Earthquake Reconnaissance in Bangkok, Thailand, following the 28 March 2025 Mandalay Earthquake

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We extend our deepest gratitude to our Thai colleagues for hosting us, as well as to the building users who generously shared their time and insights.

Executive Summary

This report documents the impacts of the 28 March 2025 Mandalay, Myanmar earthquake on buildings in Bangkok, Thailand, with a particular focus on non-structural damage, functional recovery, geotechnical aspects and structural performance of tall buildings. Although the epicentre was more than 1,000 km from Bangkok, the event highlighted the region's vulnerability to distant large earthquakes, due to the deep sedimentary basin beneath the city and the presence of many tall buildings.

A New Zealand delegation conducted field investigations across 10 high-rise buildings, including commercial, residential, hospital, and mixed-use structures. The team combined temporary seismic instrumentation, visual inspections, building functionality surveys, and discussions with Thai engineers, academics, and building managers to document structural performance, non-structural damage, and post-earthquake recovery.

Recordings of ambient vibrations at building foundation level were used to identify the fundamental period of vibration of soil over bedrock at the visited building sites, which ranged from 6.7 to 7.8 seconds, matching the range of long period amplification observed in the earthquake ground shaking. Furthermore, ambient vibration recordings at the top-most storey of buildings were used to estimate the fundamental period of vibration of the buildings.

Non-structural elements were the most widely damaged components. Damage patterns varied with building height, likely due to peak inter-storey drifts. Buildings with vertical irregularities, such as setbacks, exhibited concentrated deformation. While structural damage was limited in most cases, observed failures in shear walls emphasise the need for improved detailing and quality control.

Across the buildings, functional disruption was moderate: most services were restored within hours to days. The most persistent functional issue was lift outages, which affected 70% of inspected buildings and, in some cases, required weeks to months for full restoration. Water supply interruptions occurred but were typically resolved within a few days. Many repairs were delayed not by physical damage severity but by insurance processes, contractor availability, and administrative bottlenecks.

Overall, the findings demonstrate that Bangkok's high-rise buildings generally performed well structurally, but showed deficiencies in terms of non-structural damage and functionality perspective. In particular, the widespread lift outages highlight a critical vulnerability for tall buildings, directly affecting accessibility, and functionality.

These insights underscore the need for ongoing collaboration between Thai and New Zealand partners to strengthen resilience in basin-affected urban regions and to develop strategies that better integrate functional recovery in future distant earthquakes.

Abstract

The 28 March 2025 Mandalay, Myanmar earthquake (Mw 7.7) generated widespread shaking across Thailand, providing a rare opportunity to examine the performance of high-rise buildings in a deep sedimentary basin located more than 1,000 km from the epicentre. This report presents findings from a New Zealand reconnaissance mission undertaken in Bangkok, focusing on estimating the fundamental period of the deep sedimentary basin, structural and non-structural damage, building functionality, and site-specific seismic observations. Ten buildings were investigated through temporary seismic instrumentation, visual inspections, functionality surveys, and discussions with engineers, academics, and property managers.

Across the surveyed buildings, non-structural elements experienced the most widespread damage, particularly infill walls, ceilings, and façade components. Damage patterns varied with building height and were amplified in structures with vertical irregularities. Structural damage was generally limited, though isolated shear wall failures emphasised the need for improved detailing and construction quality. Functionality impacts were moderate overall, with most essential services recovering within hours to days, but lift outages were extensive, affecting 70% of the buildings visited and constituting the most prolonged barrier to full functionality.

The event highlights the significant influence of basin-related amplification on distant earthquakes, the critical role of non-structural performance in maintaining building functionality, and the importance of risk communication in shaping public reuse of buildings. The insights gained from this collaboration between Thai and New Zealand partners advance seismic resilience strategies for high-rise urban regions exposed to distant ground motions.

Keywords

Myanmar 2025 earthquake, reconnaissance, basin amplification, high-rise buildings, non-structural damage and functional recovery

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1 Introduction

1.1 Mandalay EQ Overview

A moment magnitude (M_W) 7.7 earthquake struck central Myanmar on March 28th, 2025. The epicentre was located approximately 326 km northwest of Thailand's Mae Hong Son Province. Despite the large epicentral distance, shaking was widely felt across Thailand, including Bangkok, its largest metropolitan area. This earthquake was the most significant seismic event to affect Thailand since 2004.

The earthquake occurred along the approximately 1200 km-long Sagaing Fault. The fault cuts across Myanmar from the North of the country to the Andaman Sea in the South (see Figure 1). The Sagaing Fault is a consequence of the complex zone of oblique convergence between the Indian plate to the west and the Sunda and Eurasia plates to the east. This convergence occurs at a rate of 35 – 40 mm/yr (Curray, 2005).

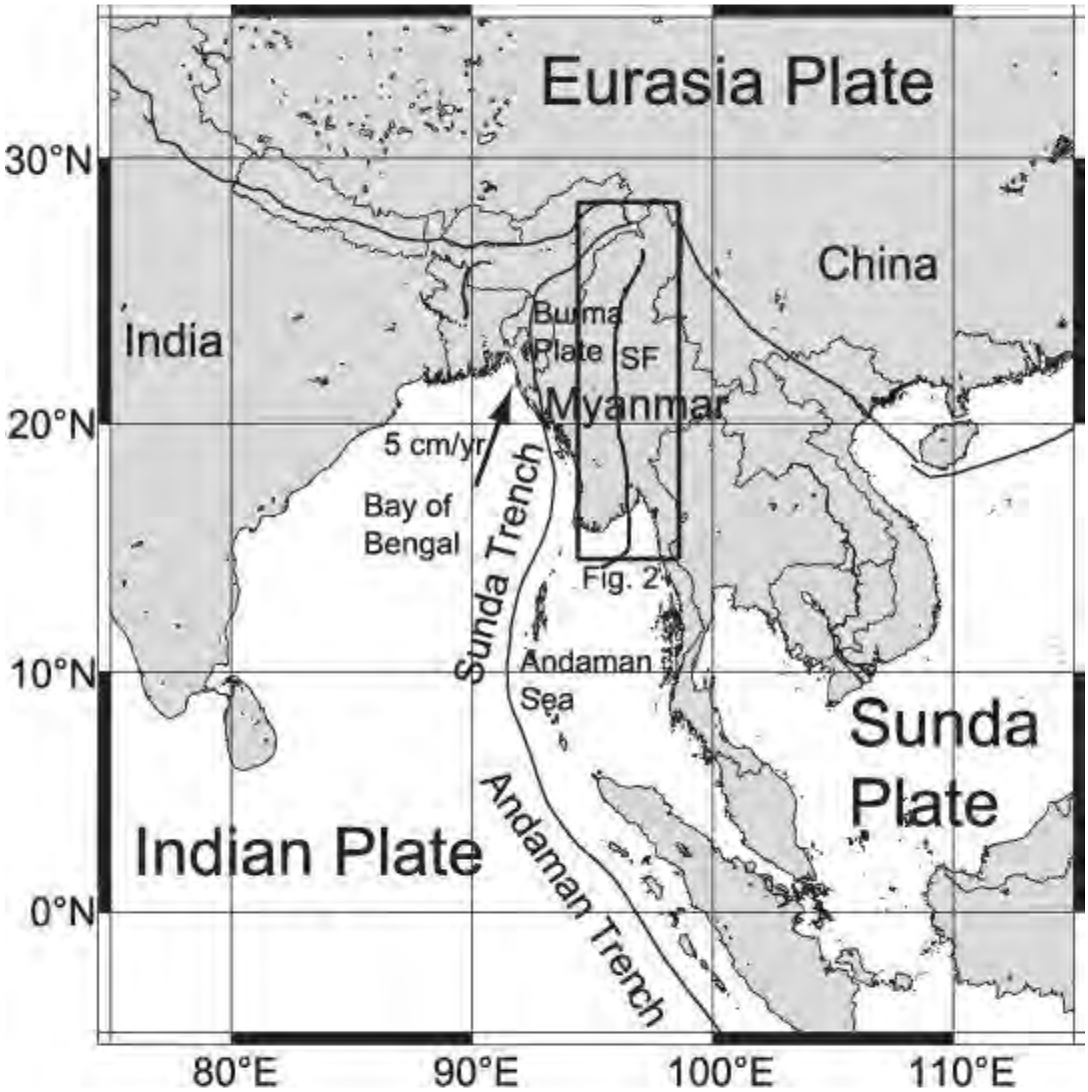


Figure 1: Regional map of the tectonic setting of Myanmar, Thailand, and neighbouring countries (Nobuo Hurukawa & Phyo Maung Maung, 2011)

The epicentre was located at 22.001°N, 95.925°E, with a focal depth of 10 km. The event ruptured the near-vertical strike-slip fault spanning approximately 500 km, with displacements up to 5 meters. The rupture propagated predominantly from North to South (Naohiro Inoue *et al.*, 2025) (see Figure 2), at a velocity of approximately 5 km/s, which is faster than the regional crustal shear wave velocity (Pourbeyranvand, 2025). The forward directivity associated with the Southward rupture increased ground shaking in distant areas, such as Bangkok, over 1000 km from the epicentre.

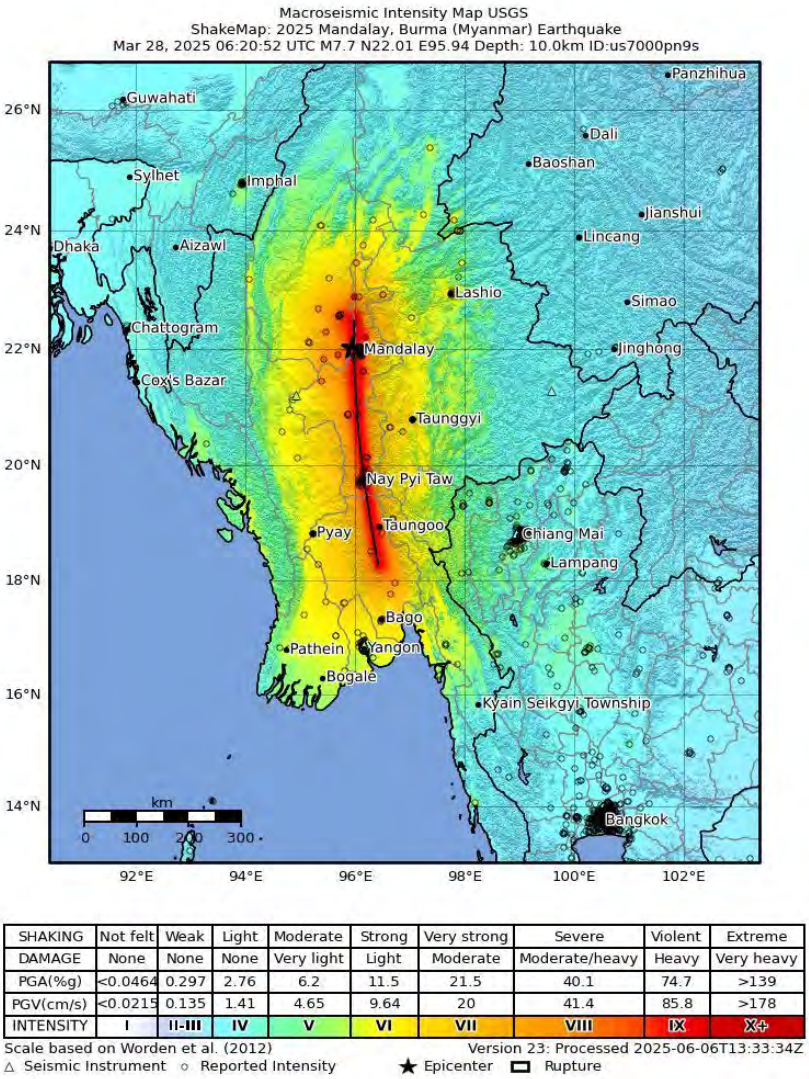


Figure 2: ShakeMap for the 2025 Mandalay Earthquake, produced by the United States Geological Survey, indicating the earthquake epicentre, the extent of fault rupture, and ground shaking intensity (USGS)

1.2 Seismicity of Thailand and Ground Shaking in Bangkok

Thailand is generally considered a low-to-moderate seismicity region, with relatively few active fault zones. However, large, distant earthquakes in high-seismicity regions (e.g., the Kanchanaburi Province in the Northwest of the country, the Sagaing fault in neighbouring Myanmar, and the Arakan Subduction Zone in the Bay of Bengal) pose a significant seismic hazard to the country. Bangkok, the largest metropolitan area and capital city of Thailand, sits in a deep, soft alluvial basin that amplifies long-period earthquake ground shaking (Ornthammarath *et al.*, 2025).

The 28 March 2025 Mandalay Earthquake caused ground shaking in Thailand, with initial reports of observed damage in Chiang Mai, a city in Northwest Thailand (closer to the Sagaing Fault), and in Bangkok. Videos posted to social media showed several towers swaying in the city, seiches in rooftop swimming pools, and the notable collapse of the under-construction State Audit Office building (The Nation, 2025).

Acceleration time histories for the two horizontal components at four strong-motion stations located in Bangkok are shown in Figure 3. The peak ground acceleration was 0.019 g, 0.022 g, 0.020 g, and 0.023 g at the BKSI, KMUA, PTNA, and PWSA stations, respectively. These modest short-period accelerations and relatively long-duration records highlight the large epicentral distances of the strong motion stations from the source. The locations of strong motion stations are shown in Figure 6 in the site visit section below.

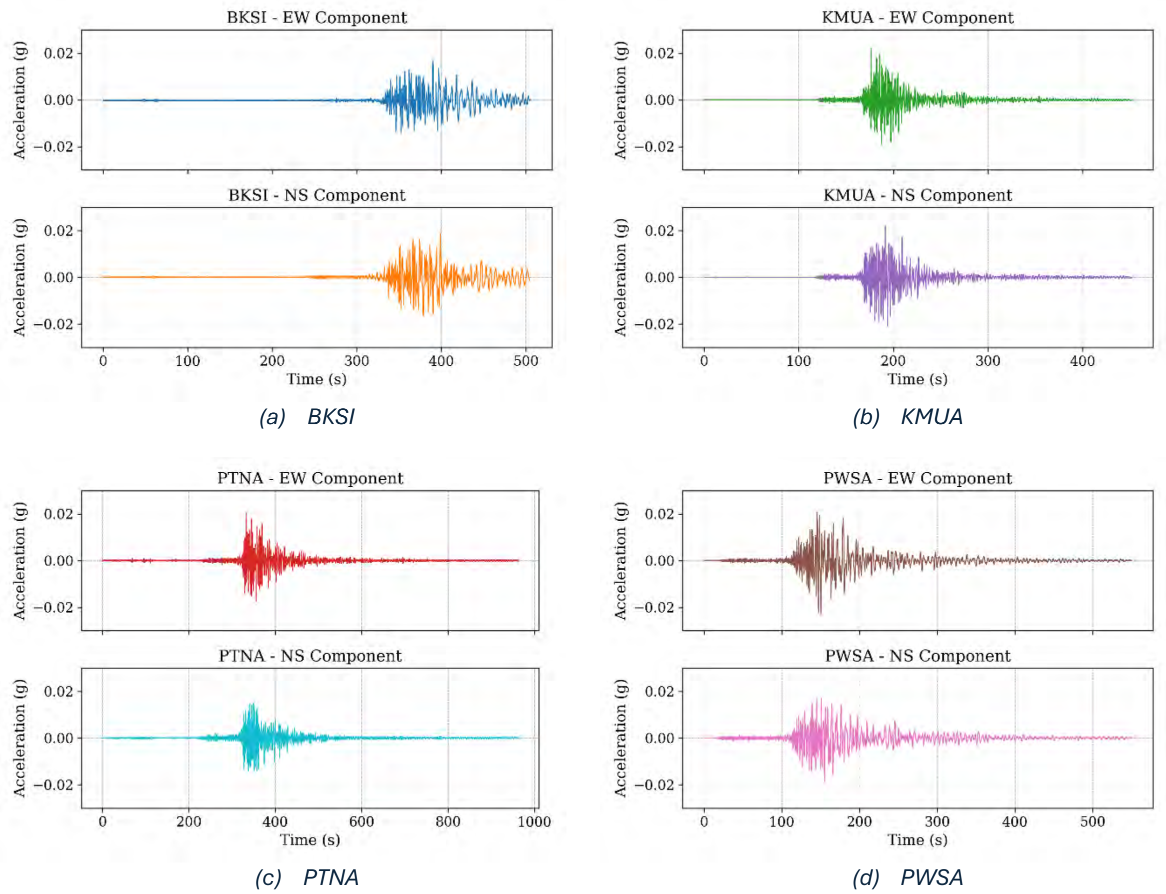


Figure 3: Acceleration time histories of the 25 March 2025 Mandalay Earthquake on the horizontal components of four strong motion stations located near Bangkok, Thailand

The 5% and 2.5% damped pseudo-spectral acceleration response spectra for horizontal components of ground shaking at the four strong motion stations are shown in Figure 4. Assuming 5% damping is common for many structures, but 2.5% may be more representative of the damping for the tall, slender towers in Bangkok. BKSI, PTNA, and PWSA all have long-period peaks in spectral acceleration centred at 6 seconds. Furthermore, BKSI and PWSA have an additional peak centred at 8 seconds on the East-West component. These long-period peaks may be attributed to the response of the deep, soft soils underlying Bangkok, and the period ranges may correspond to the fundamental vibration period of many of the tall, slender towers common in recent construction across the city.

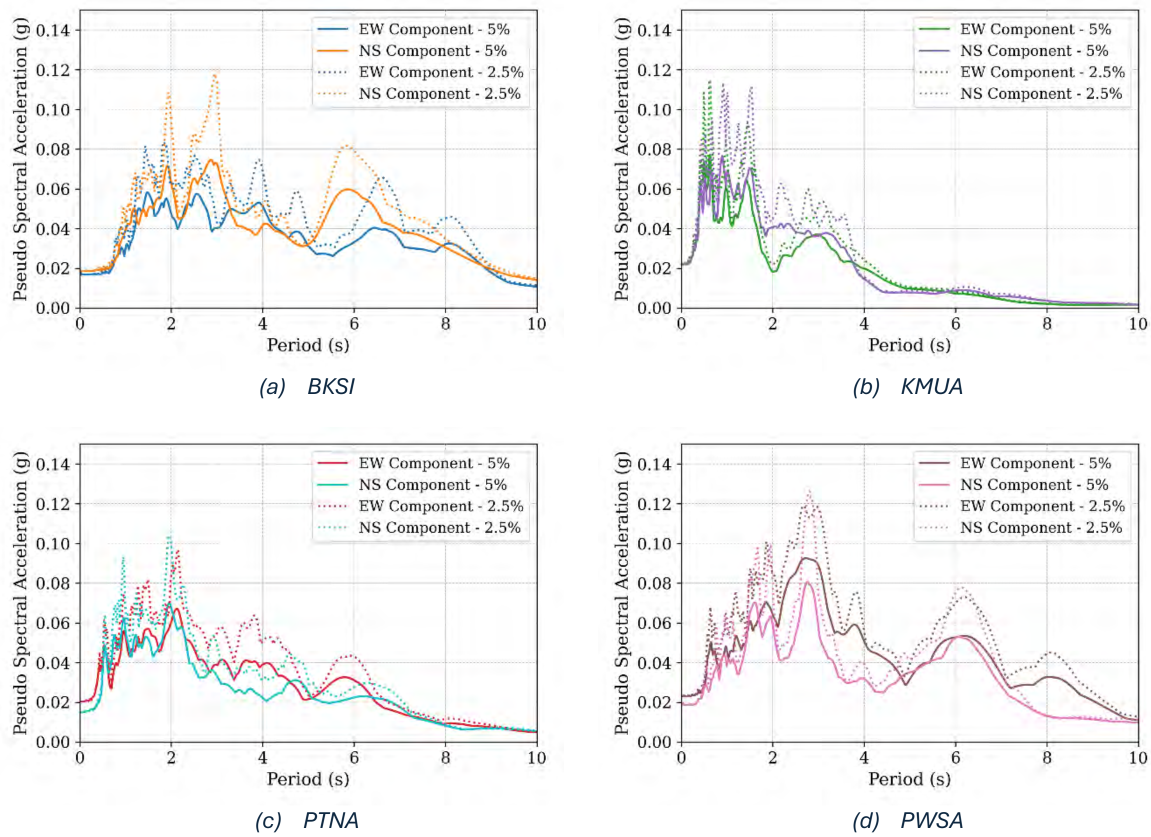


Figure 4: Pseudo-spectral acceleration response spectra at 5% and 2.5% damping for 25 March 2025 Mandalay Earthquake records from four strong motion stations located near Bangkok, Thailand

The 5% and 2.5% damped spectral displacement response spectra for horizontal components of ground shaking at the four strong motion stations are shown in Figure 5. Again, the long-period peaks at 6 and 8 seconds are present in the displacement response spectra, likely due to the response of the deep, soft sediments beneath the city. Such period ranges may correspond to the fundamental vibration period of many of the tall, slender towers common in recent construction across the city.

