unidirectional and bidirectional shaking was applied using the Imperial Valley, El Centro 1940, Array #9 earthquake record, scaled to different peak table accelerations (PTAs), as listed in Table 2. The selected ground motion was scaled to match the target spectrum over the period range of interest (0.08–1.97 s). Further details of the scaling procedure are provided in Bagheri (2022) and Chan et al. (2026). The test began with a bidirectional white noise test with a PTA of 0.05 g to identify the dynamic properties, such as natural frequencies and damping ratios of both the structure and the non-structural elements. The same white noise was repeated after each ground motion input to assess changes in the system's dynamic properties throughout the testing sequence. Additionally, intermittent physical inspections of the partition walls, their connections, and sealant joints were conducted throughout the testing to document visible damage (Table 2).

3. Experimental Results

Acceleration and displacement data were recorded during the tests at a sampling frequency of 256 Hz and processed using a fourth-order Butterworth low-pass filter with a cut-off frequency of 50 Hz to

Table 2. Shaking sequence for NSE testing in the ROBUST project.

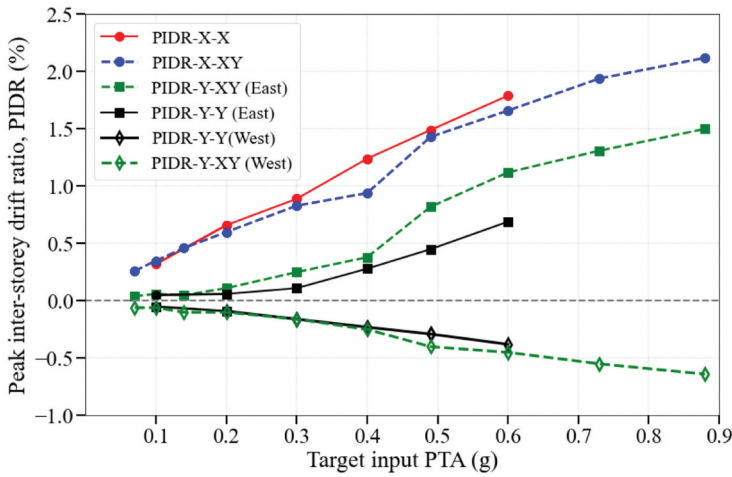
Test number	Shaking direction	Target input PTA (g)		Inspection*
		X	Y	
1	X	0.10	–	
2	Y	–	0.10	
3	XY	0.07	0.07	
4	XY	0.10	0.10	Inspection 1
5	X	0.20	–	
6	Y	–	0.20	
7	XY	0.14	0.14	
8	XY	0.20	0.20	Inspection 2
9	X	0.30	–	
10	Y	–	0.30	
11	XY	0.30	0.30	Inspection 3
12	X	0.40	–	Inspection 4
13	Y	–	0.40	Inspection 5
14	XY	0.40	0.40	Inspection 6
15	X	0.49	–	Inspection 7
16	Y	–	0.49	Inspection 8
17	XY	0.49	0.49	Inspection 9
18	X	0.60	–	Inspection 10
19	Y	–	0.60	
20	XY	0.60	0.60	Inspection 11
21	XY	0.73	0.73	
22	XY	0.88	0.88	Inspection 12

*Visual inspections were conducted after the indicated shaking events, and the observed specimen damage was documented.

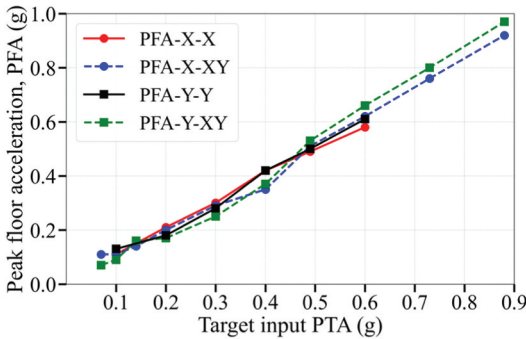
remove high-frequency noise. The filtered data were subsequently used to determine the seismic responses of the structure and the partition walls, as well as the dynamic characteristics of the partition wall system.

3.1. Peak Seismic Response of the Structure

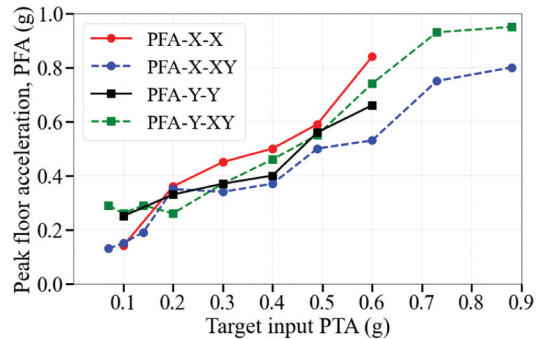
Figure 12 shows the peak inter-story drift ratios (PIDRs) at Storey 1 and peak floor accelerations (PFAs) at the ground level and Level 1 of the structure for different ground motion intensities applied in the longitudinal (NS/X) and transverse (EW/Y) directions. Along the longitudinal (NS/X) direction, the drift response was nearly symmetric between positive and negative drift cycles (Fig. 13a). Therefore, the PIDRs were obtained by averaging the absolute peak values recorded by the displacement sensors. However, the peaks of negative (towards the west) and positive (towards the east) drift cycles along the transverse (EW/Y) direction differed, particularly after the 0.49 g bidirectional input, as shown in



(a) Peak inter-story drift ratios at Storey 1.



(b) Peak floor accelerations at ground level



(c) Peak floor accelerations at Level 1

Figure 12. Peak structural response at Storey 1.

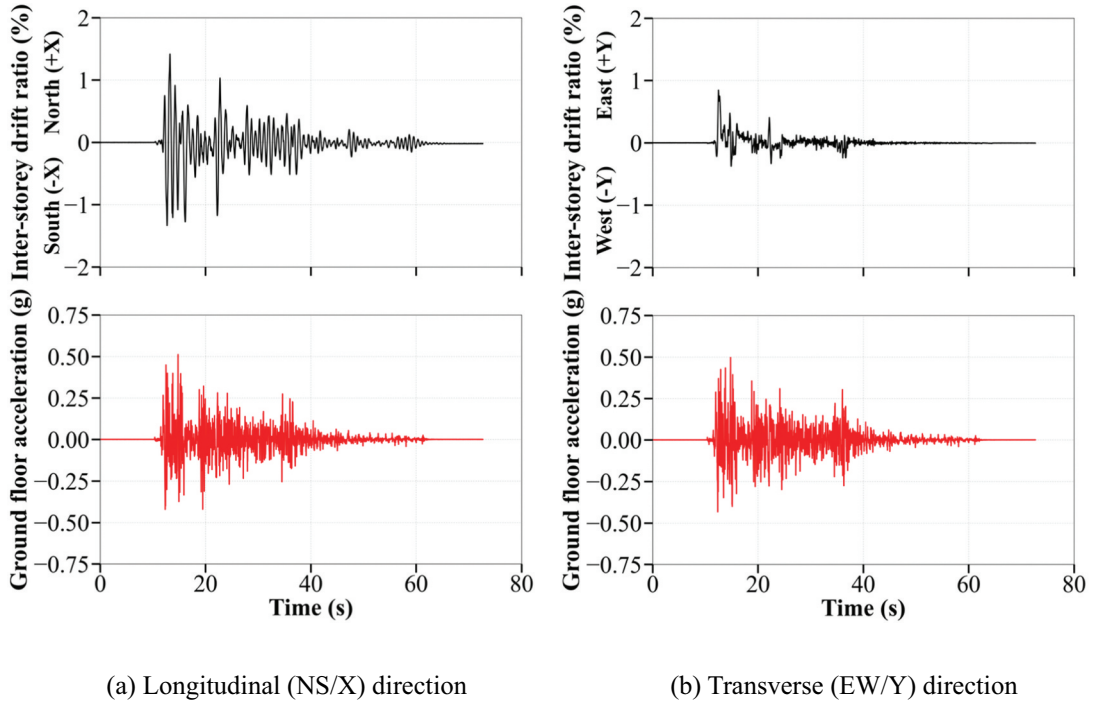


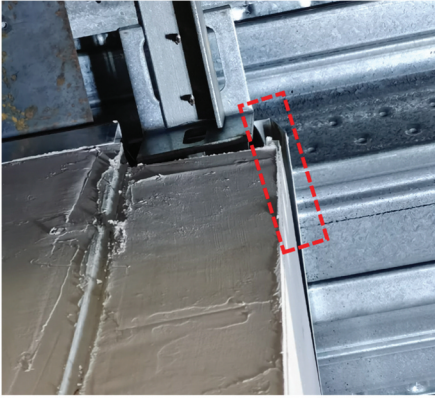
Figure 13. Inter-storey drift ratios at Storey 1 recorded by P2(X) and P2(Y) and floor accelerations at ground level recorded by AS2 under 0.49 g bidirectional shaking (see Figure 4 for instrument location).

Fig. 13b, highlighting the need to assess drift demands in the east and west directions separately. Hence, the PIDRs along the transverse (EW/Y) direction were determined by separately averaging the positive and negative peak values recorded by the displacement sensors. Unlike the drift response, the acceleration response was nearly symmetric between positive and negative cycles in both the longitudinal and transverse directions (Fig. 13). Therefore, the PFA at each floor level was determined by averaging the absolute peak accelerations recorded by the two accelerometers in each direction. It should be noted that accelerometer AS2 (refer to Fig. 4 for its location) was not properly oriented until the 0.40 g longitudinal shaking (test 12; Table 2); therefore, data recorded before this shaking event were deemed unreliable and excluded from the analysis.

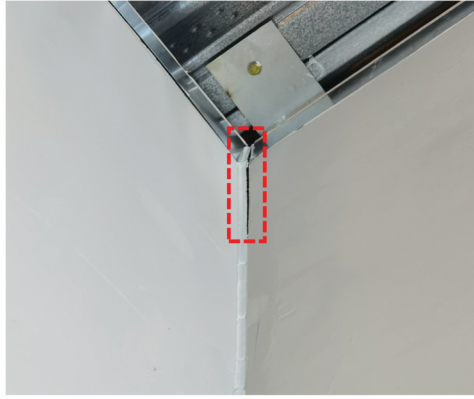
The results show that PIDRs increased with shaking intensity under both unidirectional and bidirectional shaking. The longitudinal (NS/X) direction consistently showed higher PIDRs than the transverse (EW/Y) direction. This difference can be attributed to the presence of the concentrically braced frame in the transverse direction, which increased lateral stiffness and limited the drift response. The maximum PIDR of 2.12% occurred in the longitudinal direction compared with 1.50% in the transverse direction during 0.88 g bidirectional shaking. In contrast, the maximum PFA was 0.97 g in the transverse direction, slightly exceeding the 0.92 g recorded in the longitudinal direction under the same shaking event.

3.2. Observed Seismic Damage

Any physical damage to the partition walls was documented through photographs and recorded in damage description sheets during visual inspections. Based on these observations, the planar walls exhibited better performance compared to the L- and T-walls, with no damage observed throughout the testing program. In contrast, the first observed damage in the L- and T-walls was localized



(a) Localised deformation of the top
DST flange (1.24%)



(b) Sealant debonding at the top of the
junction between the main and return wall
(1.49%)

Figure 14. Damage observed in L- and T-walls at different peak inter-storey drift ratios.

deformation of the DST flanges in the return walls, as illustrated in Fig. 14a. This deformation was first observed at a PIDR of 1.24% during 0.40 g shaking in the longitudinal (NS/X) direction. After the 0.49 g longitudinal (NS/X) shaking, sealant debonding between the main and return walls was observed in both the L- and T-wall configurations. These openings first appeared at a PIDR of 1.49% at the top of the junctions of L1Y–L1X, L4Y–L4X, T1Y–T1X, and T2Y–T2X, as shown in Fig. 14b. Additionally, the aluminum angles, both with and without external steel angles, performed well throughout the testing sequence, with no visible deformation. Moreover, no noticeable influence of the SAPs on the damage patterns of the partition walls was observed during the tests.

After the 0.49 g longitudinal (NS/X) shaking (1.49% PIDR), localized wear of the plasterboard surface at the top of the L2X wall and deformation of the top DST flange were observed (Fig. 15a). Due to a construction error, the overlap of the plasterboard within the DST was approximately 5 mm, and



(a) Scraping off the plasterboard and
deformation of the top DST flange
(1.49%)



(b) Localised plasterboard damage with
spalling and detachment (2.12%)

Figure 15. Localised damage observed at the top edge of L2X wall and corresponding peak inter-storey drifts.

the preceding 0.40 g bidirectional shaking prevented the plasterboard from fully returning to its original position within the DST. During subsequent rocking motions, the plasterboard repeatedly pressed against the sharp edge of the DST flange and slid against it, generating frictional wear and causing localized plasterboard damage (extending over an area of approximately 200 mm × 35 mm), as shown in Fig. 15b. This unintended damage was not directly related to inter-story drift demand but was instead construction-related; it has, therefore, been excluded from the damage state classification. Such plasterboard damage would be concealed within the plenum space in commercial buildings.

3.3. Damage States

Three damage states (DSs) were defined according to the repair needs and their impact on the intended rocking behavior and/or functionality (fire resistance, acoustics, thermal insulation) of the wall system, based on the relevant literature (Bhatta, Cook, and Sattar 2025; Shrestha et al. 2026; Taghavi and Miranda 2003). The definitions of these DSs are discussed in the following subsections.

3.3.1. Superficial (DS₁)

The superficial damage state refers primarily to aesthetic damage that may not be immediately noticeable. This level of damage does not affect the intended rocking behavior and/or functionality of the wall system. It either requires no repair or only minor, non-urgent repair. Some examples include screw impressions, localized deformation of the DST flanges, and thin plaster cracks between adjacent plasterboard panels.

3.3.2. Moderate (DS₂)

The moderate damage state involves damage that would be easily noticeable and partially impairs functionality of the wall system and may affect its intended rocking behavior. Therefore, repairs are required to restore the functionality and seismic capacity of the walls, but they may be scheduled during regular maintenance, with temporary measures applied if necessary to prevent progressive damage. Typical examples of this damage state include corner crushing and cracking of plasterboard, sealant tearing or debonding at wall junctions, and flexural yielding of aluminum angles. The required repair actions may include replacing the damaged plasterboard, potentially replacing a portion of the framing, and repainting or resealing the joints.

3.3.3. Severe (DS₃)

The severe damage state involves damage that compromises the system's rocking behavior and/or functionality and requires urgent repair. Typical examples include severe damage to plasterboard, wall separation at junctions, damage to the wall framing, and tearing of the DST through the slab fasteners. The required repair involves replacement of the partition wall, including the steel framing and plasterboard.