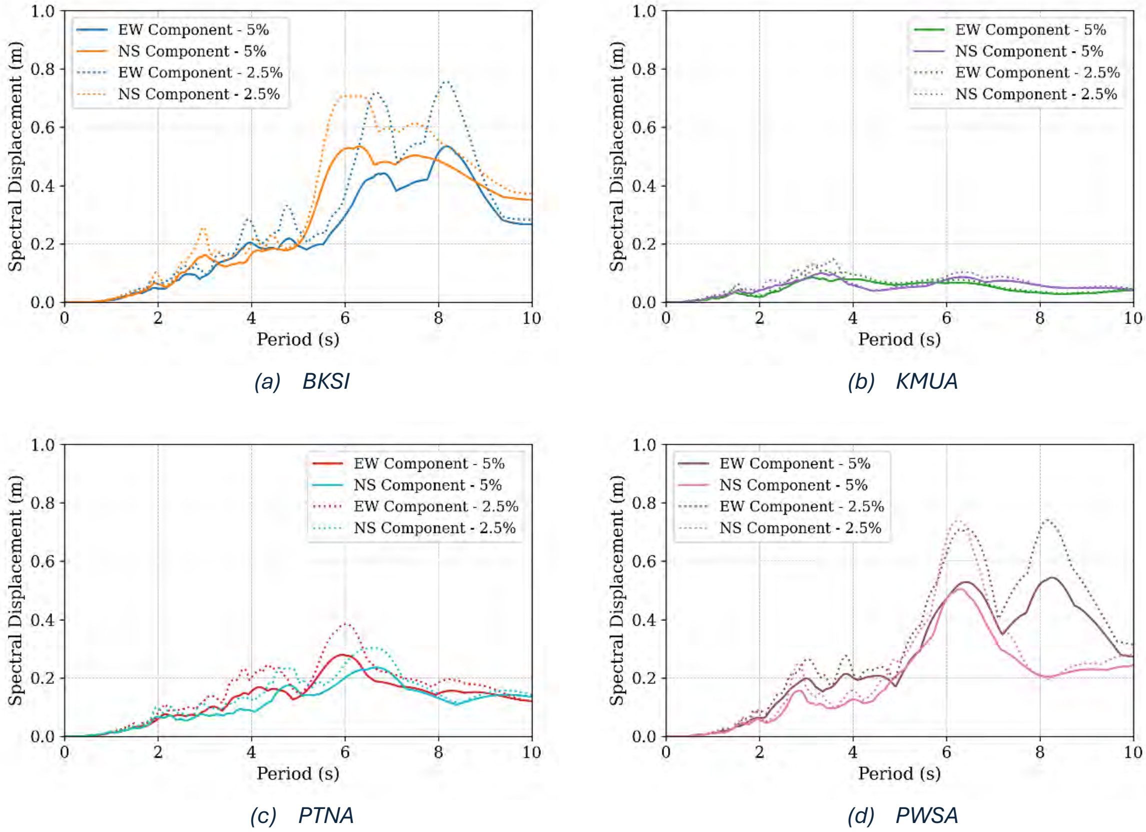


Figure 5: Spectral displacement response spectra at 5% and 2.5% damping for 25 March 2025 Mandalay Earthquake records from four strong motion stations located near Bangkok, Thailand

1.3 Building code requirements and practice in Bangkok

Thailand's seismic design requirements have evolved substantially over the past three decades, with several updates specifically addressing basin and deep-soil site effects in Bangkok. The first national seismic regulation was introduced in 1997, followed by the first formal design standard in 2009. More recently, the 2018 revision of the Thai seismic design standard incorporated major updates, including revised probabilistic seismic hazard assessments and new design spectral accelerations for Bangkok that explicitly account for deep-basin amplification effects (Poovarodom *et al.*, 2018). Each of the building codes is based on ASCE 7, and detailing requirements for concrete elements follow the requirements for "Ordinary Moment Frames" and "Ordinary Walls" as per ACI 318. This detailing is broadly aligned with the nominally ductile detailing provisions in NZS 3101:2006. Design demands are typically based off of 5% damping applied mass proportionally regardless of the height of the building and an inelastic design spectra with an R=5 (which effectively reduces demands to 20% of the elastic spectra) for ordinary concrete wall buildings.

1.4 Delegation and Objectives

Following the earthquake, there was general interest in the New Zealand Earthquake Engineering community to respond to and learn from the Mandalay Earthquake. Due to political instability and significant differences in building design and construction practices, a reconnaissance trip to Myanmar was deemed infeasible. However, the observed shaking of tall buildings in Bangkok, despite the large epicentral distance, presented potential learning opportunities for the New Zealand Earthquake Engineers, particularly regarding the performance of tall buildings in the

Auckland CBD despite long distances from seismic sources. Building functionality and non-structural damage were other aspects of interest. Thus, a delegation of academic and practising engineers was formed in April 2025.

Following discussions regarding the scope of the reconnaissance work, the delegation team was finalised, and funding for the trip was secured. The delegation was funded by QuakeCoRE, Toka Tu Ake Natural Hazard Commission, the New Zealand Society for Earthquake Engineering, and the American Concrete Institute (ACI). The delegation visited Bangkok, Thailand, from 11 to 18 June 2025.

The delegation was co-led by Dr Andrew Stolte, Senior Lecturer in Geotechnical Engineering at the Waipapa Taumata Rau | University of Auckland, and Dr Catalina Miranda, Lecturer in Disaster Risk Management at the Joint Centre for Disaster Research at Massey University. Other members of the team included:

- Dr Lucas Hogan, Senior Lecturer in Structural Engineering at the University of Auckland,
- Dr Jitendra Bhatta, Postdoctoral Researcher at the University of Canterbury, specialising in non-structural damage.
- Dr Jitendra Bothara, Structural Engineer, Director at ResiPro International Engineering Ltd
- Dr Andrew Baird, Structural Engineer at Beca New Zealand
- Dr Chungwook Sim from Konkuk University in South Korea, who joined the New Zealand delegation as part of his role at the ACI 133 disaster reconnaissance committee.

The team aimed to support knowledge-sharing between Thailand and New Zealand, contributing to a broader understanding of disaster resilience. The primary goals of the visit were to:

1. Assess the fundamental period of vibration (e.g., site period) of local soils under affected buildings using broadband seismometers to improve understanding of the deep sedimentary basin and the role of local site conditions on shaking amplification of buildings.
2. Document examples of both structural and non-structural damage in the built environment. This included observing damage to non-structural elements, such as ceilings, facades, and building services, to understand how different construction types and details responded to shaking.
3. Explore the functional recovery of buildings following the event via a systematic survey. The survey sought insights from building users, such as facility, building, and property managers, on the functionality and recovery timelines of various building components (e.g., lifts, lighting, water supply, HVAC systems, windows, façades, etc.). The aim was to understand how damage to these elements affects the building's ability to return to full functionality.

1.5 Functionally data collection

Despite significant research on functional performance and resilient design, a noticeable gap persists between theoretical design expectations and actual building performance following major earthquakes. This discrepancy is not limited to high-seismicity regions; even buildings located far from the earthquake source can experience disruptions to functionality. Field observations frequently show that construction quality and the degree of code enforcement strongly influence the extent and nature of damage, contributing to variability in post-earthquake building performance.

Past earthquake reconnaissance missions have largely prioritised documenting physical damage, with less focus on systematically assessing building functionality or estimating timeframes for return to function. As a result, the functional consequences of structural and non-structural damage remain under-documented, despite their central role in community recovery and continuity of services.

Functional-recovery assessments require more comprehensive data than traditional damage-focused reconnaissance. Beyond observing physical damage, teams must capture how functions are impacted by both structural and non-structural damage, as well as the time and dependencies required to restore those functions. There is a need for engagement with occupants and hence consider ethics procedures (Hariri-Ardebili & Speicher, 2024).

The factors influencing functionality are numerous and range from structural to non-structural damage, and from cosmetic to service damage. Observed factors that influence post-earthquake functionality may include: extent of building/component damage; building material type; pre-event functional condition; interdependence with lifeline infrastructure; options for relocation and system redundancy; availability and scheduling of resources; organisational networks and capacity; social/community preparedness; governmental roles; and multi-organisational collaboration. These factors collectively shape the pace and feasibility of restoring services to support reoccupancy, functionality, and full repair

A systematic survey instrument was used to document building functionality, damage to structural and non-structural elements, and associated recovery times. The survey was provided by the National Institute of Standards and Technology (NIST) Earthquake Engineering Group and modified to fit the Thai context. The instrument captured: levels of observed damage; time to recovery to the current state; reasons for loss of functionality; element-specific questions (e.g., electrical, water supply, sewer, stairs, lifts, and architectural components); element-specific recovery times; reasons for failure; and repair/impeding factors.

2 Overview of Building Visits

The reconnaissance study focused on high-rise buildings in central Bangkok and its surrounding areas. In collaboration with Thai partners (including academics, structural engineers, and graduate students), the team investigated 10 buildings, including four known to have structural damage (refer to Table 1). These visits were facilitated by Prof. Pimanmas, Prof. Leelataviwat, and Weravat Tudthong (Managing Director at Beca's Thailand Office). The locations of the visited buildings and three strong-motion stations in Bangkok are shown in Figure 6, noting that station PTNA is approximately 40 km north and is not shown.

The investigated buildings include commercial, residential, hospital, and mixed-use structures. All inspected structures were constructed using reinforced concrete with post-tensioned flat slab floors with concrete walls as the main lateral resistance system, except for one building, which utilised a concrete-filled steel tube (CFST) and braced frame system. The buildings investigated were consistent with typical construction practices in Thailand.

The reconnaissance team conducted systematic visual inspections to document earthquake-induced damage to both structural and non-structural elements. All observed damage was documented with photographs and locations. Each building was temporarily instrumented with broadband seismometers at the foundation and topmost accessible levels to estimate the fundamental periods of (1) the soil above a stiffer geologic stratum and (2) the building. The visits included well-performing buildings to identify effective seismic-resistant design features and construction practices that contributed to their resilience

Several features of the structural system of the investigated buildings are summarised in Table 1. RC core wall systems dominate (80% of cases), particularly in commercial (C1, C2, C3) and older residential (e.g. R2 and R3) buildings. The hospital visited had several buildings on-site; those visited included an 11-storey and a 25-storey building, with mixed structural systems. The residential buildings included the oldest (1993) and newest (2025) buildings visited, and range in height from 30 to 45 stories. The commercial and mixed-use buildings were taller, with one building visited (R4) being 79 storeys.

The increased use of high-strength concrete since the early 2000s may have enabled slimmer structural sections, resulting in more flexible buildings than pre-2000 structures. Additionally, the widespread adoption of structural analysis software from the mid-2000s may have facilitated the design of more refined structural systems. As a result of these developments, on-site observations indicated a trend towards more slender structural members. This shift also suggests possible variations in code requirements and construction standards over time, though core walls remain prevalent for lateral stability in high-rise buildings.

Table 1: List of visited buildings

Building ID	Use/Occupancy type	Primary Structural System	Year of construction	Number of storeys	Height (m)	Tag (Structural Damage)
H1	Hospital	Reinforced Concrete (RC) Frame with post-tensioned flat slab and RC walls on four corners	2019	25	100	Yellow
H2	Hospital	RC, core walls on the four corners	Under construction	11	50	Green
R1	Residential	RC walls with post-tensioned (PT) flat slab	2025	45	135	Red
R2	Residential	RC, core walls (with vertical setbacks)	1993	45	135	Green
R3	Residential	RC, flat slab with band beams, core walls	2006	30	90	Green
R4	Mixed-use. Residential, hotel and commercial	RC, shear core with outriggers and PT slab	2016	79	314	Green
C1	Commercial	RC, core walls	2024	47	240	Green
C2	Commercial	RC, core walls	1995	32	67	Green
C3	Commercial	RC, core walls	2023	61	212	Green
C4	Commercial	RC+Steel (Concrete Filled Tubes)	2020	30	161	Green

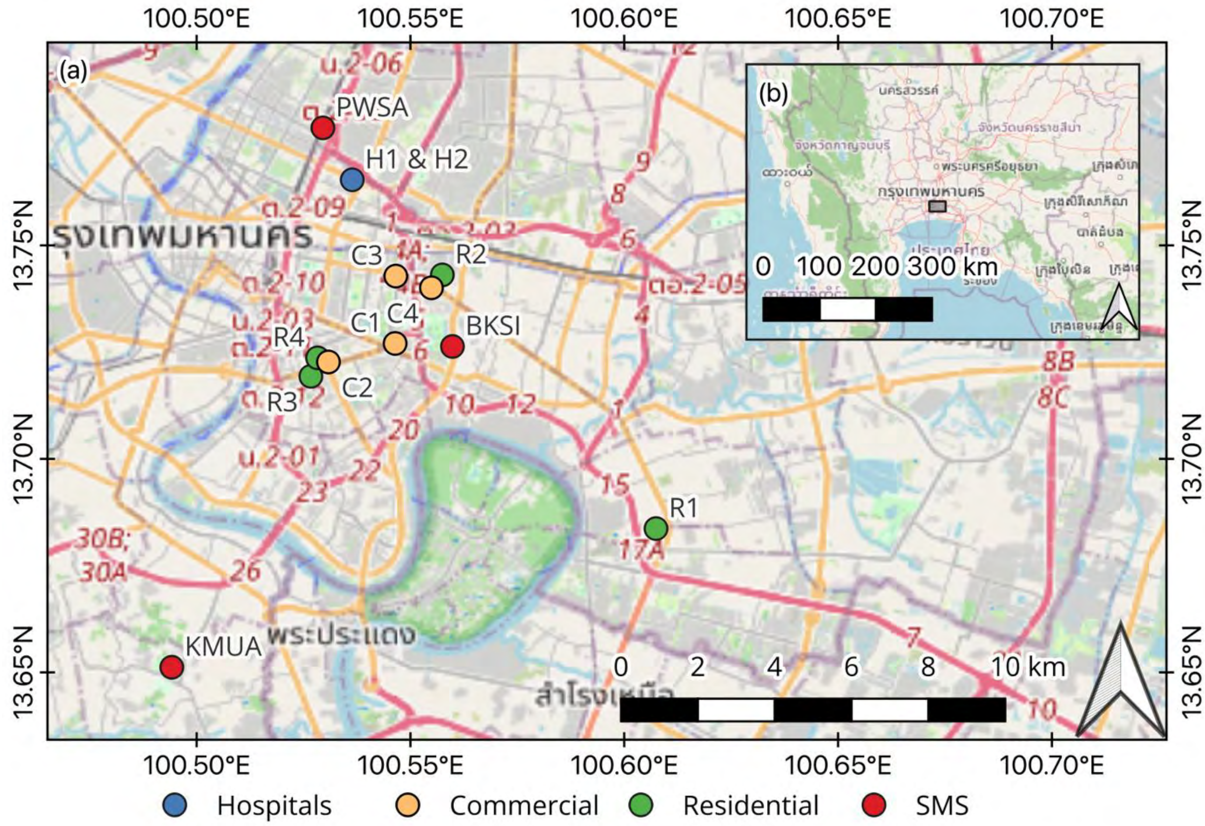


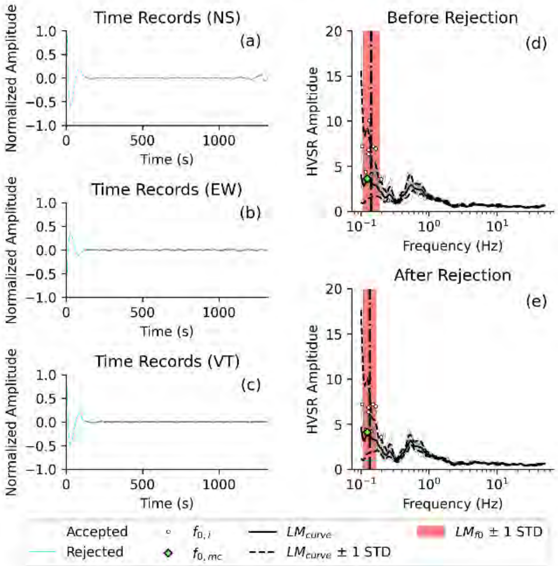
Figure 6: (a) Inset map of building and strong motion station locations in central Bangkok, Thailand. (b) Regional map with the location of the inset study area map. Base map is from OpenStreetMap

3 Results

3.1 Estimates of Site and Building Fundamental Period of Vibrations

The microtremor (ambient vibration) horizontal-to-vertical spectra ratio, abbreviated mHVSR (Nakamura, 1989), has been used worldwide to estimate the fundamental period of vibration of soft soils over stiff(er) soil or rock layers. The method involves temporarily installing three-component seismic instrumentation (e.g., broadband seismometers, as shown in Figure 7 (a)) to record ambient ground surface vibrations. After processing (as detailed in Appendix C and Molnar et al., 2022), the peaks in the resulting horizontal-to-vertical spectral ratio spectrum are attributed to the dynamic response of the soil; an example is shown in Figure 7 (b). Note mHVSR spectra are typically plotted in terms of frequency as the method is used by seismologists. The inverse of frequency (1/f) is period, which is commonly used by engineers.

(a) Photo of the typical placement of a broadband seismometer at the foundation level of a building on a concrete pad. The silver cylinder is a three-component seismometer, and the data logger is inside the black case.



(b) Example mHVSR spectra from an ambient vibration recording at the foundation level of a building. The three-component ambient vibration records are shown on the left, and the initial and refined mHVSR spectra are shown on the right. The red-highlighted frequency range shows the low-frequency (long-period) response associated with the deep, soft soils beneath Bangkok.

Figure 7: Typical Instrumentation and results of mHVSR testing

The mHVSR method has previously been used by Poovarodom and Plalinyot (2013) to characterise the soft soils beneath Bangkok, focusing on the soft soils over stiffer soil layers. However, limitations of the instrumentation in that study meant that the long-period response of the sediments over bedrock could not be resolved.

As a part of the reconnaissance efforts, the NZ delegation brought four broadband seismometer stations with a flat response to velocity from 120s (0.0083 Hz) to 200 Hz. This enabled measurement of the long-period response of the sediment basin; up to ~8-second peaks were identified in the data, by temporarily instrumenting the ground floor or foundation level of visited buildings. Best practice encourages burying sensors at the ground surface in soil to improve sensor-ground coupling. This was generally not practicable at the building visit sites, due to the built environment and the security of the sensors during the temporary placement.

While one sensor was placed at the foundation level of the visited buildings, another was temporarily placed at the top-most accessible level (typically the top floor or roof) of each building. The mHVSR method has not been widely used to measure/estimate a building's dynamic response (i.e., its fundamental period of vibration). However, initial trial studies by UoA researchers at trial sites in Wellington indicate mHVSR might be used for these estimates.

Thus, every building was temporarily instrumented at (1) the foundation level and (2) at the top-most level to estimate the fundamental site period of soil over presumably bedrock and the fundamental period of vibration of the building. The frequency, period, and log normal standard deviation associated with the three lowest frequency (longest period) peaks in the mHVSR spectra are summarised in Table 2, and the corresponding mHVSR spectra are detailed in Appendix C. When a mHVSR spectra peak period is attributed to either the fundamental period of the deep sedimentary basin or the structure, the values are bolded, and a note is made in the right-most column.

Table 2: Summary of mHVSR Testing Results

Building	Location (Floor)	Station	HV Peak 1			HV Peak 2			HV Peak 3			Notes
			$T_{HV,0}$ (s)	$f_{HV,0}$ (Hz)	σ_{ln}	$T_{HV,1}$ (s)	$f_{HV,1}$ (Hz)	σ_{ln}	$T_{HV,2}$ (s)	$f_{HV,2}$ (Hz)	σ_{ln}	
H1	B4	STN01	**7.24**	**0.14**	**0.17**	1.12	0.89	0.10	---	---	---	**Basin T_0**
	F25	STN02	7.59	0.13	0.12	**2.80**	**0.36**	**0.06**	0.81	1.23	0.09	Possible Building T_0
R1	G	STN01	4.18	0.24	0.02	1.37	0.73	0.12	---	---	---	*Missing Basin Response*
	F08	STN02	**3.70**	**0.27**	**0.02**	1.03	0.97	0.02	0.54	1.85	0.02	Possible Building T_0
R2	B1	STN01	**7.56**	**0.13**	**0.16**	1.21	0.83	0.07	---	---	---	**Basin Response**
	F44	STN02	6.86	0.15	0.19	**2.27**	**0.44**	**0.09**	0.73	1.37	0.00	Possible Building T_0
	F25	STN02	---	---	---	---	---	---	---	---	---	No Clear Peak
R3	G	STN01	**7.48**	**0.13**	**0.14**	0.92	1.09	0.31	---	---	---	**Basin Response**
	F29	STN02	8.31	0.12	0.09	**1.76**	**0.57**	**0.02**	0.56	1.79	0.03	Possible Building T_0
R4	B1	STN01	**7.80**	**0.13**	**0.06**	4.58	0.22	0.14	2.03	0.49	0.12	**Possible Basin Response**
	F76	STN02	**5.66**	**0.18**	**0.14**	2.15	0.47	0.01	0.82	1.22	0.02	Possible Building T_0
C1	B4	STN04	**7.52**	**0.13**	**0.16**	2.07	0.48	0.11	---	---	---	**Basin Response**
	50	STN05	6.31	0.16	0.14	**3.95**	**0.25**	**0.02**	2.71	0.37	0.02	Possible Building T_0
C2	B2	STN01	**7.46**	**0.13**	**0.20**	3.86	0.26	0.09	1.64	0.61	0.19	**Basin Response**
	F35	STN02	**3.59**	**0.28**	**0.21**	1.99	0.50	0.14	---	---	---	Possible Building T_0
C3	B4	STN01	4.87	0.21	0.10	2.04	0.49	0.04	1.26	0.79	0.02	*Missing Basin Response*
	F59	STN02	6.93	0.14	0.14	**4.36**	**0.23**	**0.12**	2.63	0.38	0.01	Possible Building T_0
C4	B5	STN01	**6.76**	**0.15**	**0.12**	0.59	1.69	0.09	---	---	---	**Basin Response**
	F29	STN02	6.83	0.15	0.09	**2.99**	**0.33**	**0.08**	2.08	0.48	0.03	Possible Building T_0

In ambient vibration recordings at the ground or foundation level of the visited buildings, a long-period (low-frequency) peak is consistently identified across the dataset, with a period ranging from 6.76 to 7.80 seconds. This matches the long-period peaks in the observed ground shaking records at the BKSI and PWSA strong motion stations, as shown in pseudo-spectral acceleration (Figure 3) and spectral displacement (Figure 4) response spectra, reinforcing the understanding that the deep sedimentary basin contributed to the amplification of the long-period ground motions.

In the ambient vibration recordings at the top-most accessible floor, typically a peak associated with basin response and a peak associated with the building's response were identified. The fundamental period of a building is a function of its stiffness and mass and is often empirically correlated with building height. The estimated fundamental periods of the building are reasonable, based on the delegation's understanding of the structural lateral resistance systems and building height. However, additional modal analysis of the structures is needed to confirm these observations.

3.2 Observed structural damage

As noted in Table 1, only two of the buildings that were investigated showed appreciable structural damage. Most of the green-tagged buildings had either very limited or no structural cracking observable. This section describes only the observations from the three buildings with appreciable structural damage, C3 (green tagged), H1 (yellow tagged), and R1 (red tagged).

3.2.1 Damage to Building C3

Building C3 is a 61-storey RC core wall building constructed in 2023. It has (2) C-shaped walls and (1) I-shaped wall, (2) L-shaped walls, all with webs in the x direction. The gravity system consists of a 200 mm thick RC slab on beams, with RC columns on the perimeter. The building experienced minor structural damage, which consisted of 0.3 mm cracking in the soffit of beams at the exterior of the 14th floor and 1.0 mm wide cracking in the topping slab. The location of the damage is indicated in Figure 8. The damage occurred on gridline 4.1 in an area where the collector beam is offset from the gravity column. It is suspected that the eccentricity of the collector beam, and the torsional eccentricity caused by the wall distribution being biased towards the western side of the building, contributed to the damage occurring in this location. This floor damage was visible due to the floor being un-tenanted at the time of reconnaissance. It is not clear if such damage occurred elsewhere in the building, but the damage was not visible in the other un-tenanted floors visited.

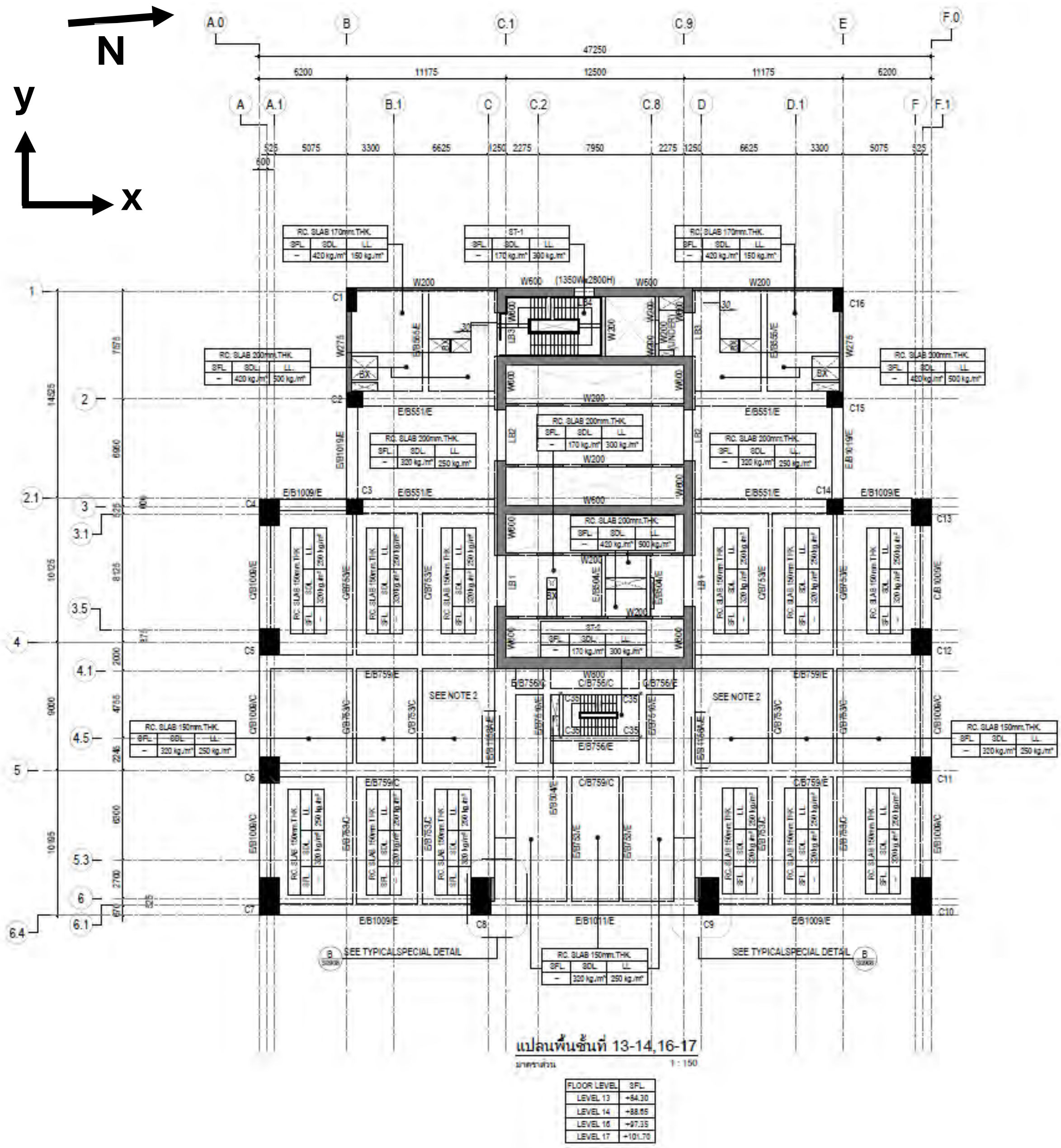
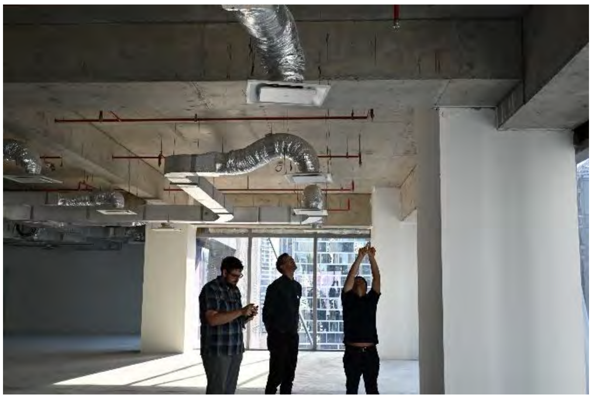
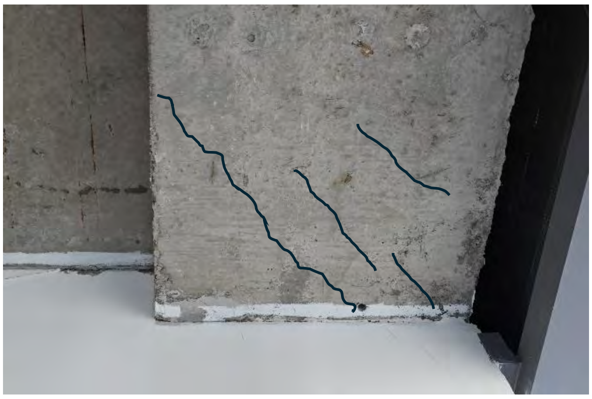


Figure 8: Building C3 plan (Typical story 14th floor)

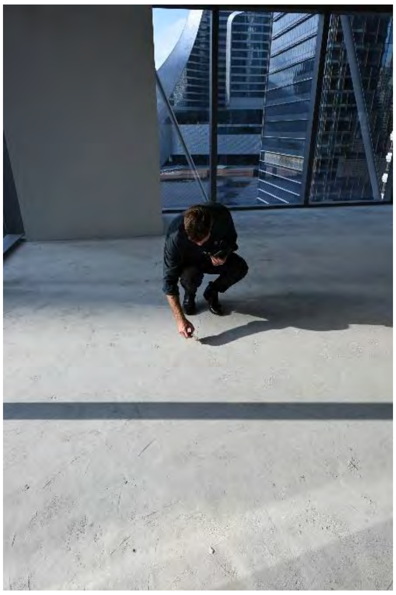


(a) Location of beam and ceiling slab damage

(b) Joint at beam and slab damage



(c) Cracking on soffit of beam (crack width ~0.2 mm)



(d) Crack in floor topping

(e) Measurement of floor topping crack (crack width = 1.0 mm)

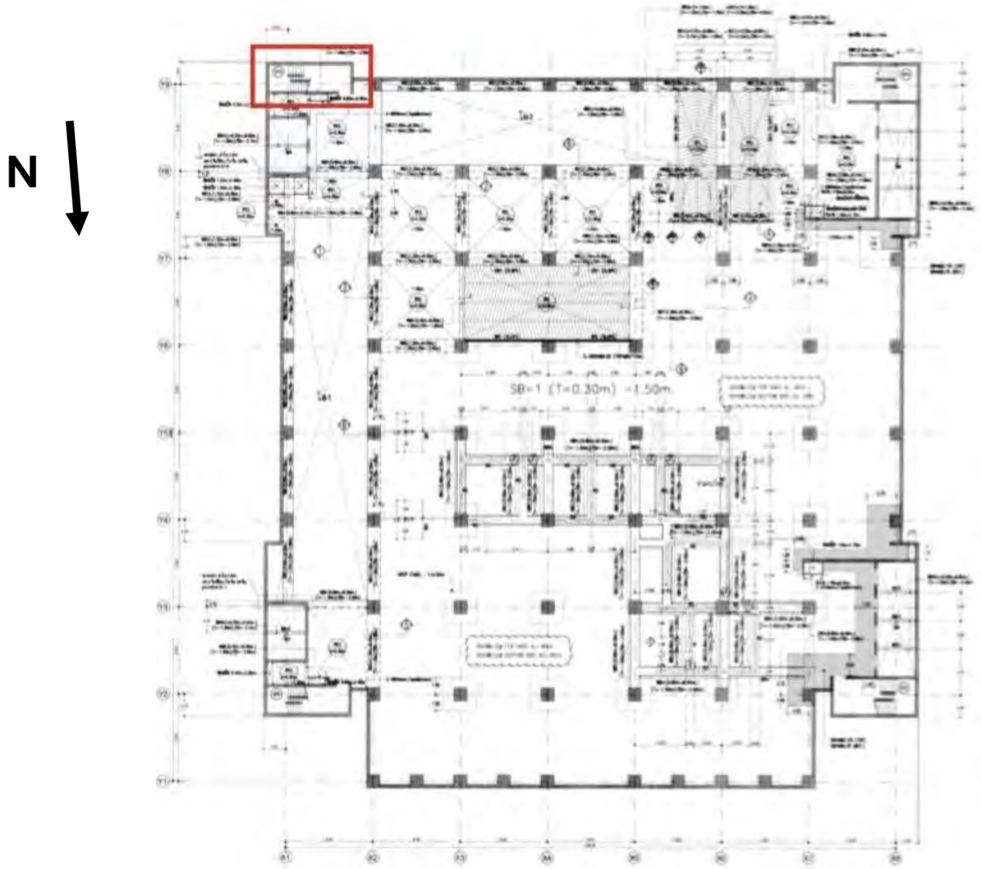
Figure 9: Building C3 Structural Damage Photos

3.2.2 Damage to Building H1

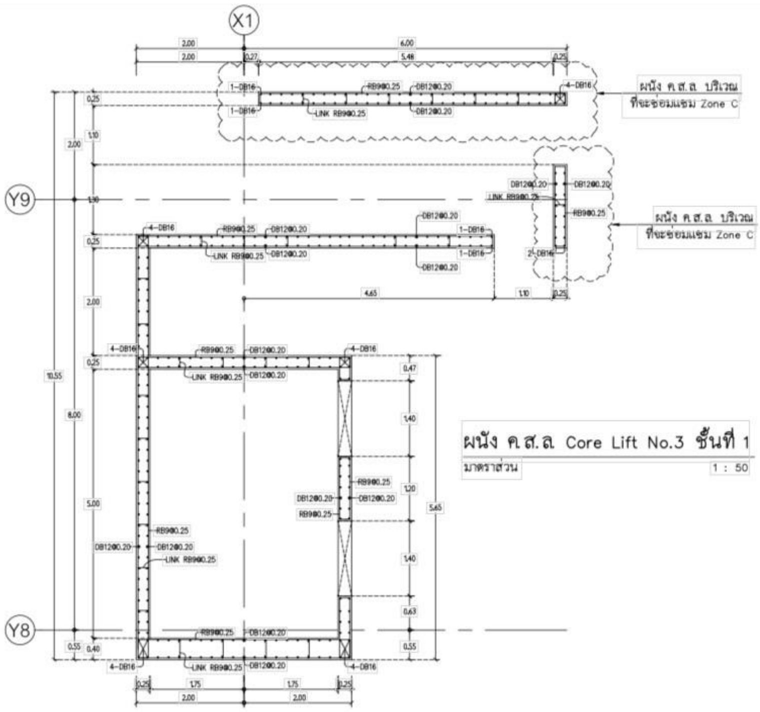
Building H1 is a 25-storey hospital building constructed in 2019. The main structural system is reinforced concrete columns (1.2 m x 1.2 m) with post-tensioned flat slab and RC walls (200 mm thick) surrounding the stairwells at each corner of the building (Figure 10). The building suffered a through thickness failure of three walls surrounding the stairwell in the southeast corner of the building at what appears to be the top of a lap splice in the ground floor (Figure 11). No other structural walls or columns exhibited damage.

The likely causes of the wall failures appear to be a function of two aspects. First, boundary element hoops that are specified on the structural drawings (Figure 10b) do not appear in the wall boundary elements. Second, it was reported to the reconnaissance team by Thai colleagues that concrete testing of the wall indicated a strength of approximately 15 MPa while 30 MPa was specified. This effective doubling of the axial load ratio and lack of boundary element hoops due to construction error would help explain the localised nature of this failure.

During the reconnaissance visit, the wall was being repaired with a concrete jacket (Figure 12). Loose concrete had been removed, and 90-degree hooks had been epoxied into the original wall. Jacket reinforcing was welded onto these hooks.

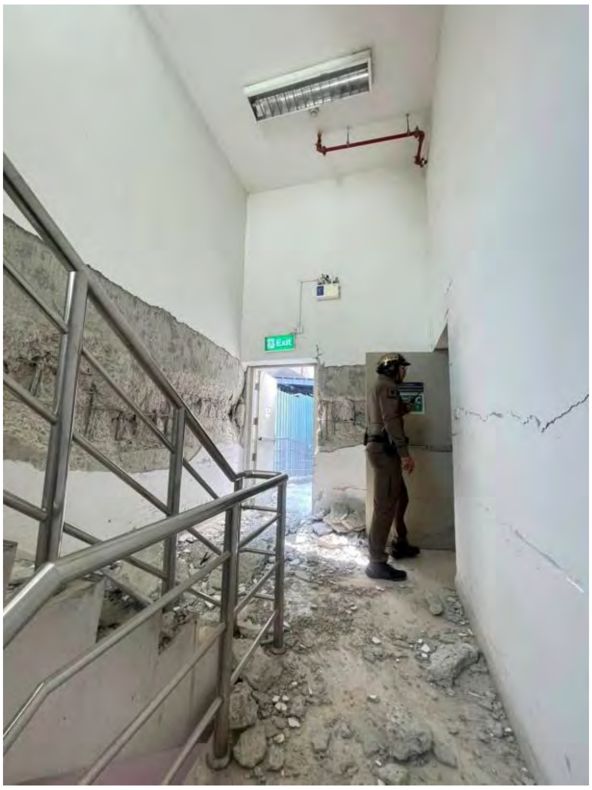


(a) Floor Plan of First Storey



(b) Building H1 Blowout of Damaged Area

Figure 10: Building H1 Plan (First Story)



(a) Through-thickness failure of the wall along gridline Y9

(b) Through-thickness failure of the wall before repair

(c) Through-thickness failure of the wall in the north-south direction

(d) Rebar splicing in the short reinforced concrete wall along the X-direction

Figure 11: Building H1 Structural Damage Photos

(a) Repair to Y-direction wall

(b) Repair to exterior Y-direction wall with epoxied dowel bars

(c) Jacketing repair of wall

Figure 12: Repairs to failed walls in Building H1

3.2.3 Damage to Building R1

Building R1 is a 45-storey apartment constructed in 2025. This apartment building is one of four buildings on the same site with the same architectural layout and structural system. The main structure consists of RC walls with post-tensioned flat slab and two 200 mm thick RC core walls around stairwells and lifts on the south side of the building, with 200 mm thick concrete blade walls in the transverse direction (Figure 13).

The building suffered severe damage through an axial compression failure at the 9th floor the core walls on the south elevation of the building (Figure 14). This resulted in a 150 mm settlement at floor 9, resulting in a residual roof drift of 500 mm (approx. 0.5% residual drift). The walls around the stairwell and lift core were crushed and shortened, as evidenced by the buckled bars that were left protruding from the walls after emergency repairs were conducted. The floors below level 9 either had no damage or very minor cracking.