For individual wall components, the assigned DS depends on the observed severity and extent of the damage. In this study, for the sealant joints, DS₁ corresponds to wrinkles developed in the sealant, whereas DS₂ corresponds to localized sealant debonding or tearing that may result in the loss of acoustic/fire sealing while the partition wall remains intact (Shrestha et al. 2026). DS₃ is characterized by extensive sealant failure together with significant joint opening or instability of the wall due to damage requiring replacement. Similarly, for the DST, DS₁ corresponds to localized DST flange deformation, DS₂ to widespread flange deformation that could impair the intended rocking behavior, and DS₃ to tearing or failure of the track around the slab fasteners (Magliulo et al. 2022). Based on this classification, the observed damage was categorized into different DS, and inter-story drift ratios at which each DS was first observed during the tests were correlated, as shown in Table 3. For each wall configuration, the drift ratio corresponding to a given DS was taken as the minimum in-plane inter-story drift ratio. Additionally, the proposed DSs were correlated with the non-structural performance levels defined in ASCE/SEI 41–23 (2023). According to ASCE/SEI 41–23 (2023), the Operational

Table 3. Peak in-plane inter-story drift ratios at which damage states were first observed.

DS	Non-structural performance levels	Damage typology	Peak in-plane inter-story drift ratio (%)		
			Planar	T-wall	L-wall
Superficial (DS ₁)	Operational	Localized deformation of DST flange	>1.50	1.24	1.24
Moderate (DS ₂)	Operational – Position Retention	Sealant debonding at wall junctions that can cause loss of wall functionality while the partition wall remained intact	>1.50	1.49	1.49
Severe (DS ₃)	Life Safety	No specific damage was observed during testing	>1.50	>2.12	>2.12

Performance Level indicates that non-structural components retain their functionality in the building that they provided before the earthquake. The Position Retention Performance Level permits damage that may prevent the components from functioning immediately, but they remain secured in place. The Life Safety Performance Level permits damage, provided that the resulting consequences do not pose a life-safety threat. As summarized in Table 3, Superficial (DS₁) corresponds to the Operational Performance Level, as only localized deformation of the DST flange was observed, and the partition wall remained fully functional without requiring repair. Moderate (DS₂) corresponds to a performance level between the Operational and Position Retention Performance Levels, since the wall remained secured in place but experienced a partial loss of functionality due to sealant debonding at the wall junctions. The resulting damage was easily repairable and could be addressed during routine maintenance rather than requiring immediate intervention. Throughout the testing program, no damage corresponding to the Life Safety Performance Level was observed, as no instability, loss of anchorage or falling hazards occurred.

3.4. Comparison with Existing Low-Damage Systems

Table 4 compares the inter-story drifts corresponding to the different DS of the rocking wall system tested in this study with other low-damage partition wall systems reported in the literature. Damage observations reported in previous studies were reclassified to align with the DS definitions presented

Table 4. Comparison of seismic performance and damage progression in existing low-damage systems.

Author	Low-damage system	Wall configuration	Loading type	Design drift ^a (%)	Inter-story drift ratio (%) [Demand-to-Capacity ratio (DCR)]		
					DS ₁	DS ₂	DS ₃
Davies et al. (2011)	Flexible track (Double slip track)	T-wall	Quasi-static	–	1.00	1.35	1.84
Magliulo et al. (2014)	Seismic gap	Planar wall	Dynamic	0.60	–	(0.58–0.98) ^b [0.97–1.63]	>1.08 [1.80]
Fiorino, Bucciero, and Landolfo (2019)	Seismic gap	Planar wall	Dynamic	1.43	0.89 [0.62]	1.39 ^c [0.97]	>4.33 [3.02]
Araya-Letelier, Miranda, and Deierlein (2019)	Friction/Sliding connection	I-wall	Quasi-static	1.30	>1.50 [1.15]	2.10 [1.62]	3.00 [2.31]
Hasani and Ryan (2021)	Telescoping detail	Planar wall	Quasi-static	–	0.43	>5.00	>5.00
This study	Rocking DST	Planar	Dynamic	1.49	>1.50 [1.00]	>1.50 [1.00]	>1.50 [1.00]
		T– and L– walls		2.03	1.24 [0.61]	1.49 [0.73]	>2.12 [1.04]

^aDesign drifts were calculated using the available data from previous studies.

^bA drift of 0.58% corresponds to silicone sealant detachment, whereas a drift of 0.98% corresponds to the corner crushing of the plasterboard.

^cThe minimum value for the east- and west-side walls reported by Fiorino, Bucciero, and Landolfo (2019) was adopted.

in Sec. 3.3 to facilitate a consistent comparison. As shown in Table 4, the rocking wall system achieved seismic performance comparable to existing low-damage partition systems. Considering the demand-to-capacity ratio (DCR), defined as the inter-story drift ratio at a given DS divided by the design drift, the L- and T-walls reached DS₁ and DS₂ at DCRs of 0.61 and 0.73, respectively, while DS₃ was not reached up to a DCR greater than 1.04. The planar rocking wall remained undamaged up to the in-plane PIDR of 1.50%, corresponding to a DCR of approximately 1.00, indicating that no damage was observed up to its design drift. Overall, the DCR provided a reasonable indication of the expected performance, as the rocking walls sustained either no damage or moderate damage up to the design drift and avoided severe damage slightly beyond it. In comparison, the seismic-gap system reported by Fiorino, Bucciero, and Landolfo (2019) exhibited DS₁ and DS₂ at DCRs of 0.62 and 0.97, respectively, whereas the friction/sliding connection tested by Araya-Letelier, Miranda, and Deierlein (2019) delayed damage initiation until a DCR greater than 1.15. It is to be noted that while efforts were made to ensure a consistent comparison by reclassifying the reported damage observations into a common damage state framework and comparing the corresponding DCRs, differences in loading protocols, wall configurations, construction details (e.g. stud size and spacing, track size, fastener spacing, plasterboard thickness), and documentation of damage progression may contribute to variations in the reported inter-story drift ratios associated with each DS.

3.5. Rocking Response of the Partition Walls

The rocking response of the partition walls was examined by comparing experimentally measured rocking displacements with theoretically predicted values at various PIDRs. The experimental rocking displacement for each shaking level was obtained as the peak vertical displacement (uplift) recorded by the instrument. The predicted rocking displacement “ y ” for any particular shaking event was calculated assuming rigid-body motion by multiplying the horizontal distance “ r ” from the far end of the wall (point of rotation) to the uplift measurement point (instrument location) and the wall rotation “ θ_w ” as given by $y = \theta_w \times r$ (Bhatta et al. 2020). The wall rotation can be obtained using the relationship between the story drift ratio “ θ ” and wall rotation “ θ_w ” (Bhatta, Dhakal, and Sullivan 2023a):

$$\theta_w = \theta \times \frac{H}{h} \quad (3)$$

where “ H ” is the inter-story height of the structure and “ h ” is the height of the wall.

Figure 16 shows the peak rocking displacements of the partition walls recorded by the instruments at different shaking levels, plotted against the corresponding PIDRs. Overall, the experimentally measured peak rocking displacements are lower than the theoretical predictions associated with ideal rigid-body motion. This discrepancy can be attributed to several factors, such as the presence of inevitable gaps between the studs and aluminum angles during construction, which limit the effective drift demand transferred to the wall. Relative slip between the plasterboard and the studs, as observed by Bhatta and Dhakal (2026), may further reduce the rocking response. Energy dissipation mechanisms neglected in the rigid-body assumption, such as friction at interfaces (e.g. contact of the plasterboard and studs with the DSTs), local crushing or wear at the rocking points, and minor flexural deformation of the aluminum angles, may also contribute to the reduced rocking amplitudes. Although no visible deformation of the aluminum angles was observed during the tests, any damage to these components at the wall junctions could remain concealed until the plasterboard sustains significant damage requiring repair.