The likely causes of this damage are not clear. Potential exacerbating factors would be the very narrow aspect ratio and the limited walls in the building's transverse direction, generating large overturning demands on relatively thin (200 mm) core walls on the southern elevation of the building. No lap splices were evident in the buckled bars or damage photos, so it is unlikely that this damage was a result of the termination of longitudinal reinforcement. Another possible contribution is that it is common construction practice in Thailand to use 3-storey jump forms. It is possible that there was an issue with the concrete pour at the start or termination of one of these lifts. No concrete materials testing was available to the reconnaissance team, so the influence of material properties is conjecture at this time. While only one building exhibited significant damage, one of the adjacent towers appeared to have the onset of similar damage at the 9th floor, with a bulging of the wall in a horizontal line (Figure 15). This bulging is also visible one storey above the axial failure in the damaged building.

The building was undergoing emergency stabilisation while the reconnaissance team visited. The stabilisation included propping of the wall with steel columns and corbels to connect the sections above and below the failure plane. The failed core walls were cast with concrete to provide compression capacity, and the buckled longitudinal bars were left on the interior of the building. Following the initial stabilisation, the building was being repaired with a variation of the "speed core" system from AISC using 16 mm plates with bolts cored through the existing wall spanning between floors 8 to 10. Gap between plates and wall will be filled with grout and the plates are connected together with a single groove full penetration weld. Once the grout has cured, the anchors through the thickness of the wall will be pre-stressed.

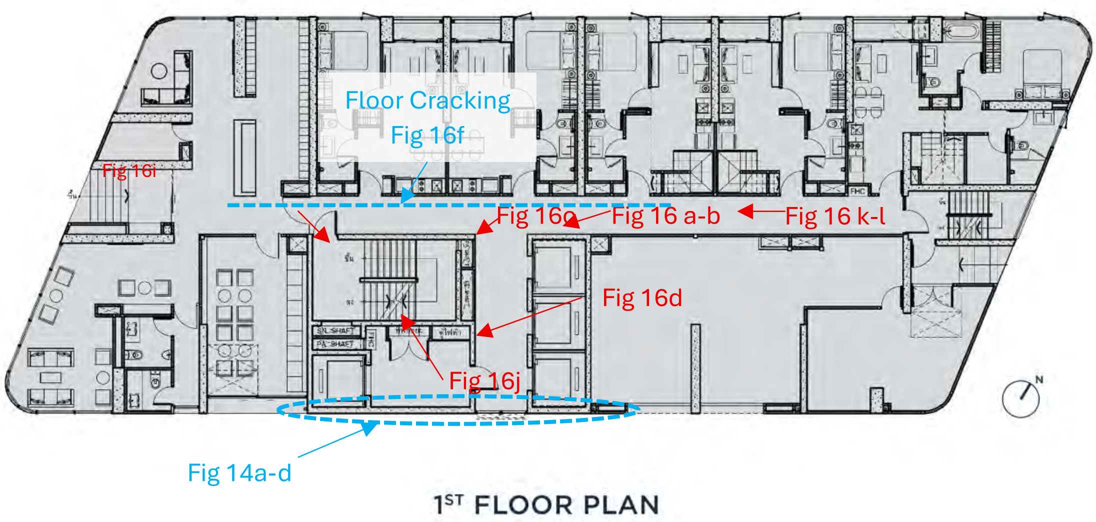
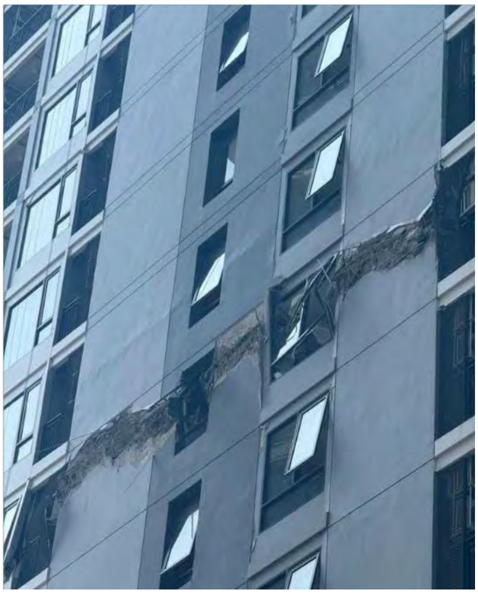


Figure 13: Plan of Building R1

(a) Wall damage on the 9th floor

(b) Wall damage on the 9th floor



(c) Wall damage on the 9th floor

(d) Wall damage on the 9th floor

Figure 14: Structural Damage photos from of Building R1 AIT presentation

(a) Adjacent Building

(b) Observed bulging on the adjacent building

Figure 15: Potential damage to the building adjacent to Building R1. Further investigation did not find any damage to the building in the photograph, nor to the other two buildings. Note that all four buildings have the same architectural and structural configuration, but some minor differences in height and orientation.

(a) Steel plates attached on walls of the 8th floor

(b) Steel plates attached on walls of the 10th floor

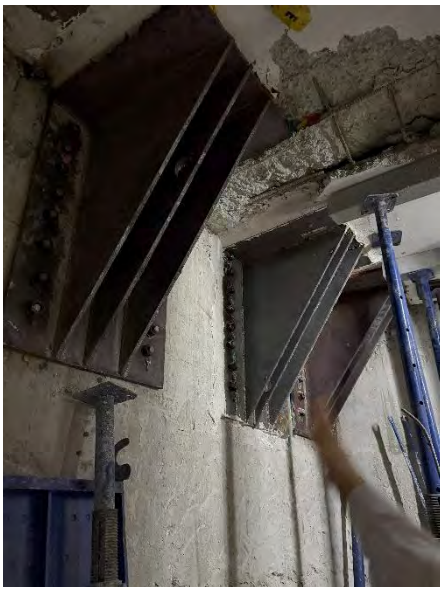
(c) Holes drilled into the wall to have bolts pass through the wall, 9th floor

(d) Buckled bars left exposed, 9th floor

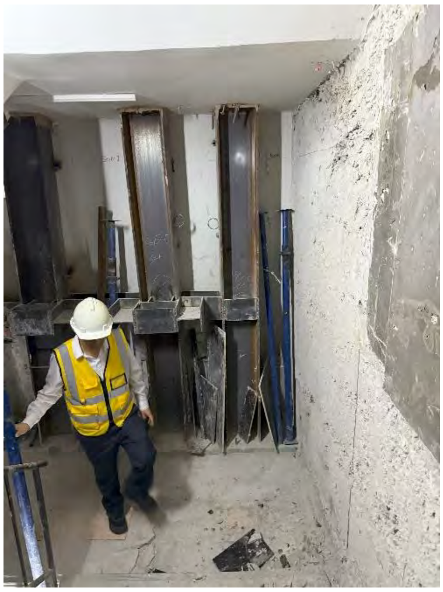
Figure 16 : Building R1 structural damage and repairs

(e) Sandwiching of RC wall with steel plates, wall thickness 200 mm

(f) Damage to the floor



(g) New support system for supporting new steel columns in the upper level



(h) New steel columns supported by corbels

Figure 16 (continued): Building R1 structural damage and repairs

(i) Stair slab cut through to insert a steel plate

(j) Buckled rebar in stair well

(k) Repair and strengthening, 9th floor

(l) Support system along hallway, 8th floor

Figure 16 (continued): Building R1 structural damage and repairs

3.3 Observed damage to non-structural elements

This section documents the observed damage to non-structural elements across the surveyed buildings. Results are presented by Bhatta *et al.* (2026).

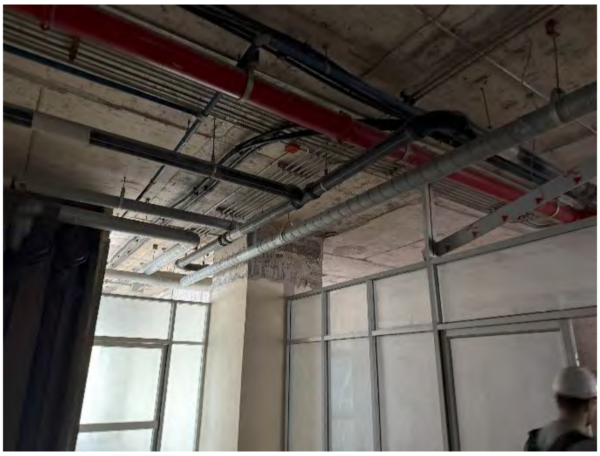
3.3.1 Infill walls (most vulnerable NSE component)

Typical infill walls employed in these buildings were typically autoclaved aerated concrete (AAC) block walls or clay brick infill when located in a high moisture area such as bathrooms or the building exterior (Figure 17). Interestingly, ACC blocks are relatively new to the construction practice; the visited building has hollow concrete block walls.

(a) A view of infill walls and services pipes

(b) Hollow brick (75 mm thick) infill wall

(c) Services pipes layout, note no bracings

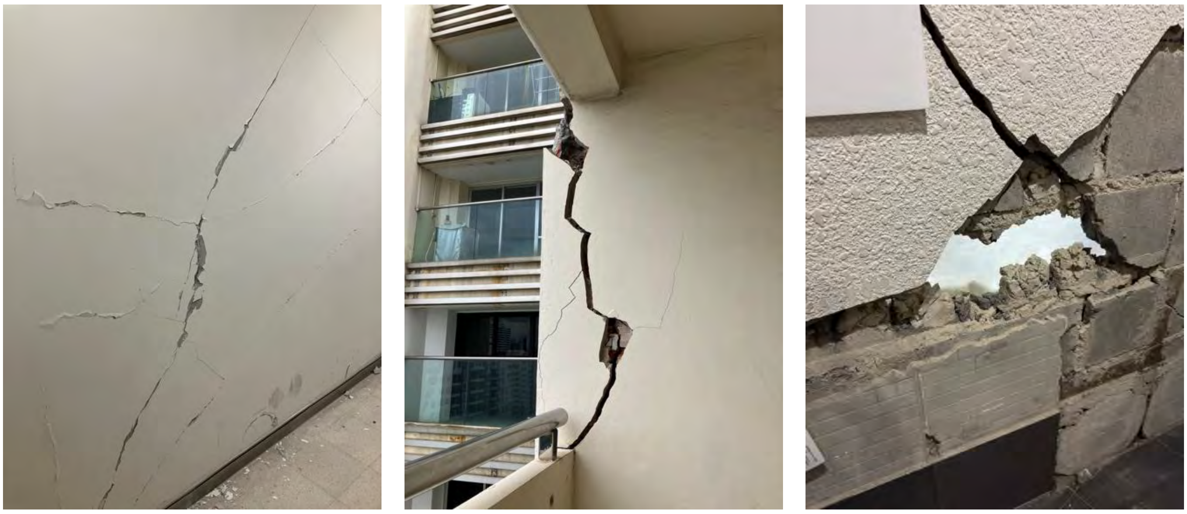


(d) Services pipes layout, note no bracings

Figure 17: Construction of non-structural elements

The infill walls emerged as the most vulnerable NSE component across all buildings. Severe cracking consistently occurred at interfaces with structural elements, particularly around door openings and at slab connections, as seen in buildings R1 (8th and 9th stories) and C2 (10th and 20th stories). The damage patterns revealed height-dependent vulnerability such as the 8th to 15th stories in building R3 and above 57th storey in building R4 showing concentrated damage.

Material performance varied significantly where traditional clay brick infills (in building R2) exhibited brittle failure at building setbacks, while modern cold-formed steel infills (in building C4) demonstrated superior performance with only minor cracking at the interface of structural walls and infills. However, newer buildings using lighter but brittle materials (e.g., aerated blocks) still suffered cracking, suggesting the need for improved seismic detailing. The vertical setback at the 14th floor of building R2 created a stiffness discontinuity that exacerbated exterior wall damage, highlighting how architectural features can amplify seismic vulnerability. Overall, damage was especially prominent at the interface between infill walls and structural elements, indicating deformation incompatibility, stress concentrations and insufficient detailing at these junctions. Typical damage to infill walls is shown in Figure 18.



(b) Damage to interior infill wall on Floor 15 of Building H1

(b) Damage to exterior clay brick wall on Floor 26 of Building R2

(c) Damage to interior infill wall on Floor 17 of Building C2

Figure 18: Photos showing typical damage observed in infill walls

3.3.2 Ceiling and interior finishes damage

Plasterboard ceiling (prevalent in 80% of buildings) showed consistent damage at perimeters (at wall interfaces) (for e.g. in buildings H1, R2, and C2) whereas grid-and-tile ceilings were less commonly reported as damaged (only tile displacement was observed in building H1). The damage severity correlated with building height, with stories one-third to one-half up the building experiencing more pronounced cracking, perhaps due to greater drift demands. The wall-to-ceiling interfaces are observed to be recurring weak points, possibly indicating inadequate edge detailing. Typical damage to ceilings is shown in Figure 19.

(a) Damage to grid-and-tile ceiling on Floor 18 of Building H1 | *(b) Damage ceiling on Floor 15 of Building R2* | *(b) Damage ceiling on Floor 18 of Building R2*

Figure 19: Photos showing typical damage observed in ceilings

3.3.3 Mechanical and plumbing services

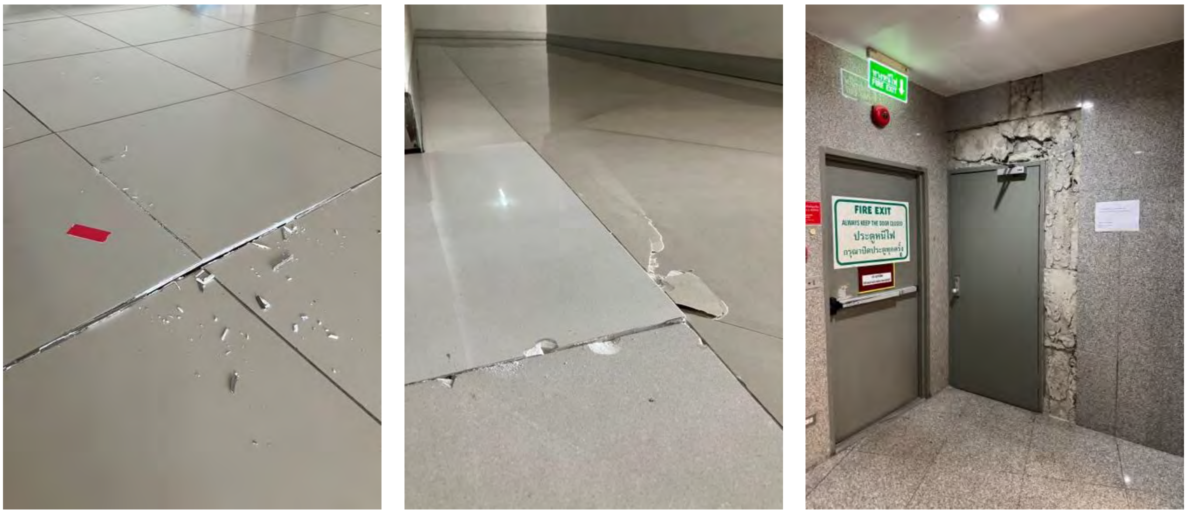
Mechanical services revealed almost no damage and most of them remained functional. Sprinkler pipe interactions caused wall damage in building R1. Water pipework damage led to rebar corrosion and the shutdown of water supply to affected taps in building R3 (in 5th and 6th stories). Although, ejection of sprinkler escutcheons (in building H1) is considered as minor damage it indicates possible interaction between sprinkler heads and adjacent ceiling. There was swimming pool surge tank leakage in building R3 and water tank damage in building R4. Typical damage to mechanical and plumbing services is shown in Figure 20.

(a) Ejection of sprinkler escutcheon on Floor 18 of Building H1 | *(b) Damage to water supply tap on Floor 5 of Building R3* | *(a) Damage leaking water on Floor 5 of Building R3*

Figure 20: Photos showing typical damage observed to mechanical and plumbing services

3.3.4 Floor and wall tiles

Minor to moderate cracking of ceramic tiles was reported in multiple buildings (e.g., building H1, R3, C3), affecting both floors and wall finishes (see Figure 21). Ceramic and granite tiles showed uplift and detachment (e.g., buildings R2 at 39th floor and C3). The upper floors demonstrated greater tile damage, likely due, local deformation or differential movement between structural elements and finishes. In buildings C2 and C3, granite tiles detached and fell, posing potential safety hazards. It is a possible indication of poor anchorage or bonding of the tiles to the substrate.



(a) Damage to floor tiles on Floor 18 of Building H1 | *(b) Damage to floor tiles on Floor 15 of Building R3* | *(c) Damage to wall tiles on Floor 17 of Building C2*

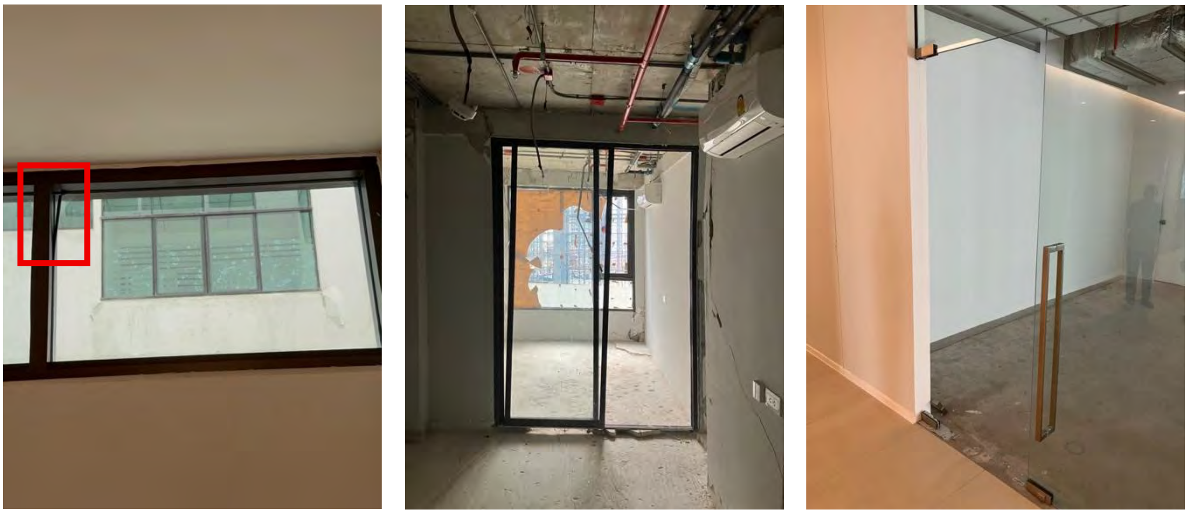
Figure 21: Typical damage observed to floor and wall tiles

3.3.5 Cladding and glazing system damage

Cladding and glazing systems were damaged in several buildings (e.g., in buildings C2, C3), where the damage was mostly concentrated at the interfaces between cladding and structure. In building H1, windows were seen with dislodged gaskets while broken cladding seals had led to water leakage in building R3. In contrast rocking concrete cladding system and modern unitized systems (in building C4) performed better. As such, cladding performance varied significantly between buildings.

3.3.6 Access issues

Jammed doors (in building R1) and damaged glass door (in building C3) lateral deformation in buildings. Moreover, 70% of the 10 inspected buildings, spanning residential, commercial, and hospital occupancies, experienced at least one lift becoming non-operational following the earthquake. This recurring issue suggests that lift systems may be particularly vulnerable to seismic shaking, even in cases where structural damage was minimal. Typical damage to door frames and glass doors is shown in Figure 22.



(a) Dislodgement of window gasket on Floor 18 of Building H1 | (b) Damage to door frame on Floor 8 of Building R1 | (c) Damaged (missing) glass door on Floor 18 of Building C3

Figure 22: Photos of typical damage to doors and windows

3.3.7 Damage pattern across building heights and vertical irregularities

The damage distribution revealed possible vertical trends, with mid-height stories (8th to 20th) showing the most extensive NSE damage across multiple buildings, especially R3 and C2. These stories, perhaps, correspond with peak inter-storey drift ratios in many high-rise buildings, where flexible modes of vibration result in larger relative displacements between floors. Vertical irregularities, like building R2's 14th floor setback, created localised damage concentrations, suggesting that geometric and stiffness discontinuities cause higher deformation incompatibilities resulting in local overstress in NSEs. Building R3 also showed significant damage around the 5th and 6th stories, which housed plumbing, water systems, and a swimming pool surge tank. It indicates that mechanical rooms, typically located in podiums or mid-height areas, can suffer secondary damage due to system failures or water leakage. These patterns also confirm that seismic demands can vary significantly along building height, which can be further exacerbated by mass and geometric irregularities both vertically and in plan. Moreover, preliminary observations suggest that inspected buildings with a higher wall-to-floor area ratio sustained lesser damage as compared with buildings having lower higher wall-to-floor area ratio.

3.4 Functionality

3.4.1 Functional recovery observations

Across all buildings in the dataset, the overall functional impact was moderate, with rapid recovery of essential services but prolonged disruptions to lifts and non-structural elements.

Most buildings regained full or near-full functionality within hours to a few days. In almost all cases, damage to ceilings, interior partitions, and façades did not prevent ongoing use, though repairs were often delayed by insurance processes or limited contractor availability. Several buildings relied heavily on insurance coverage to repair damage.

The persistent source of functional loss across buildings was lift damage, with multiple high-rise buildings experiencing the failure of several lifts, sometimes affecting access to upper floors for weeks or even months. Lifts were primarily stopped due to triggering of seismic safety switches, which lock down the lift to the nearest floor when a shaking exceeding PGA at 1.0 m/s^2 (0.1 g) or less is detected at the base of the lift pit (CEN, 2022). Many systems have a lower bound triggering threshold of 0.1-0.4 m/s^2 and manual reset by a lift technician is required if the lock out is triggered. There was one building that reported having lift cables become entangled. Water supply interruptions were generally short, typically resolved within hours or a few days. Even when water service was disrupted on specific floors, most buildings remained operational. A summary of the loss in services of the visited building is presented in Table 3.

Table 3: Summary of damage services

Building	Total number of lifts	Damaged lifts	Days to full recovery	% of damaged lifts	Days to full recovery of water supply
Building C1	41	13	77	31.7	0.17
Building C2	13	9	77	69.2	0.17
Building C3	33	5	45	15.2	0.00
Building C4	10	0	-	0.0	3.00
Building H1	12	1	5	8.3	3.00
Building R2	7	1	7	14.3	0.17
Building R3	5	1	60	20.0	2.00
Building R4	21	7	10	33.3	0.00
Average			**40.1**	**24.0**	**1.1**

Perceptions of safety played a role in a few cases, influencing decisions about occupancy even when buildings were structurally sound.

Overall, the building stock showed good functional performance, with rapid recovery of basic services, manageable non-structural damage, and the main long-term impact resulting from the repair of lifts.

3.4.2 C1 – Commercial Building

After the earthquake, all occupants were evacuated, but they were able to return to use within a few hours, indicating a rapid recovery of essential services. Damage was largely manageable and covered by insurance. Functional disruption was primarily caused by lift outages, with 13 of 33 lifts damaged; 4 had already been repaired, and the remaining were in progress. Minor issues such as leaks from expansion joints contributed to reduced operational capacity but did not prevent reoccupation.

3.4.3 C2 – Commercial Building

This building regained full functionality within 4 hours, showing strong immediate resilience. However, some architectural systems, such as façade, glazing, ceilings, lighting, and interior partitions, sustained damage, with repairs pending up to the visit. Functionality was also influenced by occupants' perception of safety rather than by complete service failures. The lift system experienced significant loss, with 9 of 13 lifts non-operational, and insurance was expected to cover most repairs.

3.4.4 C3 – Commercial Building

The building remained in use despite damage, achieving full functionality at the time of reporting. The most significant disruption came from lift failures: 5 of 33 lifts lost functionality, largely due to cable twisting, and restoring full operation took around 45 days. Damage to ceilings and interior walls did not affect building usability. Repair delays were mainly administrative (e.g., slow quotation processes), though insurance will cover costs.

3.4.5 C4 – Commercial Building

This building (started construction in 2018) recovered functionality within 3 days, once water supply was restored. Although it suffered ceiling and interior wall damage, these repairs were completed within one week and one month, respectively. The main cause of downtime was the loss of water supply, which required 3 days to resolve. Additional delays were attributed to engineering mobilisation rather than structural issues.

3.4.6 H1 – Hospital

The building was partially usable following the event. A single lift (one of twelve) failed but was repaired within 5 days. Functional loss was mostly due to ceiling and interior wall damage, which took two months to repair and limited floor access. Water supply recovered within 3 days, but overall use depended heavily on accessibility constraints and users' perceptions that the building was unsafe, despite services largely functioning.

3.4.7 R2 – Residential Apartments

This apartment building (built 1993) returned to full functionality within 4 hours and never fully lost any major services. Slight interior and façade damage remains unrepaired but does not affect usability. One of the seven lifts failed and required one week for repair. Overall, the building maintained strong functional continuity throughout.

3.4.8 R3 – Residential Apartments

The building (opened in 2006) regained full operation within one day, with only isolated service loss. The 6th floor lost water for 2 days, but occupants continued to use the building. One lift (1 of 5) was non-functional for two months, and delays in repairing walls and ceilings were tied to insurance processes. Façade and internal damage were scheduled for completion within the next two months.

3.4.9 R4 – Mixed-Use High-rise

The building (built between 2008–2009) was fully functional but required two months to regain full capability. Lift disruption was severe: 7 of 21 lifts failed, and only one lift initially reached the 54th floor, forcing residents on the 54th–73rd floors to walk upstairs until additional lifts were restored over a 10-day period. A resident on the 74th floor required temporary relocation. Water tank damage was repaired the same day and did not impact use. Minor non-structural damage in ~30 apartments did not prevent occupancy. The hotel component reopened 3 days after the earthquake; however, the building manager noted that repairs progressed slowly because work could only be undertaken between 10 am and 3 pm, when most guests had checked out. Repair costs will be claimed from insurance later.

4 Plans for Analysis and Further Work

The observations collected during the Bangkok reconnaissance provide a strong foundation for a more detailed analysis of building performance during large distant earthquakes. Several avenues for further work have been identified.

First, the functionality survey data collected from building managers will be analysed to better understand how disruptions affected building use, with a focus on lifts. Notably, the minimal structural damage in these surveyed buildings indicates that lift failures occurred independently of severe structural damage. The unique dataset collected during the visit, which systematically captures lift functionality, damage rates, recovery metrics, photos, and the impact on usability, along with measured building periods from on-site sensors and PGAs at stations, enables direct comparison with existing fragility models.

Second, collaboration with Thai partners will continue to support the interpretation of basin-related ground motion characteristics, including the influence of deep sedimentary structures on long-period amplification.

5 Conclusion

The 28 March 2025 Mandalay earthquake provided an important opportunity to evaluate the performance of Bangkok's high-rise buildings under long-period shaking generated by a distant seismic event. The reconnaissance findings show that while structural systems generally performed well, non-structural damage was widespread and had substantial implications for building functionality.

The ambient vibration recordings at building foundation levels, identified the fundamental period of vibration of soil over bedrock at the visited building sites, which ranged from 6.7 to 7.8 seconds, matching the range of long period amplification observed in the earthquake ground shaking. Furthermore, ambient vibration recordings at the top-most storey of buildings were used to estimate the fundamental period of vibration of the buildings.

Across the 10 buildings investigated, infill walls, ceilings, façade components, and interior finishes exhibited the most consistent damage, particularly at mid-height levels where inter-storey drifts were greatest. Vertical irregularities such as setbacks further amplified localised damage. Structural damage was limited overall, though isolated shear wall failures highlight the need for improved detailing and construction quality in some buildings.

Functional impacts were generally moderate, although some disruptions had significant consequences for building usability and recovery. Most essential services were restored quickly, but lift outages emerged as the most persistent challenge, affecting 70% of lifts and delaying full functionality for weeks to months. These disruptions, combined with occupant perceptions of safety and administrative repair delays, underscore the complex interplay between physical damage, operational capacity, and human factors.

The observations from Bangkok provide several important lessons for cities with growing numbers of tall buildings, particularly Auckland, where deep sedimentary deposits and long-distance earthquake sources are a fact. The widespread non-structural damage and prolonged lift outages observed in Bangkok highlight the need for greater attention to the seismic performance and functional recovery of non-structural systems in New Zealand high-rise buildings. In particular, façades, ceilings, partitions, and building services require improved detailing and recovery planning to reduce downtime following future earthquakes. The findings also emphasise the importance of considering occupant perceptions, post-event inspection processes, and communication strategies as part of resilience planning, as these factors strongly influence building re-occupancy and recovery. More broadly, the reconnaissance reinforces the value of integrating functionality and recovery objectives into New Zealand's seismic resilience frameworks, especially for urban centres exposed to long-period basin effects and distant large earthquakes.

The insights gained from this collaboration between Thai and New Zealand partners contribute to improved understanding of distant-earthquake effects and support efforts to strengthen the resilience of high-rise urban areas.

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A Appendix A – Outline of Delegation meetings and Activities

A.1 Meeting with Dr Punchet Thammarak

On day one, the delegation met Dr Punchet Thammarak at the Asian Institute of Technology (AIT) in the morning. Dr Thammarak was previously a lecturer at AIT and currently owns and operates a company for building inspection and monitoring. Following the Mandalay Earthquake, he inspected several buildings and shared his initial insights with the delegation. The conversation with Dr Thammarak highlighted the societal issues that shape perceived risk, which is further complicated by political and financial factors.

Dr Thammarak shared several insights which included:

- Inspections in tall buildings revealed hidden structural vulnerabilities and inconsistent construction quality (especially in core walls).
- Socioeconomic factors limiting earthquake/seismic awareness.
- Public perceptions and informal incentives, rather than legal enforcement, were most likely driving owners' post-earthquake inspections.
- Concerned international companies/tenants have also conducted a detailed inspection of the occupied building, perhaps due to safety concerns.

A.2 Meeting with Professor Pimanmas and the Thailand Structural Engineers Association - Arranged visit to a hospital

A meeting with Professor Amorn Pimanmas (Professor at Kasetsart University and President of the Thailand Structural Engineers Association), Engineer Watanapong Hiranmarn, (General Secretary of the Thailand Structural Engineers Association (TSEA)), and three structural Engineers from Arup (Theerasak Kaewkhluab, Phamornsak Noochit, and Mei) facilitated the sharing of insights from the TSEA community and the identification of sites for the delegation to visit.

Professor Pimanmas presented the background of seismic design in Thailand and the structural performance of buildings in Bangkok following the Mandalay Earthquake. He noted one building collapse, 10+ high-rise buildings with observed structural damage, and 100's of buildings with observed non-structural damage. Two and a half months after the event, repairs and retrofits were underway at most buildings.

In the afternoon, the delegation visited a hospital that had sustained structural damage to the shear walls around one stairwel. We were welcomed by hospital workers, including the Hospital Director, and engineers working on the repair. The Hospital Director provided useful insights into the perceptions of building users regarding the earthquake and the building's functional recovery, which informed the results noted in the main body of this report.

A.3 Meetings with Professors Warnitchai, Leelataviwat, and Poovarodom

The delegation met with Profs. Pennung Warnitchai (AIT), Sutat Leelataviwat (KMUTT), Nakhorn Poovarodom (TU) and at AIT. Prof. Leelataviwat provided an overview of non-structural damage and shared a large repository of buildings with non-structural and structural damage. Prof.

Warnitchai provided an overview of the seismological and earthquake engineering context of Bangkok and Chiang Mai. Prof. Poovarodom provided info on micro-zonation of Bangkok and a low-cost accelerometer network. He emphasised the contrasting ground motions and structural responses observed in Bangkok and Surin—two cities located at similar distances from the Myanmar epicentre—attributing the differences to the presence of deep and extensive soft soil deposits beneath Bangkok.

Following this visit, Prof. Leelataviwat helped the delegation identify and secure permission for several site visits during the remainder of the delegation's time in Bangkok.

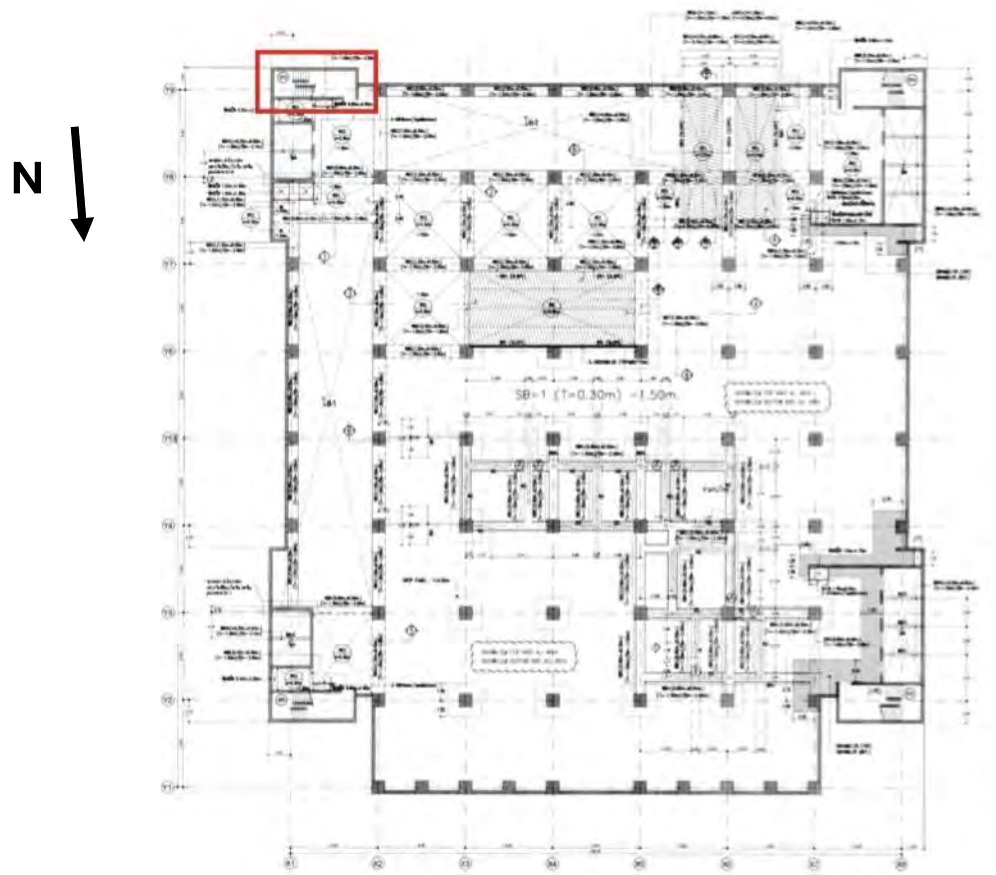
A.4 Meeting at Department of Public Works and Town & Country Planning (DTP)

The delegation visited the Department of Public Works and Town Planning for a meeting to exchange knowledge and best practices in post-earthquake repair and retrofitting. Mr. Pongnara Yenying, Director-General of the Department of Public Works and Town Planning, assigned Mr. Pisut Sukhum, Chief Engineer of the Department of Public Works and Town Planning, to welcome the research team from New Zealand. The meeting aimed to analyse lessons learned from the Mandalay earthquake, focusing on infrastructure, critical damage, and building repair and renovation in Thailand. The objective of the visit was to collect data for research and exchange knowledge on building assessment in earthquake-prone areas, such as Bangkok.

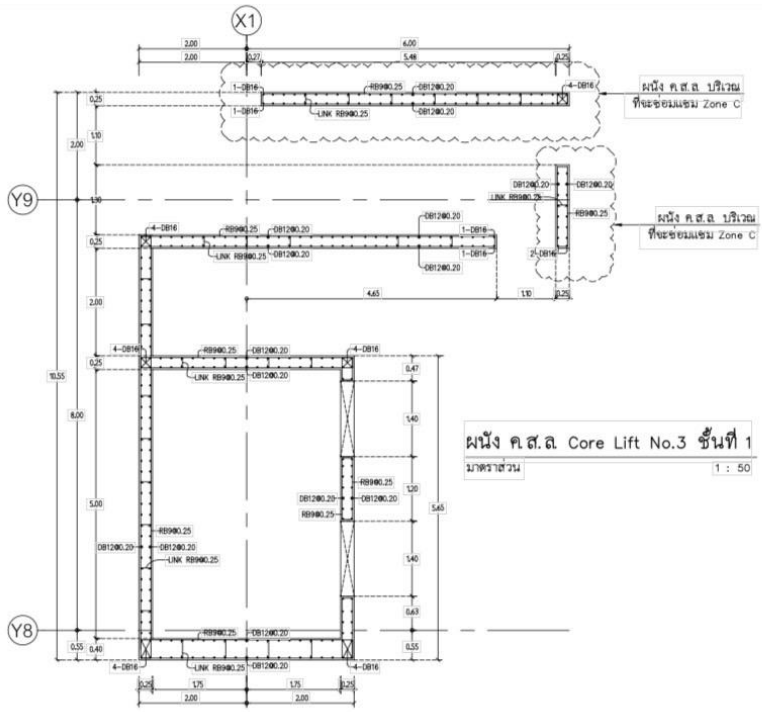
B Appendix B – Building Reports

B.1 Building H1

Building use	Hospital
Tag	Yellow
Structure	RC Frame with post-tensioned flat slab and RC walls (200 mm thick)
Floors	25
Year	2019
Main Structural damage	• Through-thickness failure of two walls in the southeast corner of the building in ground floor only • Longitudinal bars buckled along the wall length
Likely causes	• Appears to have occurred at a cold joint in the wall above the lap • Poor confinement • Low concrete strength – samples gathered by Thai engineers showed approx. 15 MPa while 30 MPa specified
Non-structural elements typology	• Infill walls: Aerated infill block (cladding and interior walls) and hollow clay block (around toilets) • Mostly plasterboard ceilings, grid-and-tile ceilings in washrooms • Service pipes without bracings
Main Non-Structural Damage	• Minor to moderate damage to many infill walls • Minor damage to plasterboard and grid-and-tile ceilings • Minor damage to ceramic tiled floor and wall coverings • Dislodgement of gaskets in exterior windows • Instances of sprinkler ring ejection (escutcheon plates)
Functionality	• One out of 12 lifts stopped working and was fixed after 5 days • Building is partially in use – all services are working. Damage to the walls and ceiling stopped the use. • Water supply recovered after 3 days • Functioning floor – ceiling and interior walls repairs took 2 months. • In general, the building lost function due to lack of accessibility, damage to architectural systems (e.g., ceilings; interior partitions; cladding), and perception of occupants that the building is unsafe.
Comments	• Damaged walls on the exterior of the stairwell. • Repair being undertaken while on site, repair is utilizing 90-degree hooks at the boundary for transverse reinforcement • Building did not lose functionality immediately after the earthquake (11 operations active during the earthquake), no issues with backup generators, decision was made to vacate the building due to wall damage and repairs required to internal wall finishes. • Building renovation happening simultaneously - building expected to be fully operable in July 2025 – opening floor-by-floor. • Aerated infill block (block + CS plaster on both faces with chicken mesh) and hollow clay block (with CS plaster). Block thickness 75 mm. Bond beams are included in the tall wall. • No diagonal bracing to hanging services ducts.