For the L- and T-wall configurations, the peak rocking displacements at the free ends of the main walls (recorded by the instruments placed at the top of the walls, see Fig. 4 for instrument locations) are closer to the theoretically predicted values, whereas the peak rocking displacements measured near the wall junctions (recorded by the instruments placed at the bottom of the walls) show greater deviation and remain well below the theoretical predictions, as shown in Fig. 16a,b. This behavior can be attributed to the effect of the sealant and the interaction between the main wall and the return wall.

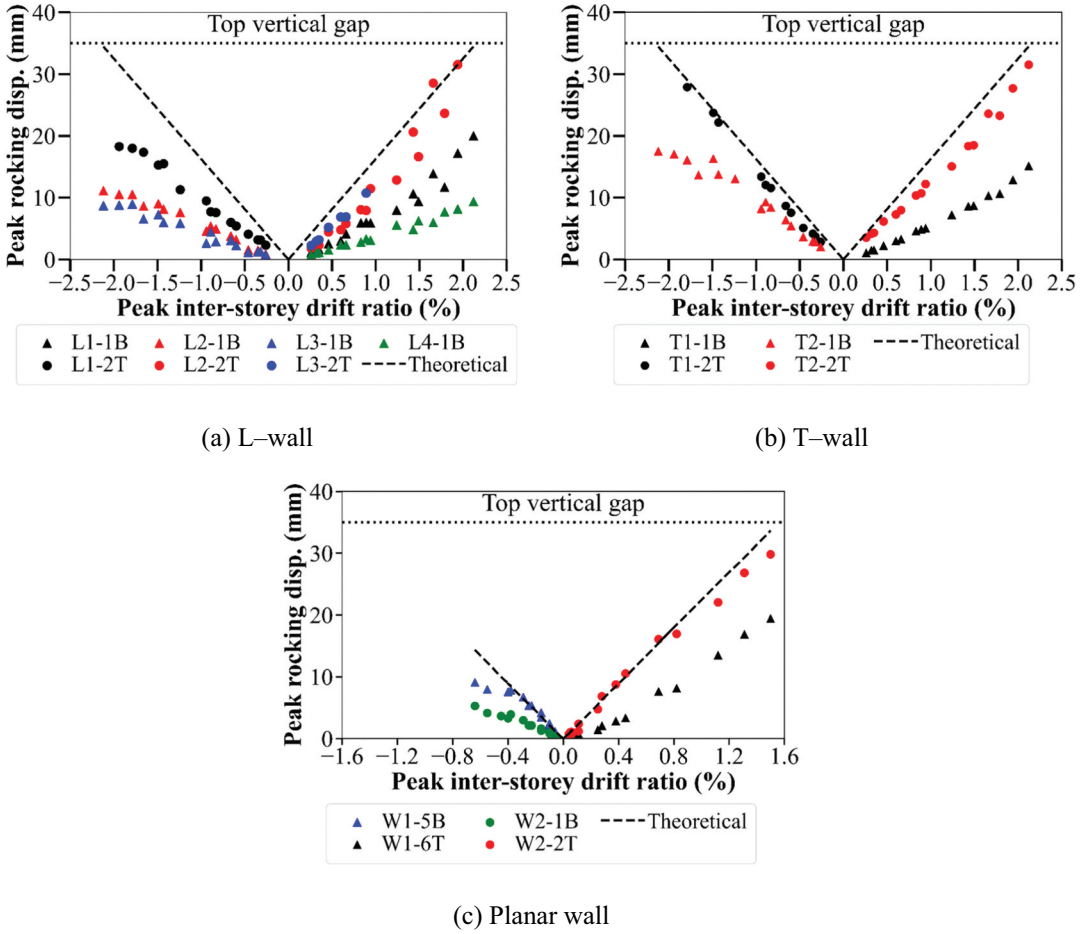


Figure 16. Comparison between experimentally measured and theoretically predicted rocking displacements. The horizontal dotted lines represent the 35 mm top vertical gap (refer to Figure 4 for instrument locations).

Consistent with this observation, Bhatta and Dhakal (2026) reported that inter-storey drift was primarily accommodated by flexing of the flanges in the DST of the return wall. Additionally, Fig. 16c shows that the peak rocking displacements of the planar walls at the bottom (measured by instruments W1-5B and W2-1B) are smaller than those at the top (measured by instruments W1-6T and W2-2T). This trend corresponds to the difference in the PIDRs, which are smaller in the west direction than in the east direction of the structure (Fig. 13b). Figure 16c also illustrates that the rocking displacements measured at the wall ends with open vertical joints closely follow the theoretical rocking line, whereas those measured at the wall ends with sealant-filled joints (Fig. 9c) deviate more and are noticeably below it.

An exception to this general trend was observed for the L1X wall. The peak rocking displacement recorded at the free end of the L1X wall (instrument L1-2T) was comparable to that measured at the wall junction (instrument L1-1B). This behavior was primarily attributed to a construction error that left a clearance of approximately 15 mm between the inner track flange and the screws, as shown in Fig. 17a. At smaller drifts, the wall could rock freely within this clearance. Once the rocking displacement exceeded the available space, the screws came into contact with the inner track, resulting in local deformation of the track and limiting the free rocking motion, as shown in Fig. 17b. This interaction may also have contributed to relative movement between the plasterboard and the studs. These

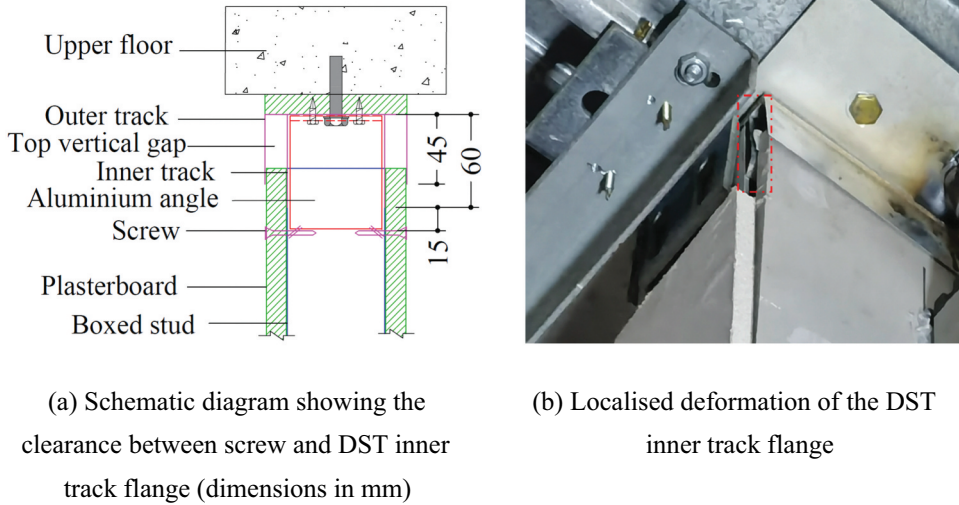


Figure 17. Localised deformation of the DST inner track flange at the free end of the L1X wall.

observations suggest that screw–track engagement influenced the measured rocking displacements at the free end of the L1X wall once the clearance limit was reached. It should also be noted that most of the instruments fixed to the top of the walls detached during the higher intensity shaking events; consequently, no data are available for the 0.88 g bidirectional shaking (2.12% PIDR). Particularly, instrument L3-2T detached after the 0.30 g bidirectional shaking (0.83% PIDR), whereas instrument L4-2T detached after the first few shaking events. Hence, the data were used for L3-2T only up to 0.83% PIDR, and data for L4-2T were excluded from all analyses.