(a) Floor Plan of First Storey



(b) Building H1 Blowout of Damaged Area

Figure 23: Building H1 plan (1st story)

(a) Repair to through-thickness failure of wall in y-direction

(b) Repair to through-thickness failure of wall in x-direction

(c) Through-thickness failure of the wall in the X-direction

(d) Through-thickness failure of short wall in X-direction with repair initiated, looking from inside

Figure 24: Structural Damage photos of Building H1

(e) Rebar splicing in the short RC wall along X-Dir

(f) Jacketing repair of wall

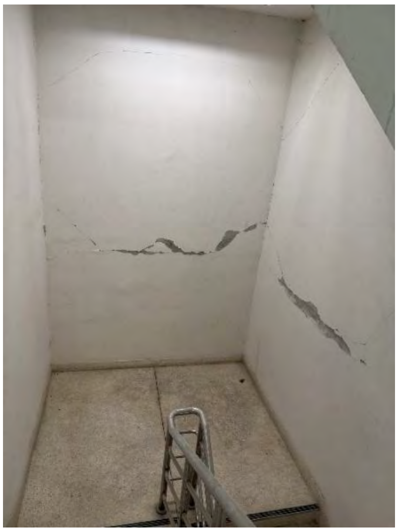
(g) Damage to wall on gridline Y9 the perimeter (next to stair)

(h) Repair to exterior Y-direction wall with epoxied dowel bars

Figure 24 (continued): Structural Damage photos

(i) Damage to Y-dir wall on the perimeter (next to stair), wall under repair

(j) Damage along cold joint

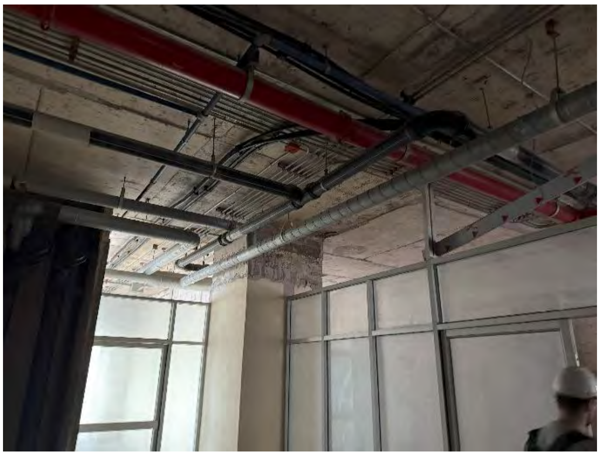


(k) Cracking looking towards basement from 1st floor

Figure 24 (continued): Structural damage photos of Building H1

(a) AC infill wall (75 mm thick), note mostly dry joints

(b) Hollow brick (75 mm thick) infill wall



(c) A view of infill walls and services pipes

(d) Services pipes layout, note no bracings

Figure 25: Construction of non-structural elements

(a) Damage to infill wall

(b) Falling of sprinkler rings (escutcheon plates)

(c) Damage to ceramic tile wall covering

(d) Damage to ceramic tile floor covering

(e) Damage to plasterboard ceiling

(f) Ceiling tile dislodgement

(g) Dislodgement of window gasket

Figure 26: Typical Non-Structural Damage photos in the 18th Story

B.2 Building H2

Building use	Hospital building (under construction)
Tag	Green (no damage)
Structure	RC
Floors	11
Year	Under construction (structure complete, non-structural walls under construction, services not installed
Main damage	No damage
Likely causes	N/A
Non-structural elements typology	Infill walls: Aerated infill block Mechanical services are hung without braces.
Comments	• RC columns + stairwell with RC core in all four corners • Prestressed bonded slab

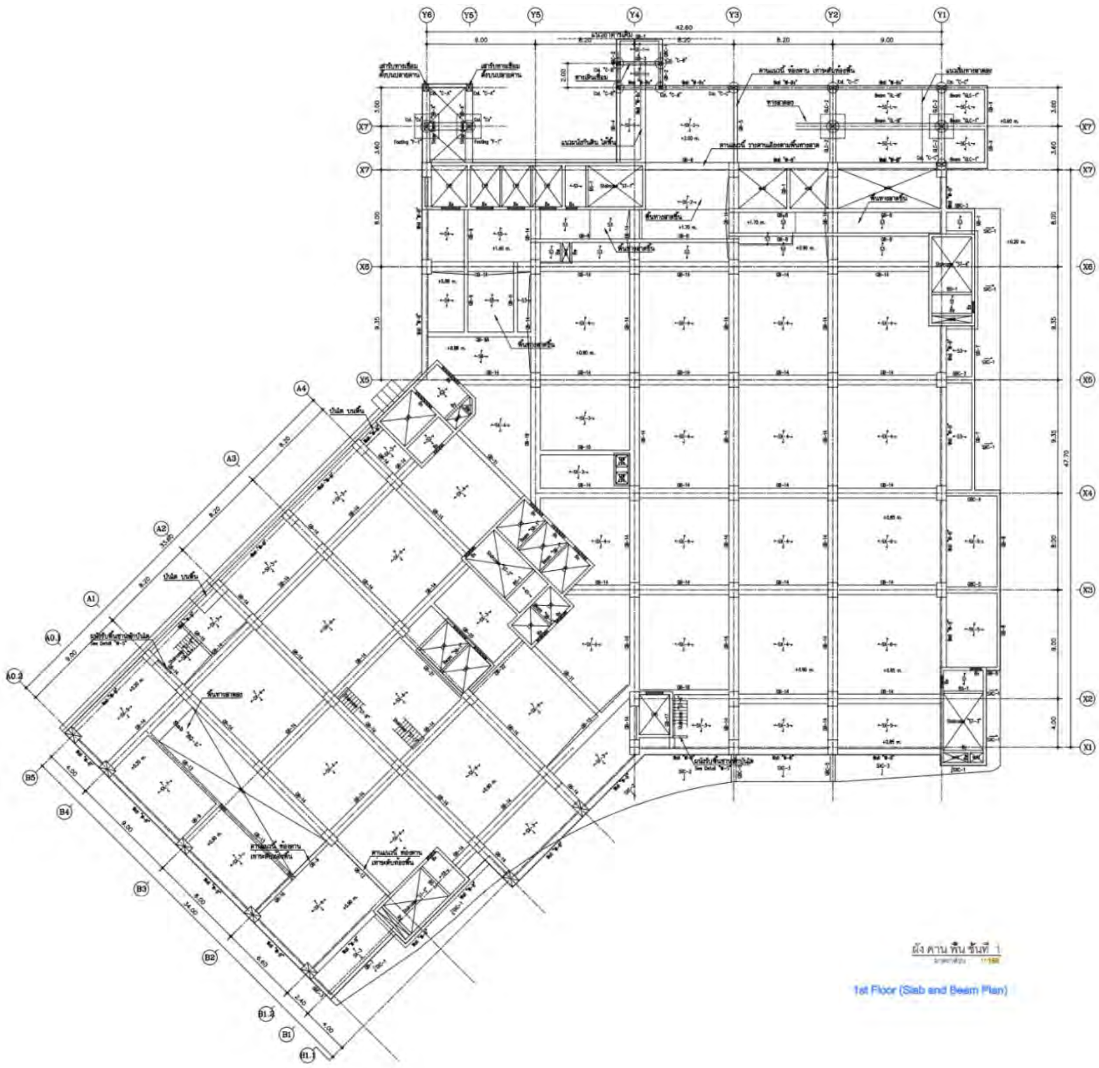
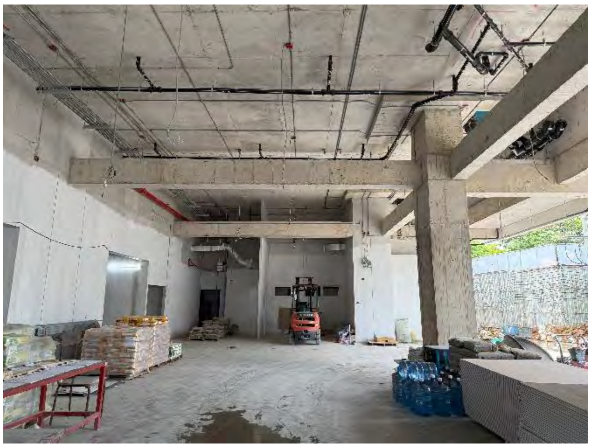
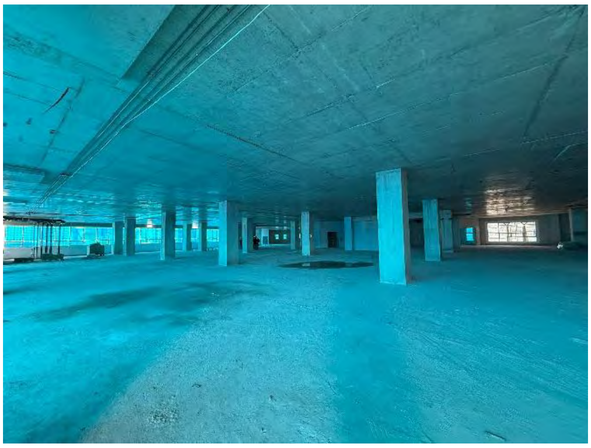


Figure 27: Key plan Building H2 Green



(a) Bracing beam

(b) Staircase with structural RC walls (typical thickness 250 mm thick)



(c) A view of PT slab

(d) PT slab with column head

Figure 28: The building structure

(a) Non-structural infill wall (wall height approximately 6 m) (ground floor)

(b) Typical hanging arrangement of services conduit system

Figure 29: Construction of non-structural elements

B.3 Building R1

Building use	Apartment (a group of four buildings with the same design)
Tag	Red
Structure	RC walls with post-tensioned flat slab and RC walls 2 "core walls" around stair wells and lifts, perimeter wall on the side and concrete blade walls in the transverse direction
Floors	45
Year	2025 (just completed, likely designed following the 2019 code)
Construction system	RC walls in the long direction, blade columns in the short direction and columns
Main Structural damage	• Horizontal compression failure at floor 9 in all core walls • 150 mm settlement at floor 9, resulting in a roof drift of 500 mm (approx. 0.5% residual drift)
Likely causes	• Overturning of the building in the transverse direction and overloading of stairwell/ lift core walls, resulting in concrete compression crushing/ buckling of bars • Too few walls (2 walls, app 4 m long and 200 mm thick) in the building's longitudinal direction to carry overturning demand • Higher mode effects or poor concrete quality in one pour. Similar wall-crushing damage (see below) has been observed in an adjacent building (at least one building). Bar buckling damage in the adjacent building is manifested by a ripple in the wall at level 9.
Non-structural elements typology	• Infill walls: Masonry infills with mortar and wire-mesh • Primarily plasterboard ceiling on light gauge steel grid
Main Non-Structural Damage	• Severe damage to walls on 8 and 9 stories • Minor damage to plasterboard ceiling • Doors were jammed – hard/unable to open due to unlevel floor • Sprinkler pipe and infill wall interaction damage • Minor damage in-infill walls at the corners of doors • Severe damage to glass door frames
Functionality	• Building was partially occupied as construction was being completed, and tenants have been relocated to adjacent buildings
Comments	• The roof displacement is approximately 50 cm. • The concrete slabs lost elevation on story 8 and 9 (and above) have settled about 15 cm. • Building being repaired with "speed core" system from AISC using 16 mm plates with bolts cored through the existing wall spanning between floors 8 to 10. Gap between plates and wall will be filled with grout, plates connected with single groove full penetration weld. Plates then tensioned. • Buckled bars left exposed post stabilisation concrete pour • Propping columns only went down to floor 3 • Based on ASCE 7 building would have been designed with 5% damping (likely applied as mass proportional) and using an R=5 for ordinary shear walls • The other 3 buildings on site, which were similar in form and age, had no visible structural damage • Very minor to no damage on lower floors

	• Jump forms in typically use 3 floor lifts and 3 m lifts for concrete pours in Thailand – damage location possibly related to construction • No laps shown in damage or buckled bars • RC columns and walls • App 200 mm thick many blade walls in the transverse direction • App 200 mm thick walls around staircase and lift core • Likely (?) 250 mm thick prestressed bonded floor slab • ACC blocks are sandwiched with chicken wire mesh and cement sand plaster on both surfaces.
Special note	At least one adjacent building has the following potential damage: • Onset of buckling of rebars of the external wall at level 9 along the staircase and lift core. Buckling of rebars is indicated by the rippling of the concrete surface as visible from the driveway. Spalling of concrete not seen.

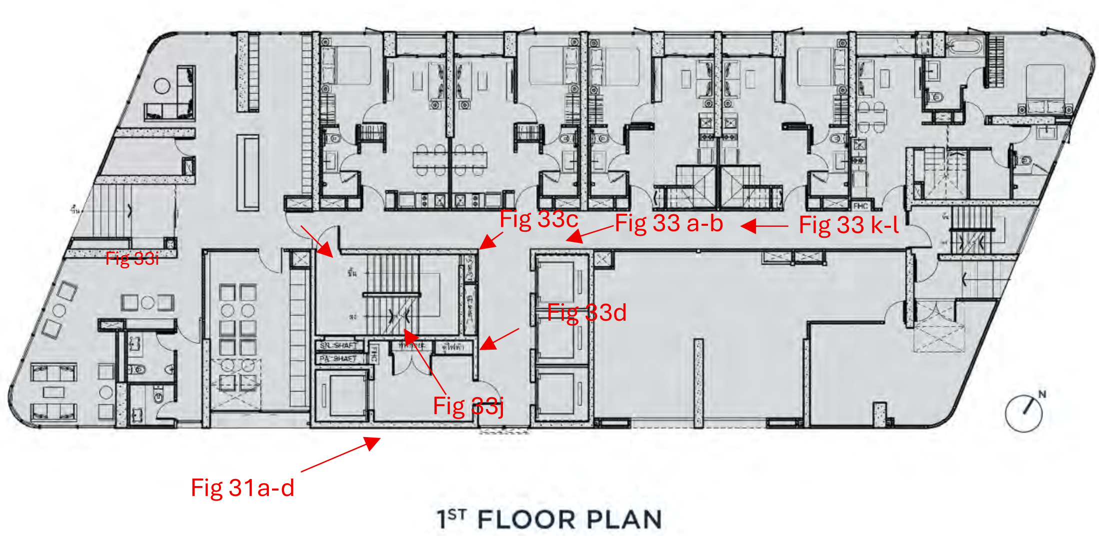


Figure 30: Building R1 plan

(a) Wall damage on 9^{th} floor

(b) Wall damage on 9^{th} floor

(c) Wall damage on 9^{th} floor

(d) Wall damage on 9^{th} floor

Figure 31: Structural Damage photos to Building R1 from AIT presentation

Figure 32: Cracking to the floor in Buildilng R1

(a) Steel plates attached on walls of 8th floor

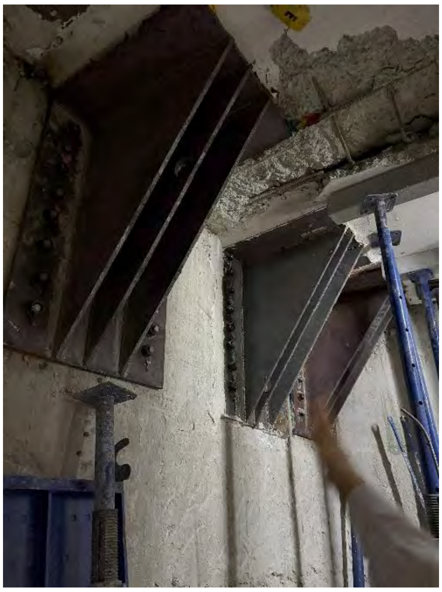
(b) Steel plates attached on walls of 10th floor

(c) Holes being drilled into the wall for bolts pass through the wall, 9th floor

(d) Buckled bars left exposed, 9th floor

Figure 33: Repair of Building R1

(e) Sandwiching of RC wall with steel plates, wall thickness 200 mm



(f) New support system for new steel columns on the level above

(g) New steel columns supported by corbels

(h) Stair slab cut through to insert steel plate

Figure 33 (continued): Repair of Building R1

(i) Buckled rebars in stair well

(j) Repair and strengthening, Level 9

(k) Support system along the hallway, Level 8

Figure 33 (continued): Repair photos (taken during the site visit)

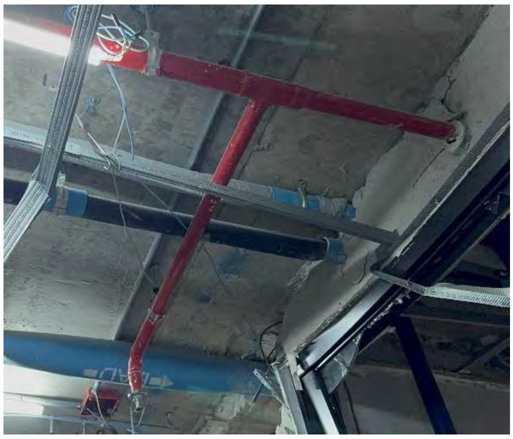
Figure 34: Potential damage to the adjacent building to Building R1

(a) Damage to infill walls on 8th and 9th stories (typical)

(b) Damage to plasterboard ceilings on 8th and 9th stories (typical)

Figure 35: Non-Structural damage photos to Building R1

(c) Damage to glass door frames, 9th storey



(d) Damage to infill walls due to interaction between pipes and wall, 9th storey

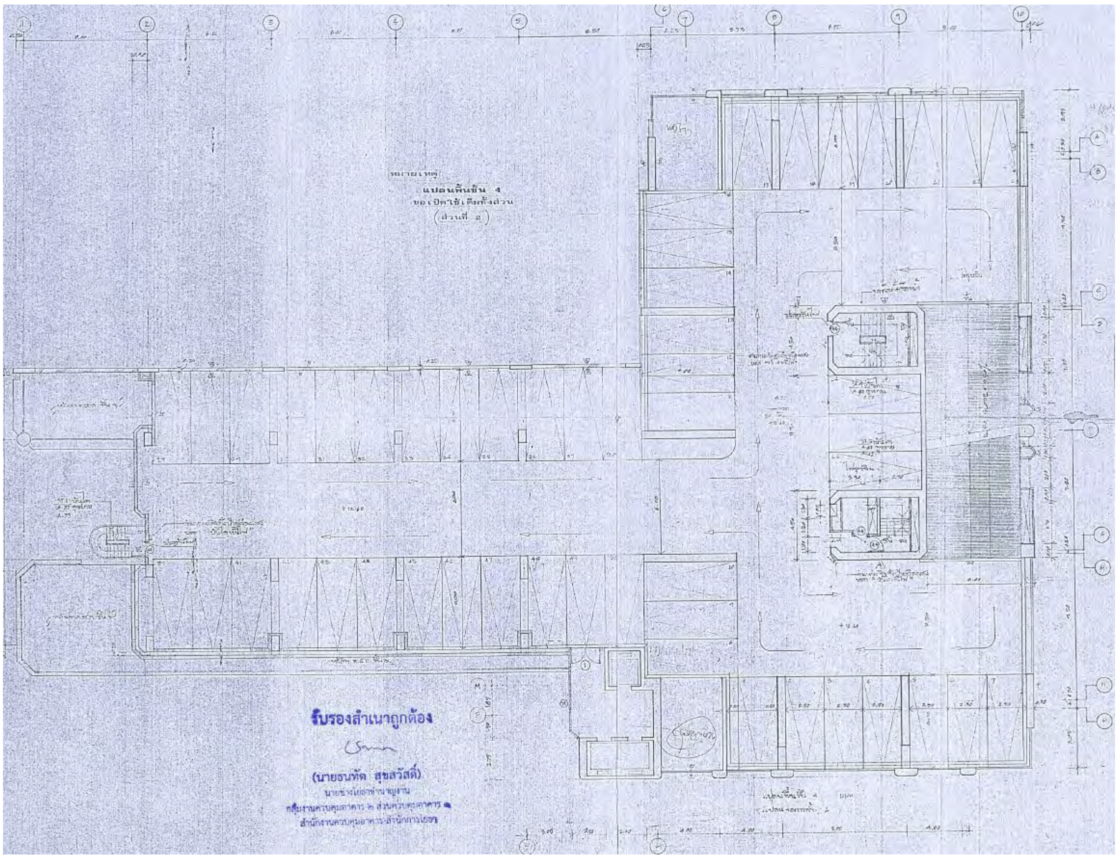
(e) A typical infill wall constructed of AC blocks with chicken mesh plastering

(f) Infill walls in toilet areas

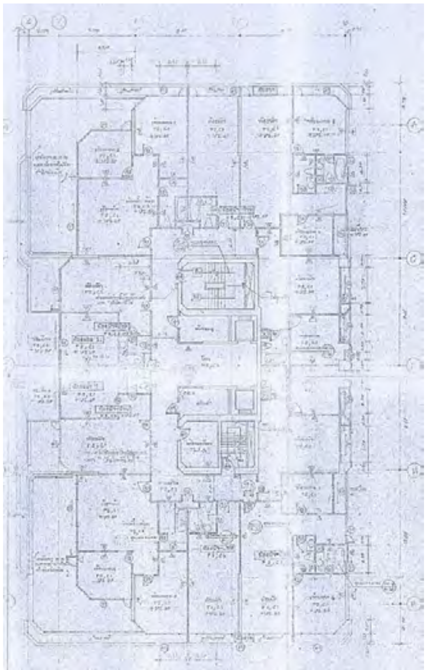
Figure 35 (continued): Non-Structural damage photos to Building R1

B.4 Building R2

Building use	Apartments
Tag	Green (no damage)
Structure	RC two core walls around lifts (~300 mm thick) ~400 mm thick planar walls
Floors	45. Set back and pool deck at 14 storeys
Year	1993
Main Structural damage	• None
Likely causes	• N/A
Non-structural elements typology	• Infill walls – clay brick with mortar • Ceilings - Plasterboard ceilings
Main Non-Structural Damage	• More non-structural exterior damage was observed where there were changes in floor plans due to setbacks (stepwise building). • Severe damage to exterior clay brick wall. • Minor-to-moderate damage to plasterboard ceilings • Minor cracks at ceiling and wall joints • Uplift/Popping of floor tiles on top floors
Functionality	• One out of 7 lifts stopped working and was fixed after a week • Building is fully functional after 4 hours. Never lost services fully. • Damage to the façade is still not repaired. • Come cracks on interior walls and ceilings and haven't been repaired
Comments	• The building was 80% occupied. Owners moved out because they were scared of another big shake (in higher stories). Also, owners moved out because the swimming area was closed. • The property manager started working in November last year and had to deal with post-EQ recovery. • Exterior repairs will cost at least $20 million (Baht). Insurance will pay for 90% of the repair costs. But there are no builders to do the work. • Old cracks (hairline) on infills were present prior to the earthquake, perhaps due to aging, which were widened and elongated after the earthquake. • NSE damage occurred throughout the building, and significant damage was concentrated between 10 and 20 stories. • Signs in lobby and lift declaring that building has been inspected by an engineer (engaged by the building management company) and they have "certified that it is safe for occupancy"

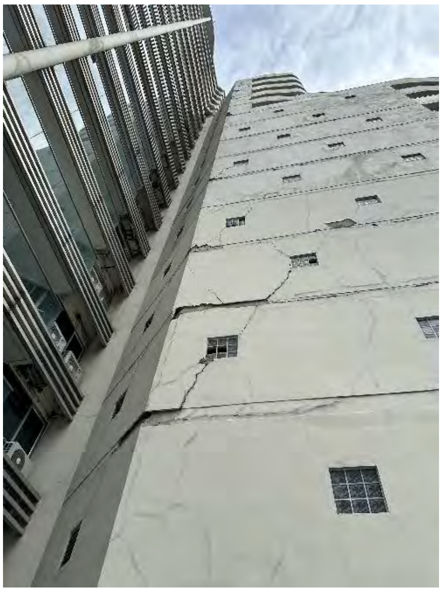


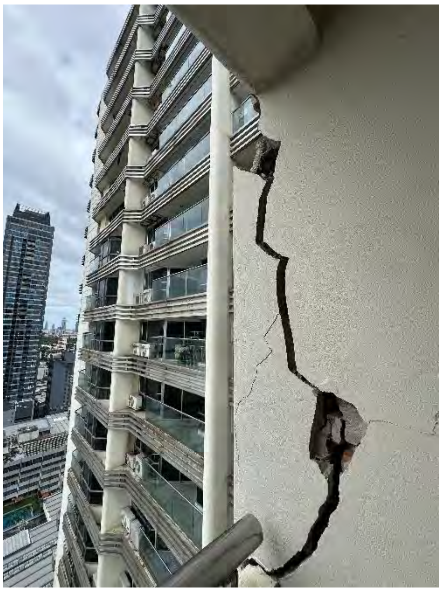
(a) Typical lower storey plan



(b) Typical tower plan

*Figure **36**: Building R2 plans*





(a) Moderate cracking around the perimeter of the exterior masonry partition wall

(b) Severe cracking to the stairwell and masonry partitions

Figure 37: Damage photos of Building R2

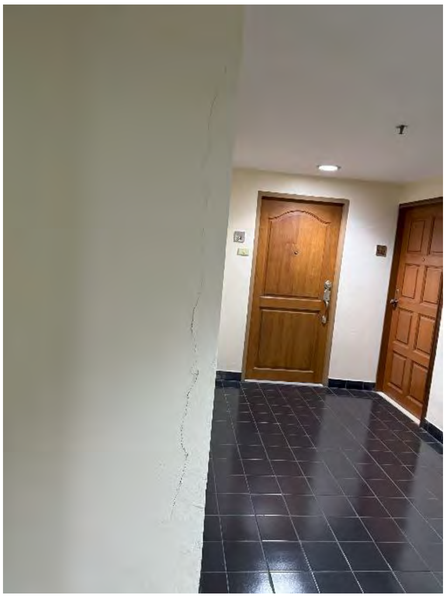
(a) Damage to exterior clay brick wall as seen from outside

(b) Damage to exterior clay brick wall as seen from inside (storey 26)

(c) Popping of ceramic tiles on the 39th floor

(d) Damage to plasterboard ceiling (storey 18)

Figure 37: Non-Structural damage photos of Building R2



(e) Cracking to non-structural wall junction

(f) Tile spalling

(g) Cracking to wall and ceiling

Figure 37 (continued): Non-Structural damage to Building R2

B.5 Building R3

Building use	Apartments
Tag	Green (no damage)
Structure	RC core. Flat slab with band beams
Floors	30. 28 occupied floors with 2 floors of lift overruns and roof, carparking up to Level 5 and upper residential
Year	2006
Main Structural damage	• None
Likely causes	• N/A
Non-structural elements typology	• Infill walls – Mainly hollow concrete blocks (75 mm thick), AC blocks (75 mm thick) • Plasterboard ceiling
Main Non-Structural Damage	• Minor cracking to infill walls and wall tiles • Minor misalignment or popping of tiles on core wall, spread out over the building • Joint cracks around infill to concrete slab joints around the 6^{th} floor • Minor uplift and cracking damage to floor tiles • Minor cracking to ceilings and wall to ceiling joints • Damage to exterior cladding seals at the interface with the structure resulting in leaking • Damage to water pipework on the 5^{th}, 6^{th} and 24^{th} floor (rebar corrosion) • Water supply to the tap with damaged pipework – shut down. • Damage to the swimming pool surge tank on 6^{th} floor caused a leak into carpark • Short height boundary wall damage and separation from the building • Damage mostly between 8 and 15 storeys
Functionality	• Building regained functionality as per today, a day after the earthquake • Floor 6th out of water due to damage and recovered function after 2 days. This did not stop the occupants from using the building. • Lift number 4 (out of 5 lifts) lost functionality for 2 months • Damage to walls/ceilings did not stop occupants from using the building. Repairs to the building have been delayed due to insurance issues. • Façade and interior walls damage will be fixed in the next 2 months
Comments	• The water system was broken on the 24th floor. • Some of the emergency stairs were not opened to occupants. • 245 units in total, and just 60 units had damage. • After the earthquake, people were evacuated from the building for 2-3 hours before they were allowed back in again. • In 2004, the construction started, and it was finalised and opened to occupants in 2007. • Everything was working after 4 hours. A week later, an inspector came to check the building. • The water system was also broken on level 6th, damaged by the wall next to the swimming pool. • Lift 4 has an issue, and it was repaired after 2 months.

	• Exterior damage is not fixed yet, but it will be in the next 2 months. Insurance will cover 40%, since cracks were there before the EQ, but got bigger after the EQ. Rainwater comes in through the cracks. • Building owners would have been okay if the insurance had covered 80-90%, but since it just covers 40%, repairs have taken longer. • Proportions of the structural elements were very different from R1 and R2, having a “larger” dimension for columns, walls, beams, and slabs. • Some cracks on infills were evidently older than the earthquake. The facilities manager noted that the earthquake had worsened these. • Signs in lobby and lift declaring that building has been inspected by an engineer (engaged by the building management company) and is “safe and can be inhabited” • The damage inside the rooms is unclear, as the infill walls were double walls (with hollow blocks).

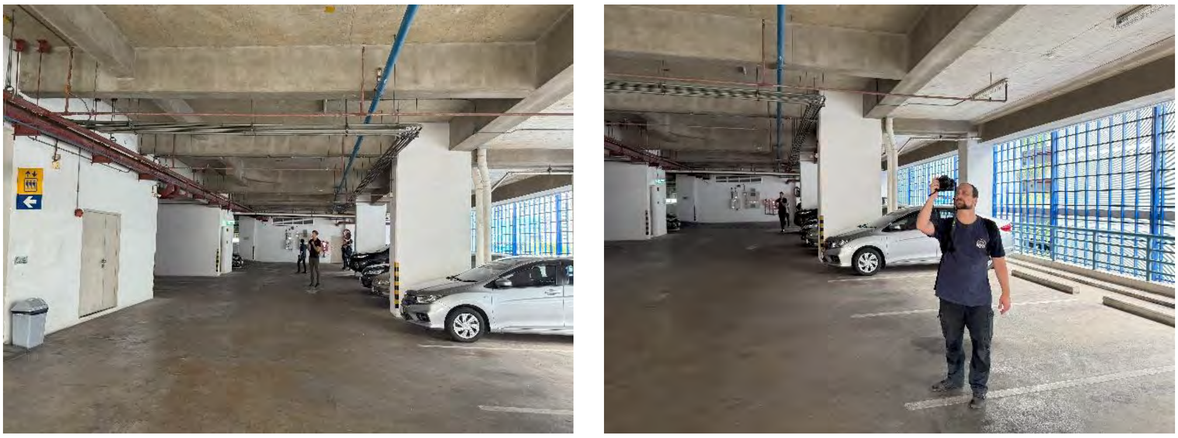
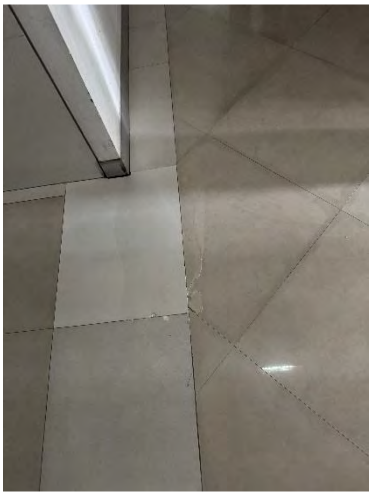


*Figure **38**: Structure of Building R3*



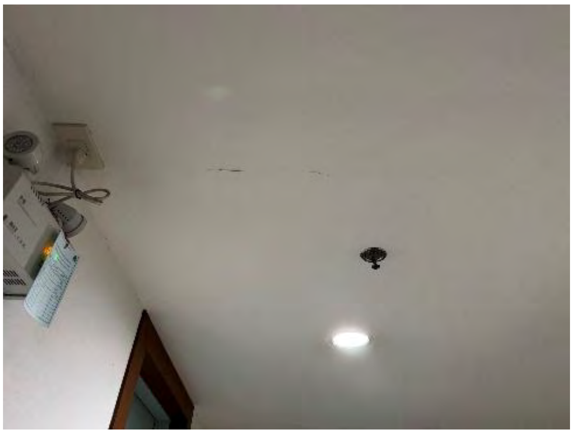
(a) Damage to floor times (floor 15)

(b) Damage to short boundary wall (Ground Floor)

*Figure **39**: Non-Structural damage to Building R3*

(c) Location of water leakage from the surge tanks of swimming pools due to joint cracks (Floor 5A)

(d) Damage to the water pipe and water shut off to the tab (Floor 6)



(e) Damage to plasterboard ceiling (Storey 15)

(f) Damage to infill wall joints

(g) Tile spalling

(h) Cracking at the interface of the structural and non-structural wall

*Figure **39** (continued): Non-Structural damage to Building R3*

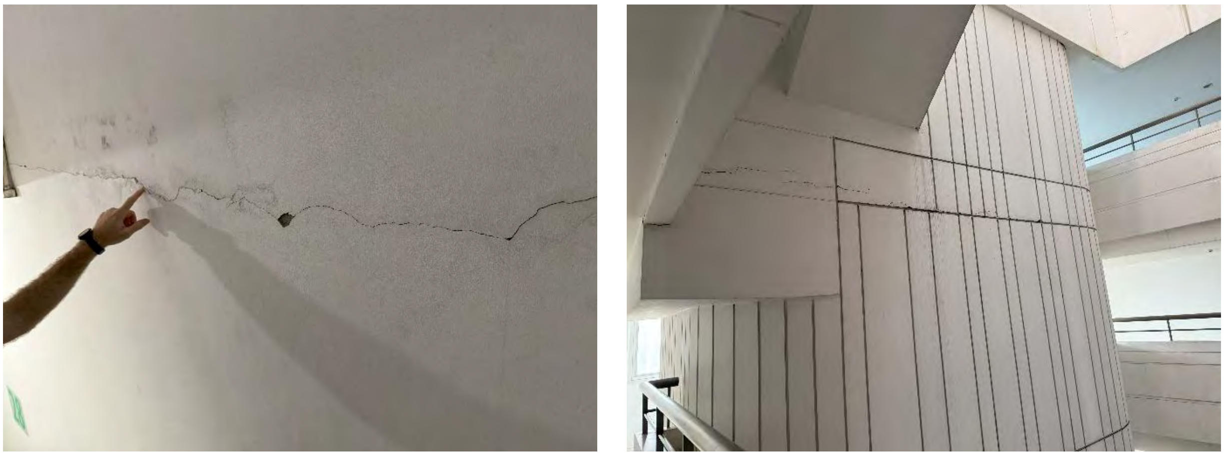


Figure 40: Damage at the interface of floor slab and non-structural walls (Level 28) in Building R3

B.6 Building R4

Building use	Mixed use
Tag	Green (no damage)
Structure	Core-wall with Outriggers
Floors	79
Year	2016
Main damage	Not observed
Likely causes	N/A
Non-structural elements typology	Infill walls and plasterboard ceilings
Main Non-Structural Damage	Minor damage to walls and ceilings reported around mid-height levels

Functionality	• Fully functional today
	• Took 2 months to regain full functionality
	• Damage to lifts –
	  o Out of 21 lifts, 7 lost function
	  o Residents live 54-73 floor. A lift was fixed on the same day to reach level 54, but occupants still had to walk upstairs. Took 10 days, one lift to go from 54 to 74.
	  o 74 floor occupant was temporarily relocated.
	• Damage to water tank – fixed the same day of the EQ.
	• Minor damage to ceilings and interior walls in around 30 apartments (>57 floors). Work is underway and did not affect occupancy.
	• Hotel area (>53 floors), everyone was evacuated. After 3 days, the hotel reopened.

Comments	Building designed for 1 in 2475 year earthquake (MCE), designed above the minimum Thai building code. The first building to use outriggers in Thailand was to increase building strength and stiffness.

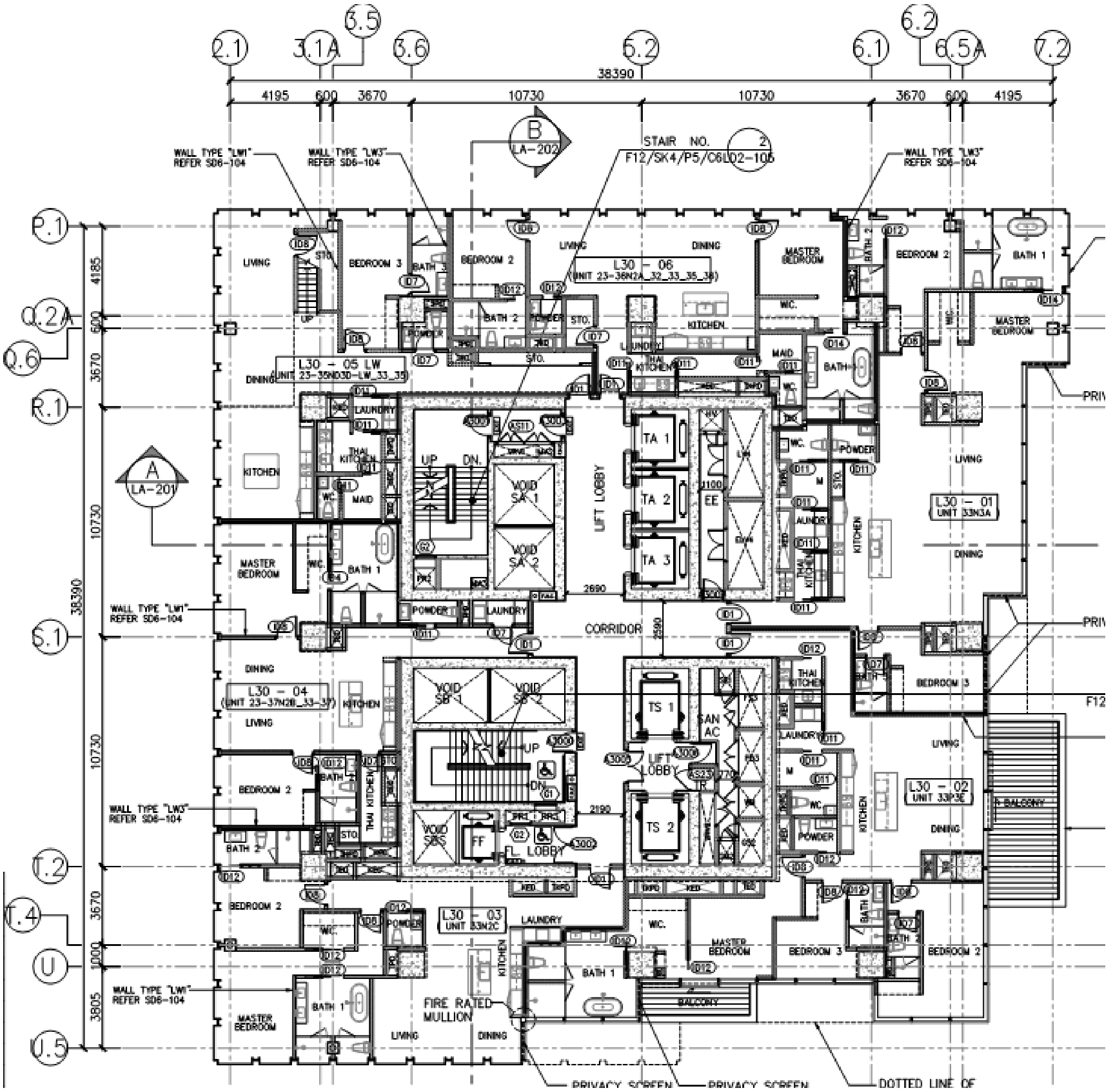


Figure 41: Building R4 plan

Figure 42: Cracking to non-structural wall in Building R4.

B.7 Building C1

Building use	Commercial
Tag	Green (no damage)
Structure	RC
Floors	47
Year	2024
Main damage	Undamaged
Likely causes	N/A
Non-structural elements typology	• Infill walls • Plasterboard ceiling
Main Non-Structural Damage	• Minor cracking to infill walls observed in upper levels
Functionality	• Fully functional today • Expansion joints caused leaks on the common podium of buildings • 41 lifts, 13 lost function, 7 have been repaired.
Comments	Commercial office with open floor plans and limited numbers of infill walls other than those around the central core walls. The building manager commented that the adjacent residential building suffered more damage because it had more masonry walls.