The experimental rocking displacements measured at the wall ends with and without sealant highlight the influence of the sealant on the rocking response of the walls. To quantify this effect, Figure 18 presents the effective peak rocking displacement due to the presence of sealant, defined as:

$$\text{Effective peak rocking displacement (\%)} = \frac{\text{PRD}(\text{sealant})}{\text{PRD}(\text{no – sealant})} \quad (4)$$

where PRD (sealant) and PRD (no-sealant) are the peak rocking displacements measured at the wall ends with sealant and without sealant, respectively. On average, the rocking displacement of the walls with sealant was approximately 50% of that without sealant, with individual values ranging from 34% to 73%. Consistent with the findings reported by Bhatta, Dhakal, and Sullivan (2023b), these results suggest that the sealant delays closure of the top vertical gap and subsequent contact between the wall and the top DST web, potentially postponing the onset of damage to the plasterboard and studs. Moreover, as the adjacent walls rock and attempt to separate, shear, and compression/tensile deformation develop within the sealant layer, which may eventually result in tearing or debonding of the sealant. Consequently, damage may initially be concentrated within the sealant, rather than in the plasterboard or framing members, altering the damage progression of the wall system. This behavior was reflected in the observed damage pattern during the tests, where the moderate damage state (DS₂), characterized by localized sealant debonding at the wall junctions, occurred without observable damage to the plasterboard or studs. Such behavior may be advantageous because it suggests that the sealant acts as a sacrificial component. As a result, post-earthquake repair would primarily involve replacing the damaged sealant rather than repairing primary components of the wall. It should be noted that commercially available sealants may differ in their mechanical properties; therefore, care should be taken to select a suitable sealant to avoid unexpected system behavior.

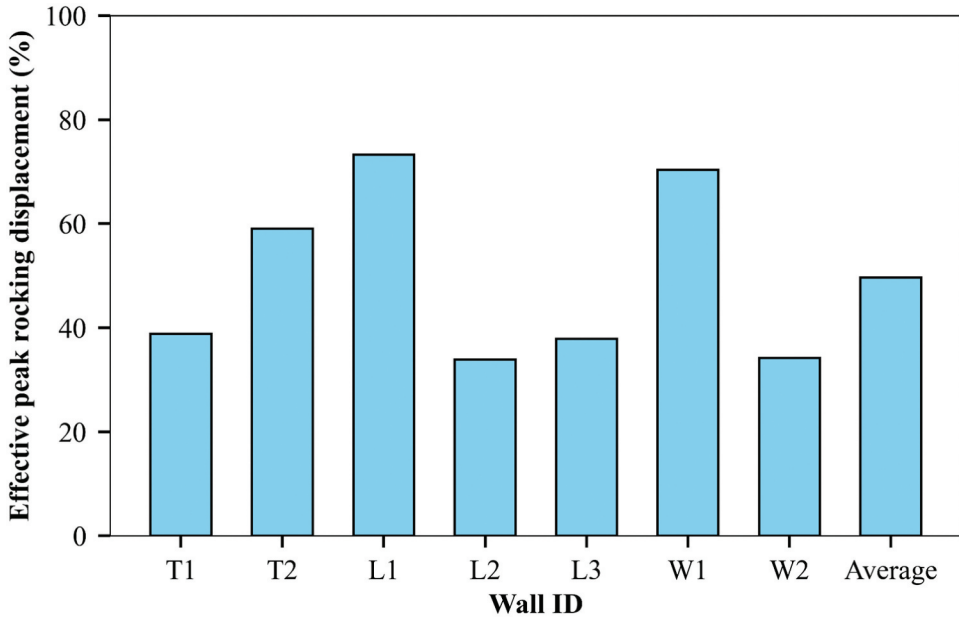


Figure 18. Effective peak rocking displacement due to the presence of sealant.

3.6. Interaction Between the Main Wall and the Return Wall

The experimental results demonstrate that the rocking response of the return walls is influenced by the main wall, and vice versa. Since the return walls were oriented along the transverse direction, their rocking during longitudinal shaking (i.e. under out-of-plane loading) would be expected to remain negligible. However, the measured response shows that the return walls exhibited rocking displacements nearly equal to those of the main walls. This indicates that the return walls did not behave independently; instead, they lifted together with the main walls due to their interaction at the junctions. This behavior can be illustrated by comparing the rocking responses of the main walls (L1-1B for the L1-wall and T2-1B for the T2-wall) with the rocking responses measured in their corresponding return walls (L1-5B and L1-6T for the L1-wall, and T2-5B and T2-6T for the T2-wall). For the L1-wall, the peak rocking displacements recorded by instruments L1-1B and L1-5B were nearly identical, whereas those recorded by instrument L1-6T, located near the far end of the return wall, were smaller, as shown in Fig. 19a. This suggests that the portion of the return wall closer to the main wall engaged more actively in the rocking, while the far end engaged less. In contrast, the T2-wall exhibited more uniform behavior: the rocking displacements of T2-1B (main wall) and T2-5B (return wall) were nearly equal, and T2-6T also showed similar levels of uplift, as depicted in Fig. 19b. This difference arises because, in the T2-wall, the main wall was located at the center of the return wall, providing symmetrical restraint and resulting in more uniform rocking at both ends. Furthermore, comparison of the rocking response of the main and return walls of the L1-wall and T2-wall under transverse shaking showed that their peak rocking displacements remained nearly equal. This consistency across both shaking directions indicates that the main and return walls behaved as an integrated system, lifting together regardless of whether the loading was applied in-plane or out-of-plane.

3.7. Relative Displacement Between the Main Wall and Adjacent Boundary Elements

Table 5 summarizes the peak relative horizontal displacements (i.e. the opening or closing of the vertical joint gap) and the cumulative residual displacements between the main wall and the adjacent

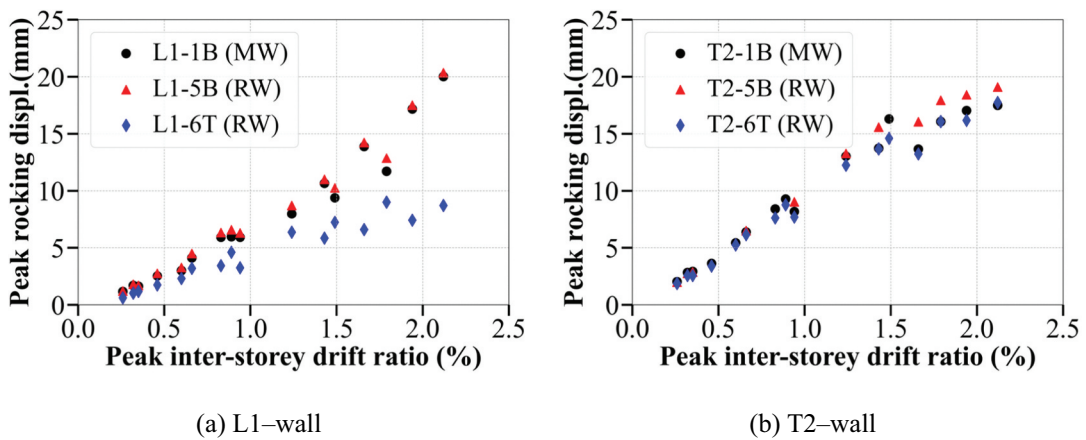


Figure 19. Peak rocking displacement of the main wall (MW) and return wall (RW).

boundary elements (return wall or column). A positive gap variation indicates gap opening, whereas a negative value indicates gap closure relative to the initial position. The peak opening and closing displacements for each instrument correspond to the largest positive and negative values, respectively, recorded over the entire shaking sequence. The cumulative residual displacement was determined by summing the residual displacements measured after each shaking event from the start of testing to the final shaking.

For all wall configurations, the peak opening or closing of the vertical joint gaps remained below 10 mm, while the absolute cumulative residual displacement did not exceed 3.50 mm throughout the testing. These small values indicate that the rocking mechanism effectively accommodated the imposed inter-storey drift demands without inducing significant permanent displacement at the interfaces. As shown in Table 5, sealant debonding at the top of the vertical joints occurred when the gap opening reached approximately 1.8–6.8 mm, depending on the walls. Since sealant debonding was localized at the top of the wall junctions, it did not affect the overall rocking response of the wall. Furthermore, a larger measured gap opening or closing does not necessarily indicate plasterboard cracking or junction separation. This is because the wall junctions incorporated vertical joints with

Table 5. Summary of relative horizontal displacement between the main wall and adjacent boundary elements throughout the testing sequence.

Instrument ID	Opening of vertical joint gap (mm)	Closing vertical joint gap (mm)	Cumulative residual displacement (mm)	Opening when sealant debonding from plasterboard was first observed (mm)
Walls along the longitudinal (NS/X) direction				
L1-3B	2.85	−1.29	−2.36	—
L1-4T	9.62	−8.05	2.09	4.62 (1.49%) [#]
L2-3T	8.44	−2.29	0.13	4.67 (1.79%)
L3-3T	6.93	−1.38	0.14	6.84 (1.79%)
L4-3T	2.14	−2.04	2.61	NA* (1.49%)
T1-3T	7.91	−4.55	3.36	5.34 (1.49%)
T2-3B	1.57	−8.65	0.04	—
T2-4T	4.09	−1.41	0.01	1.83 (1.49%)
Walls along the transverse (EW/Y) direction				
W1-1B	4.59	−1.29	0.81	—
W1-2T	1.82	−2.86	1.00	—
W1-3B	0.76	−6.41	−0.06	—
W1-4T	3.76	−2.90	−0.68	—

[#]Numerical values in parentheses indicate the peak in-plane inter-storey drift corresponding to the first observation of sealant debonding.

*NA = Reliable data were not available after the 0.14 g bidirectional (XY) shaking.

gaps of 6–9 mm filled with sealant and the measured opening or closing were predominantly due to stretched or compressed sealants. Moreover, the horizontal reference lines marked across the vertical joints of the wall interfaces remained intact even after the largest shaking event, indicating negligible relative permanent vertical displacement between adjacent walls and confirming that the walls returned to their original position after each shaking level.

3.8. Out-of-Plane Acceleration Amplification Factor

Out-of-plane acceleration amplification factors for the partition walls were calculated by normalizing the peak acceleration measured at the mid-height of each wall by the PFAs measured in the same direction at the two supporting floor levels. The top and bottom acceleration amplification factors were obtained by taking the ratio of the peak acceleration recorded by A1 for the T-wall or A2 for the planar wall (refer to Fig. 4 for instrument locations) to the corresponding PFAs at Level 1 and the ground level, respectively.

The results show that both the planar wall (A2 accelerometer) and the T-wall (A1 accelerometer) experienced out-of-plane acceleration demands higher than the corresponding PFAs, as shown in Fig. 20, except for a single case where the top amplification factor for the T-wall was slightly less than unity at a PFA of 0.29 g. The horizontal axis in Fig. 20a represents the PFAs recorded at the corresponding supporting floor: ground level for the bottom amplification factor and Level 1 for the top amplification factor. The box plot in Fig. 20b presents the statistical distribution of the out-of-plane amplification factors. The T-wall exhibited median amplification factors of 3.50 and 2.52 relative to the bottom and top floors, respectively, which were higher than the corresponding factors of 1.98 and 1.58 for the planar wall. The observed height-dependent trend in acceleration amplification is consistent with the provisions of ASCE/SEI 7–22 (2022), which specify different Component Resonance Ductility Factor (C_{AR}) values based on wall height. The standard prescribes a lower amplification factor ($C_{AR} = 1.0$) for light-frame partition walls with heights less than or equal to 2.74 m and a higher factor ($C_{AR} = 1.4$) for taller walls. In the present study, the tested planar wall was 2.7 m high, closer to the threshold height, while the T-wall height was 3.5 m. Accordingly, the higher amplification observed for the T-wall is consistent with the height-dependent trend specified by the standard. However, the experimentally obtained median amplification factors for both wall types exceeded the corresponding values prescribed by ASCE/SEI 7–22 (2022). Thus, although the standard

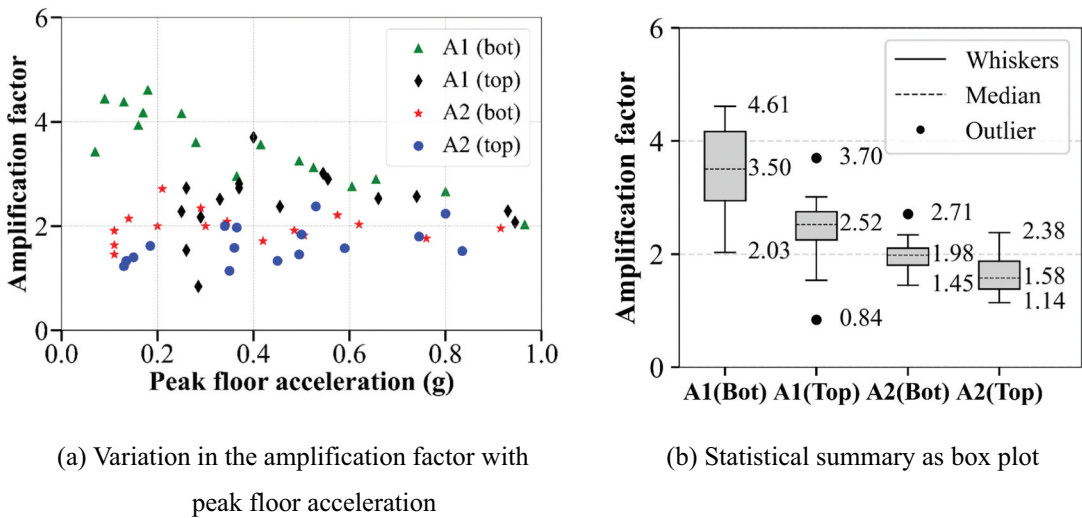


Figure 20. Acceleration amplification factor obtained at the mid-height of the partition walls.

captures the influence of wall height, its prescribed values may underestimate the acceleration amplification demands of the tested rocking wall system. The median acceleration amplification factors for both wall types fall within the bounds prescribed by the New Zealand seismic standard (SNZ TS 1170.5, 2025), which recommends a maximum value of 4.0 for flexible components and 1.0 for rigid components.

3.9. Dynamic Characterization

3.9.1. Natural Frequency

The out-of-plane natural frequencies of the partition walls were identified using frequency response functions (FRFs). For each white noise test, these FRFs were derived as the ratio of the Fourier transform (FT) amplitudes of the out-of-plane acceleration record of the partition wall to the FT amplitudes of the supporting floor accelerations in the same direction. Although multiple peaks were observed in the FRFs, one dominant peak was identified with respect to both the ground level and Level 1 accelerations, as shown in Fig. 21. The frequency corresponding to this dominant peak was taken as the out-of-plane natural frequency of the wall.

As shown in Fig. 22, the out-of-plane natural frequency (and corresponding modal period) ranges from 9.51 Hz (0.11 s) to 7.79 Hz (0.13 s) with a median value of 8.44 Hz (0.12 s) for the T-wall and from 13.18 Hz (0.07 s) to 11.90 Hz (0.08 s) with a median value of 12.50 Hz (0.08 s) for the planar wall throughout the testing sequence. Figure 22 also shows that the natural frequency of the planar wall is higher than that of the T-wall, which can be attributed to the higher stiffness provided by its shorter height. Furthermore, the modal periods of the structure are 0.67 s in the longitudinal (NS/X) direction and 0.26 s in the transverse (EW/Y) direction (Hashemi 2026). Comparison of the planar wall modal period with the longitudinal structural modal period, and the T-wall modal period with the transverse structural modal period confirms that the partition wall modal periods are much shorter than those of the structure. Therefore, the observed acceleration amplification in the partition walls is unlikely to have resulted from modal period resonance.

The identified natural frequencies exhibited relatively low variability throughout the testing program, with coefficients of variation of approximately 5.46% and 2.93% for the T-wall and planar wall, respectively. Although a frequency reduction occurred during the testing sequence, direct correlation between the occurrence of specific damage states (DS₁ or DS₂) and changes in natural

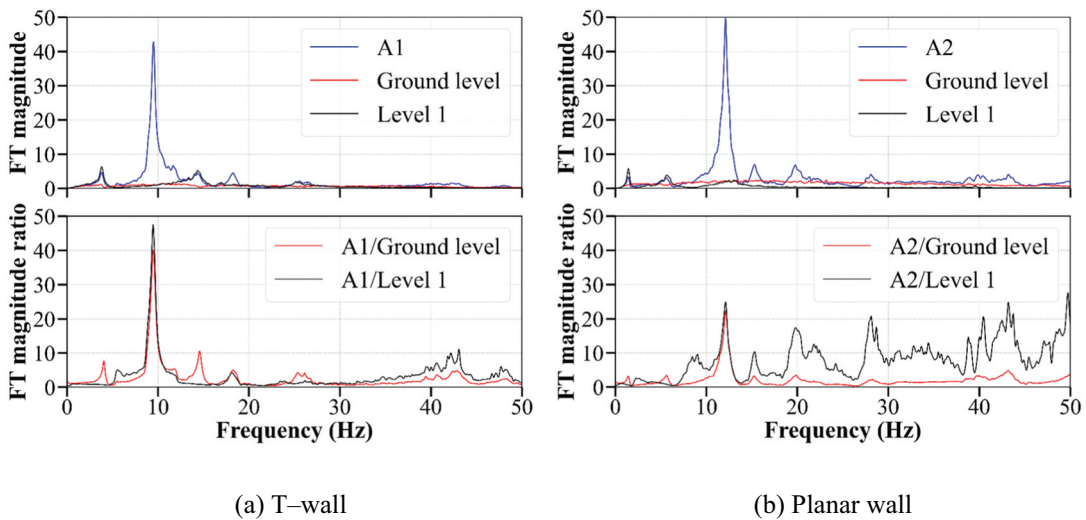


Figure 21. Representative FT and FRF of the partition wall out-of-plane acceleration (see Fig. 4 for accelerometer locations).

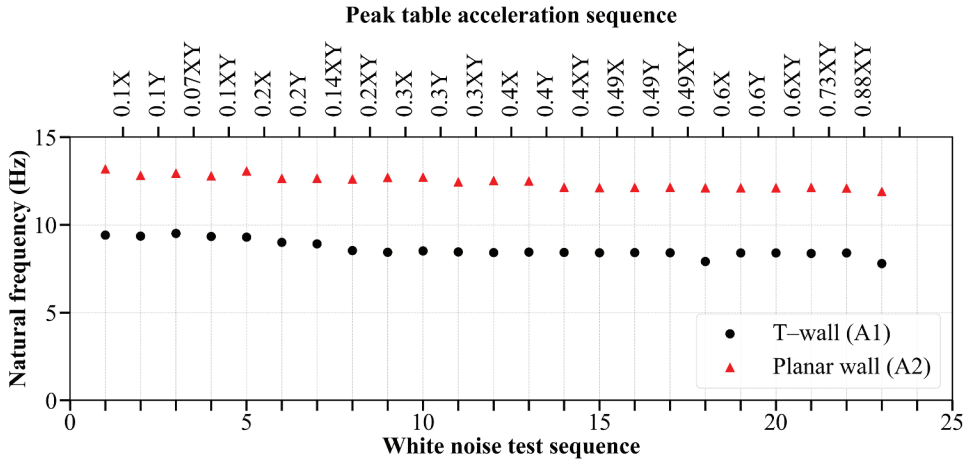


Figure 22. Variation of out-of-plane natural frequency of partition walls with white noise test sequence. The top horizontal axis indicates the shaking sequence before and after which the white noise tests were conducted.

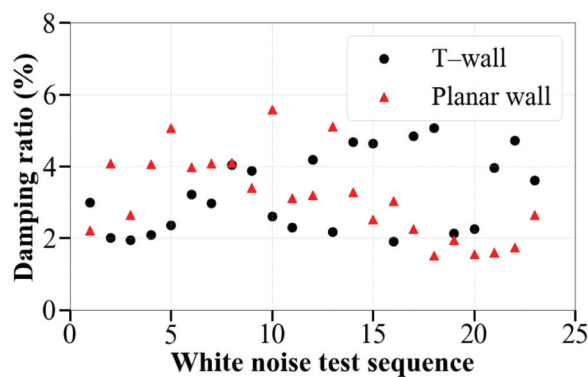
frequency was not observed. This suggests that the observed reduction in natural frequency throughout the testing program may be associated with an increase in the flexibility of the wall system, likely resulting from gradual loosening of the plasterboard-to-stud screw connections, widening of the dual slots, softening of the sealant under repeated shaking, among other unforeseen mechanisms. Environmental conditions, such as temperature and humidity, can alter the capacity of plasterboard-to-stud screw connections, as observed in timber-framed buildings with plasterboard acting as the primary lateral load-resisting system (Carradine 2018), and hence affect the dynamic properties of such wall systems. However, shake table tests in the present study were conducted in a laboratory environment over a relatively short period (around 7 hours), making significant environmental variations unlikely. Therefore, their influence on the observed frequency changes is expected to be negligible.

3.9.2. Damping Ratio

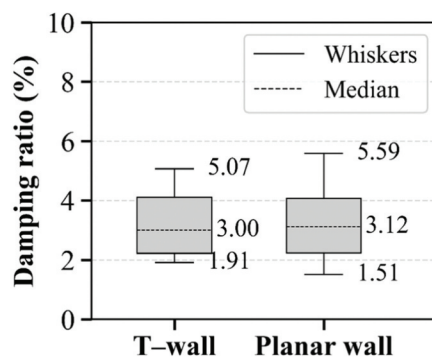
The damping ratios of the partition walls were evaluated using the half-power bandwidth method (refer, for example, to Clough and Penzien 1995), extending a standard method for structural systems to the partition walls. This method determines the damping ratio from the frequencies at which the response amplitude is equal to $1/\sqrt{2}$ times its peak amplitude. The results show that the damping ratios of both partition walls in the out-of-plane direction fall within a range of 1–6%. The variation in damping ratio across different white noise tests shows no consistent increasing or decreasing trend (Fig. 23a). The statistical summary (Fig. 23b) further highlights the overall similarity between the two wall types. The median damping ratios are nearly identical: 3% for the T-wall and 3.12% for the planar wall, and their ranges are also comparable.

4. Practical Implementation Considerations

The experimental results demonstrated the effectiveness of the rocking partition wall system in accommodating inter-story drift demands of up to 2.12% while limiting damage to localized deformation of the DST flanges and sealant debonding at wall junctions. However, several practical aspects should be considered when implementing the system in real buildings. Compared with existing plasterboard partition systems, the rocking wall system incorporates additional detailing components, such as DSTs, aluminum angles, and boxed studs reinforced with timber planks. Although the DSTs and boxed studs are not available as prefabricated products, they can be easily fabricated or assembled



(a) Variation of damping ratio with white noise test sequence



(b) Statistical summary as box plot

Figure 23. Damping ratio of the partition walls.

from commercially available steel sections and timber planks, reducing the need for proprietary products and facilitating procurement. Based on feedback from the builders, their fabrication and assembly do not require significant additional labor, but it may take some time for them to understand the details initially. The main limitation arises when suitable commercial steel sections for the DST are unavailable, in which case custom manufacturing may be required, potentially increasing construction costs and installation time. Additionally, whenever it is challenging to conceal the sealant joints and provide better aesthetics, such joints can be covered using paper tape (Bhatta, Dhakal, and Sullivan 2023b) or sacrificial L-trim joints with plaster finishing (Bhatta, Dhakal, and Sullivan 2023c) or painted over (Bhatta, Shrestha, et al. 2026).

Careful control of construction tolerances, particularly for the top vertical gap, the gap between aluminum angles and end studs, and the slot width in the DST, is essential to achieve the intended seismic performance. The top vertical gap must remain unobstructed to accommodate the intended rocking displacement. End studs should be installed in close contact with aluminum angles, without any gaps between them, to ensure rocking of wall initiates easily (Bhatta, Dhakal, and Sullivan 2023b). Furthermore, the slot widths within the DST must be sufficiently wide to allow unrestricted rocking of both the studs and plasterboard (Bhatta and Dhakal 2026). In addition, the leg-length of the aluminum angles below the top DST may also influence the rocking response of the wall since the load-carrying capacity of the angles depends on their leg-length. The thickness of aluminum angle should be selected considering their leg-length and the expected load demands. Moreover, maintenance and repair procedures for the rocking wall system are expected to be similar to those of existing partition wall systems, as it does not require specialized inspection or repair methods and is constructed using components commonly used in partition wall construction. Although a detailed life-cycle cost assessment was beyond the scope of this study, the observed low-damage behavior suggests that any additional construction cost associated with the proposed detailing may be justified by reduced post-earthquake repair requirements and improved functional recovery.

5. Conclusions

This study evaluated the seismic performance and dynamic characteristics of rocking plasterboard partition walls through shake table tests conducted on a full-scale, three-story steel-frame-composite floor building. Three configurations (planar, L-shaped, and T-shaped) of rocking partition walls with dual-slot tracks (DSTs) were installed at story 1, between the ground level and Level 1 of the tested

structure, and their seismic response was examined under unidirectional and bidirectional horizontal ground motions. Overall, the study demonstrates that the rocking partition system is a resilient, low-damage solution capable of accommodating high inter-story drift demands without severe damage. The key findings are:

- (1) Among the investigated wall configurations, the planar walls accommodated peak in-plane inter-story drift ratios of up to 1.50% without any visible damage throughout the testing sequence. In contrast, the T-wall and L-wall accommodated peak in-plane inter-story drift ratios of up to 2.12%. These walls exhibited Superficial (DS_1), characterized by localized deformation of dual-slot track flanges, at a peak inter-story drift ratio of 1.24%. Moderate (DS_2) but repairable damage, involving sealant debonding at wall junctions, was first observed at a peak inter-story drift ratio of 1.49%. The experimentally determined drift capacity for Superficial (DS_1) corresponds to the Operational Performance Level, whereas Moderate (DS_2) corresponds to a performance level between Operational and Position Retention, as defined in ASCE/SEI 41–23 (2023). No damage corresponding to the Life Safety Performance Level was observed throughout the testing.
- (2) The experimentally measured peak rocking displacements of the partition walls were generally lower than the theoretical rigid-body predictions. This discrepancy can be attributed to the construction gap between the aluminum angles and the end studs, and friction at the interface of plasterboard and stud with the dual-slot tracks. Deviations from the theoretical rocking response were minimal at wall ends without boundary elements or sealant, while the presence of sealant and boundary interactions provided additional resistance, indicating their role in suppressing the rocking response.
- (3) The tested partition walls experienced amplified out-of-plane accelerations relative to the floor accelerations, with median amplification factors ranging between 1.5 and 3.5. The taller T-wall exhibited higher amplification factors than the shorter planar wall, indicating that wall configuration and height may influence the magnitude of out-of-plane acceleration amplification.
- (4) The out-of-plane natural frequencies of rocking plasterboard partition walls were found to range from 9.51 Hz to 7.79 Hz for T-walls and from 13.18 Hz to 11.90 Hz for planar walls. Both wall configurations exhibited similar damping ratios regardless of their configuration within the 1–6% range with median values of approximately 3%.

It should be acknowledged that the findings presented in this study are based on the tested rocking partition wall system installed in a single full-scale building specimen at one story location. While the results provide valuable information regarding the seismic response of the investigated wall system, its behavior under different structural systems and their locations in the structure may vary. Buildings with different structural systems may exhibit different modal periods and damping ratios. These factors may influence component acceleration amplification and acceleration-driven damage, such as bending of the flanges in the out-of-plane direction of the DSTs or damage to the DST web at anchor points. Similarly, partition walls installed at higher stories may experience larger floor accelerations but generally smaller drift demands, potentially altering their response. Nevertheless, this study shows that inter-story drift is the governing demand parameter for the rocking partition walls with DSTs, as the rocking response and damage progression can be reasonably predicted from and correlated with the imposed drift demands. Further investigation into the seismic performance of the rocking partition wall system considering interactions with the structural, other non-structural systems, and building services could be beneficial.

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Author Contributions

CRedit: **R. K. Shrestha:** Conceptualization, Data curation, Formal analysis, Investigation, Methodology, Software, Validation, Visualization, Writing – original draft, Writing – review & editing; **J. Bhatta:** Conceptualization, Methodology, Supervision, Writing – review & editing; **R. P. Dhakal:** Funding acquisition, Project administration, Resources, Supervision, Writing – review & editing; **T. J. Sullivan:** Supervision, Writing – review & editing; **A. Tiwari:** Investigation, Writing – review & editing; **Y. Liu:** Investigation, Writing – review & editing; **Z. Yan:** Investigation, Project administration, Writing – review & editing; **G. A. MacRae:** Funding acquisition, Project administration, Supervision, Writing – review & editing; **Y. Zhang:** Investigation, Writing – review & editing; **P. Xiang:** Funding acquisition, Project administration, Writing – review & editing; **L. Jia:** Funding acquisition, Project administration, Writing – review & editing; **S. Ramhormozian:** Funding acquisition, Project administration, Writing – review & editing; **G. C. Clifton:** Funding acquisition, Project administration, Writing – review & editing; **P. Quenneville:** Funding acquisition, Project administration, Writing – review & editing; **G. Rodgers:** Funding acquisition, Project administration, Writing – review & editing; **X. Zhao:** Funding acquisition, Project administration.

Disclosure Statement

No potential conflict of interest was reported by the authors.

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Data Availability Statement

The data analyzed during the current study are partially available in figshare at <https://doi.org/10.6084/m9.figshare.31987023>. Due to the ongoing nature of the project, the complete datasets cannot be made publicly available at this stage.

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