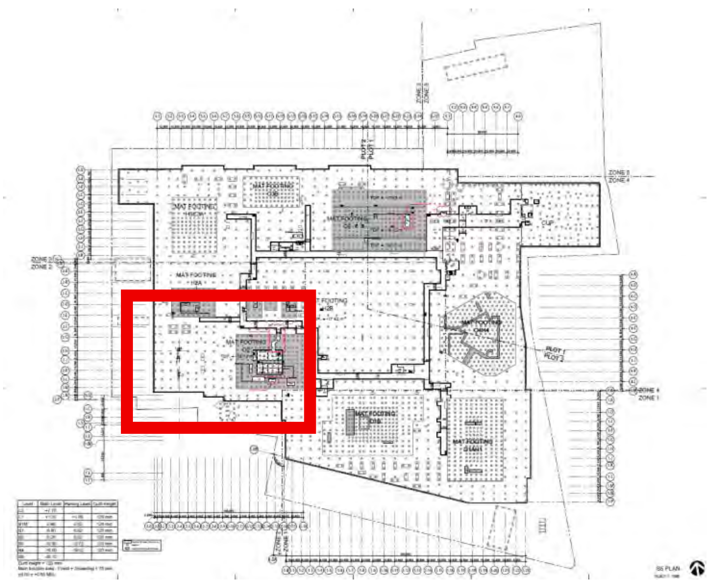


Figure 43: Site plan Building C1 Green

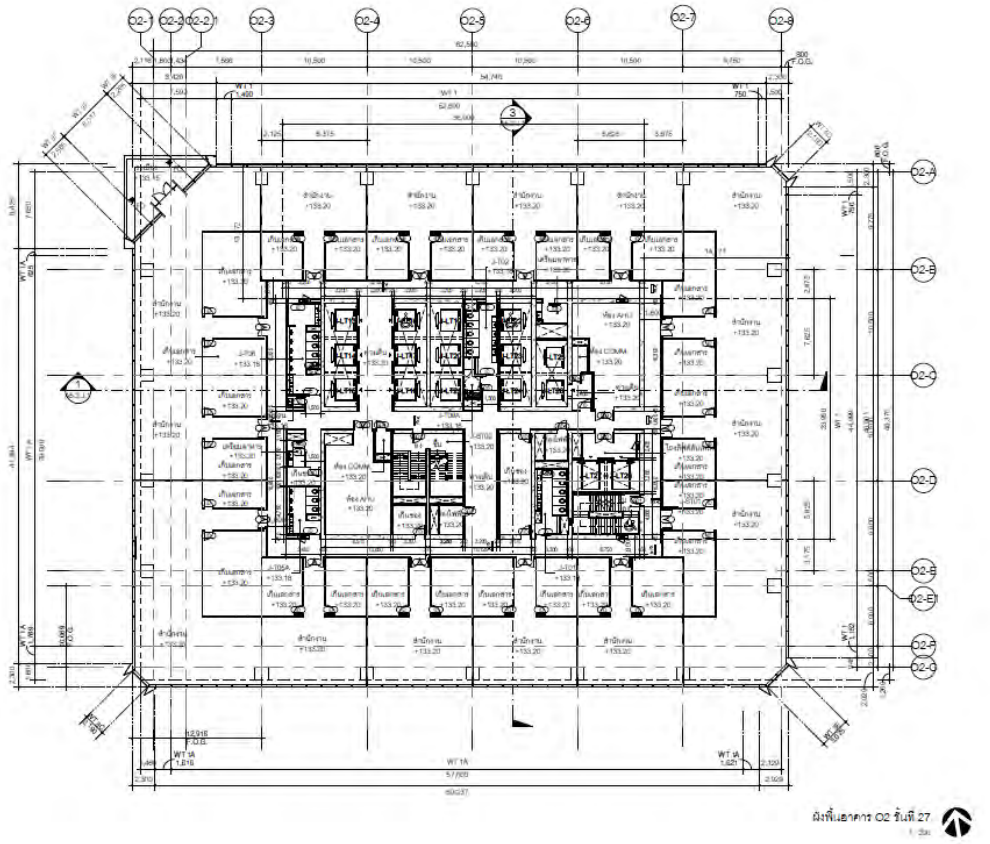


Figure 44: Building C1 plan

B.8 Building C2

Building use	Commercial
Tag	Green (no damage)
Structure	Reinforced concrete (shear walls + RC columns)
Floors	32
Year	1995 (12/12/2538 Thai calendar)
Main damage	No damages observed
Likely causes	NA
Non-structural elements typology	• Masonry clay-brick and concrete block infill walls (70 mm block + 2 x 15 mm plasters) • Plasterboard ceiling
Main Non-Structural Damage	• Falling of tiles on lift core walls • Minor damage to plasterboard and ceiling joints • Major damage to infill walls • Minor damage to external infill wall around the stair-well (Repaired) • Plaster delamination • Infill wall damage close to structural walls • Damage is in similar locations along the height.
Functionality	• Took 4 hours to regain full functionality • 13 lifts and 9 lost function – still working on it. • Damage to façade • Damage to windows or glazing – still waiting to repair some. • Damage to ceilings • Damage to lighting • Damage to interior walls (partitions) – still unrepaired • Building use totally depended on people's perception of damage. • Damage will be covered by insurance and the owners
Comments	• Prestressed unbonded (probably) slab • 30-year-old building • 32-storey building (1st to 10 stories is the car park, 2 basements) • NSE damage concentrated between 10 and 20 stories • 8 million Baht for repairing common areas • Plan size = 166 m x 22 m • Typical floor height = 3.65 m

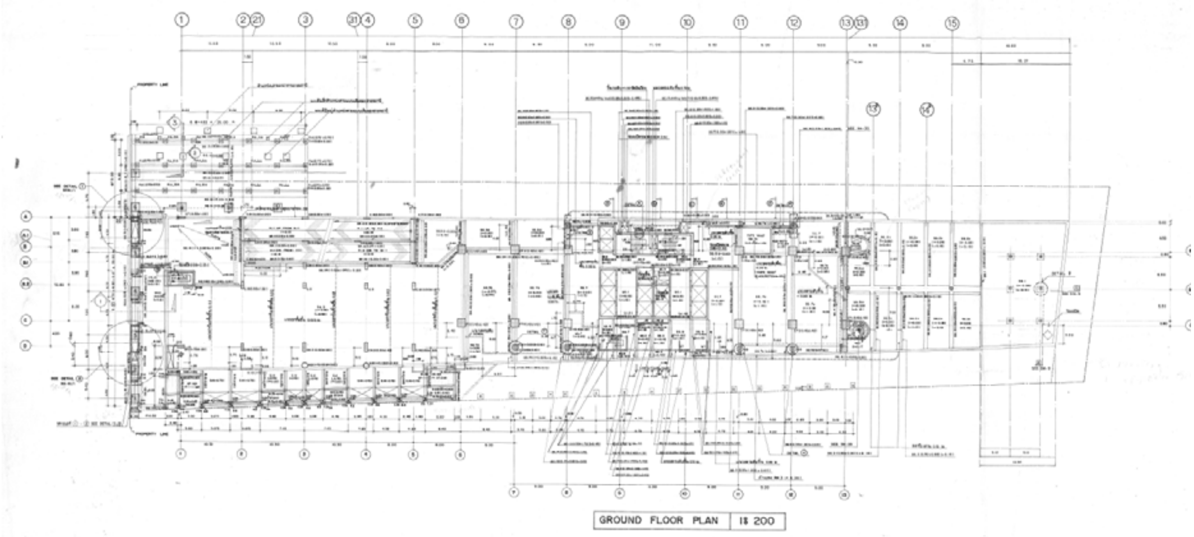
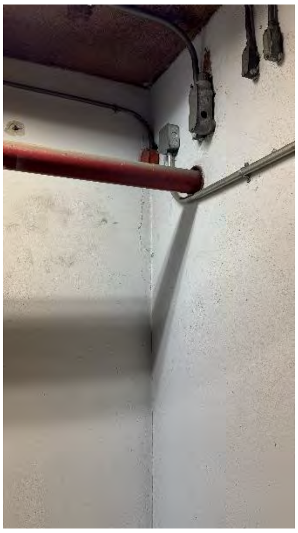


Figure 45: Building C2 plan



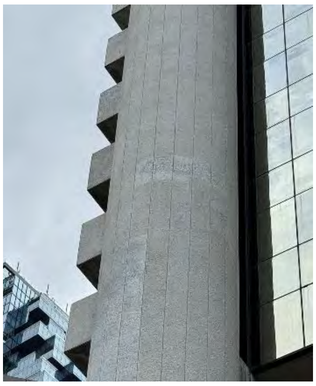
(a) Damage to infill wall (Story 34)

(b) Close-up of damage to infill wall

(c) Damage to granite tiles on core walls (Story 20)

(d) Damage to the infill wall and plasterboard ceiling joint

Figure 46: Non-structural damage to Building C2



(e) Damage to the exterior concrete block wall (plaster spalling, relatively lighter white colour) has already been repaired.

(f) Damage to infill wall (Story 17).

(g) Concrete block walls, thickness 70 mm.

(h) Cracking damage at the interface of infill and structural wall.

Figure 46 (continued): Non-structural Damage

B.9 Building C3

Building use	Commercial/ office
Tag	Green
Structure	RC concrete cores. (2) C-shaped walls and (1) I-shaped wall, (2) L-shaped walls, all with webs in the x direction. PT flat slab with beams for gravity load, RC columns on perimeter.
Floors	61
Year	2023
Main damage	Some slight cracking in the soffit of beams at the exterior of 14th floor
Likely causes	NA
Non-structural elements typology	• Concrete block and AC block infill walls • Plasterboard ceiling and acoustic grid-and-tile ceiling
Main Non-Structural Damage	• Minor to moderate damage to infill walls • Minor cracking of ceramic tiles on floors • The fridge slid about 3 m away from the original location • Falling of granite tiles • Damage to a glass door • Minor to moderate damage to architectural cladding over stairwell walls • Damage at the interface of infill and main structure
Functionality	• Fully functional at time of reconnaissance (2025/06/16) • Damage to the lift – took 45 days to regain full functionality. • Out of 33 lifts, 5 lost functionality due to cable twisting. • Damage to the ceiling and interior walls did not affect use. • Delay due to delay in getting quotes • Insurance will cover damage.

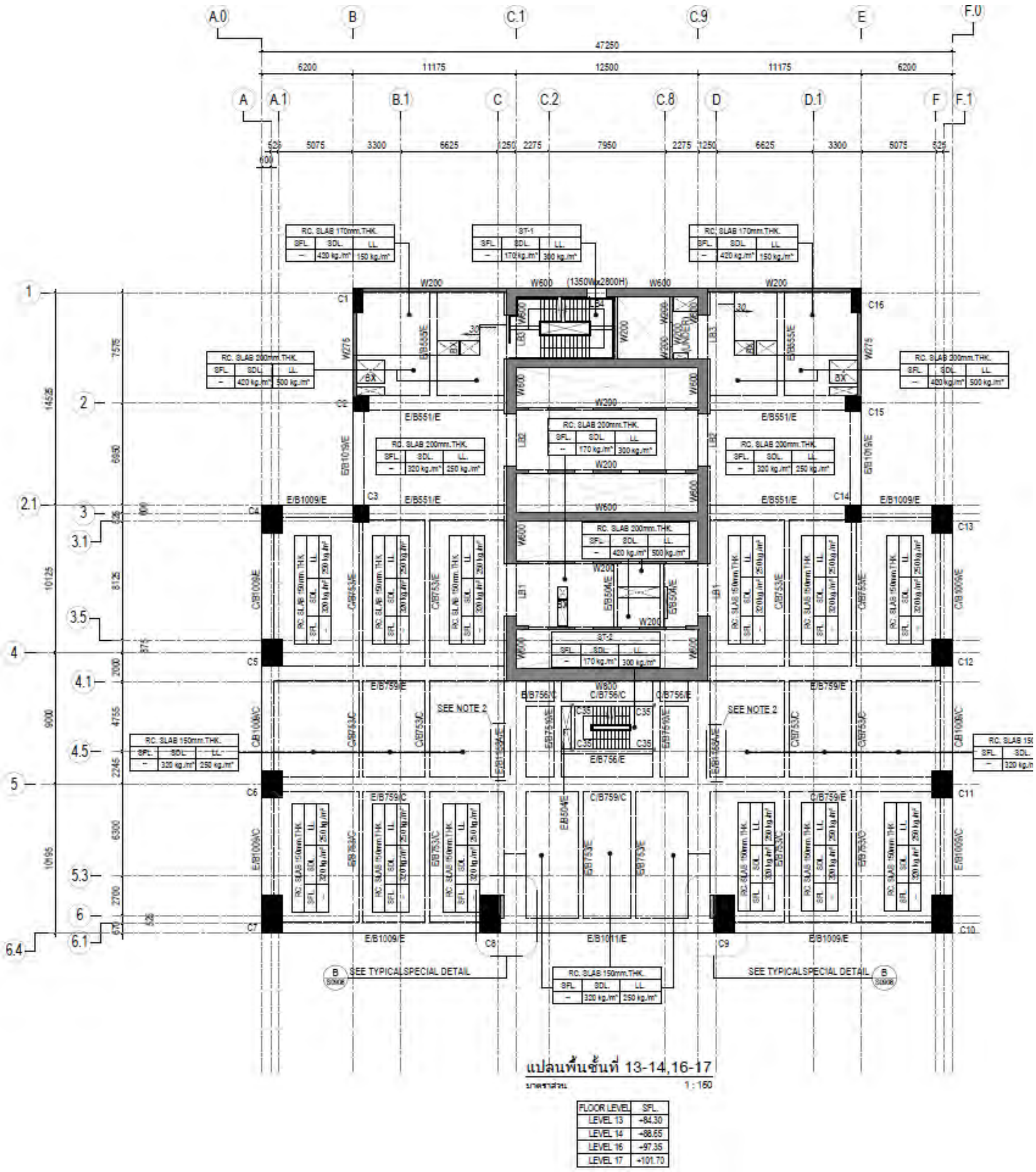
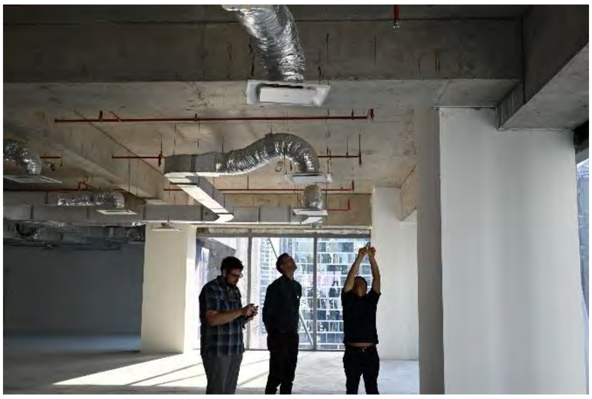
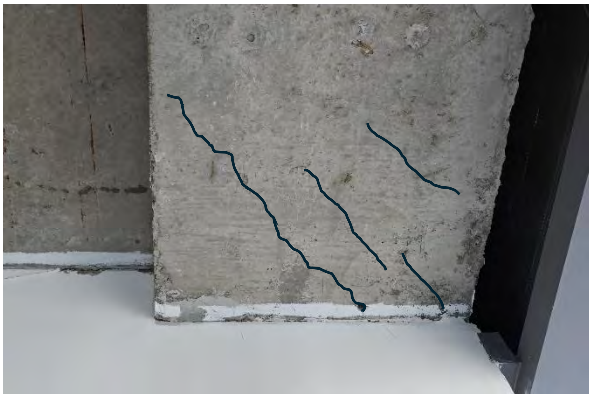


*Figure **47**: Building C3 plan (Typical story)*

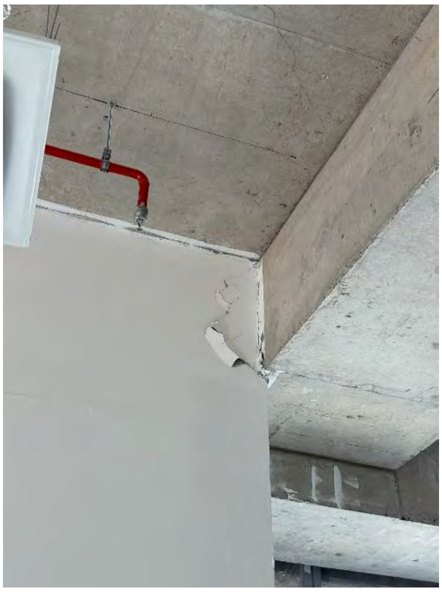


(a) Location of beam and ceiling slab damage

(b) Joint at beam and slab damage



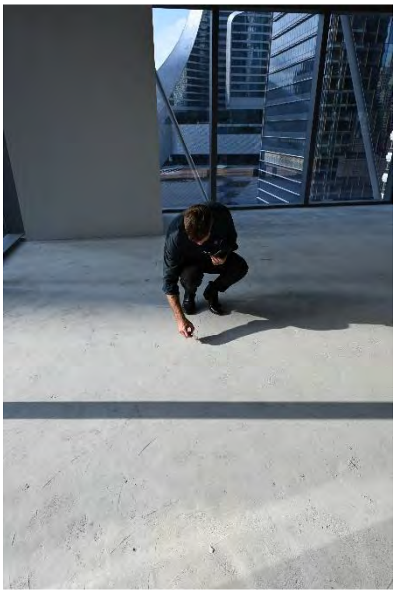
(c) Cracking on the soffit of the beam (crack width ~0.2 mm)



(d) Different view of prior images

(e) Cracking on the soffit of the floor slab by the beam exterior beam-column joint (crack width ~0.3 mm)

Figure 48: Structural damage to Building C3



(f) Crack in the floor topping

(g) Measurement of floor topping crack (crack width = 1.0 mm)

Figure 48 (continued): Structural damage to Building C3

(a) Damage to infill wall (Storey 18)

(b) Damage to infill wall (Storey 14)

(c) Damage to exterior cladding

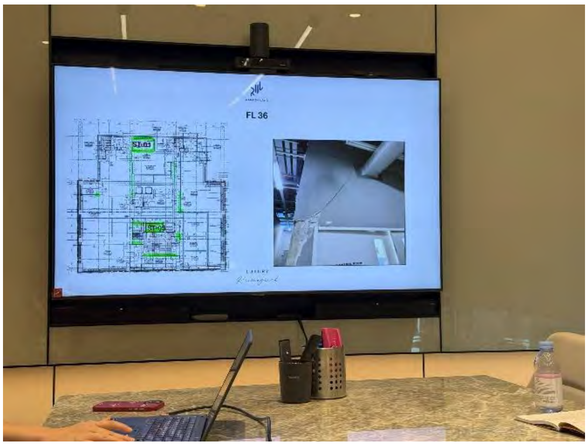
(d) Cracks on ceramic tiles (Floor 14)

Figure 48: Non-Structural damage to Building C3

(e) Falling of granite tile (Storey 18)

(f) Damage to the glass door (Storey 18)

(g) Non-functioning lifts



(h) Non-structural damage to various areas of Floor 36

Figure 48 (continued): Non-Structural damage to Building C3

B.10 Building C4

Building use	Commercial
Tag	Green (no damage)
Structure	RC + Steel (concrete-filled tubes)
Floors	30
Year	Finished construction in 2020 and started in 2018 (designed following DPT-1301/1302-2018, damping = 2.5%)
Main damage	No structural damage
Likely causes	NA
Non-structural elements typology	• Infill walls: Steel-framed plasterboard-lined walls, and steel-framed plasterboard-lined walls filled with light concrete • Suspended grid and tile ceilings and plasterboard ceilings • Raised access floor (services run both below the floor and above the ceilings) • External facade: The precast concrete cladding panels are of rocking type throughout the height of the building, and a unitised glazing system
Main Non-Structural Damage	• Minimal damage to infill walls and ceilings • Most of the damage to the infill walls occurs at the interface of the structural walls and the infill walls.
Functionality	• Damage to the ceiling, and it took one week to repair. Minor damage. • Damage to interior walls, and it took one month to repair. • The water supply lost its function, and it took 3 days to be functional. • The building was fully functional after 3 days • Delay of work was due to engineering mobilisation
Comments	• Hybrid construction: RC structure + steel (Concrete Filled Tubes (CFTs) • No outriggers present – special shear walls are designed and constructed • High R factor is used (to reduce seismic force); the main load resisting system are the shear-walls (core walls which are designed to resist 95% of the lateral load), and columns take around 5% load. • perimeter columns linked to central shear wall utilising beams with a pin joint between walls and beams • Infill panels designed for drift of 1% (ULS) and 0.3% (SLS), use of special RC shear wall • 30 Storey building with 5 stories of basement below ground level • The minor non-structural damage was fixed within 2 months after the earthquake • Dry walls used for infill panels (steel studs with cement board lining and filled with lightweight concrete)

Figure 49: Building C4 plan

C Appendix C – Ambient Vibration Results

The results of ambient vibration testing are detailed in this appendix. At visited buildings, seismometer stations, consisting of a three-component Nanometrics Trillium Compact Horizon 120s broadband seismometer and a Nanometrics Pegasus seismograph (i.e., data acquisition system) were temporarily placed at the lowest accessible (e.g., foundation or ground) and topmost accessible (e.g., top storey or roof) levels of the building and allowed to record ambient vibrations for at least 30 minutes and typically longer. At some buildings, broadband seismometer stations were moved to other levels of interest, such as the step back levels of building R2.

The Nanometrics Trillium Compact Horizon 120s broadband seismometers have a flat velocity response (sensitivity 754.3 V*s/m) from 120s (0.0083 Hz) to 200 Hz. The sampling frequency was 200 Hz, and data were continuously recorded from power-on to shutdown of the seismograph. Timing was initialised using GPS, accurate to nominally 10 μs, before temporary installation inside the buildings, enabling synchronisation of ambient vibration data.

The data were processed and analysed using Python scripts, including the hvsrpy package (Vantassel, 2021). The use of the horizontal-to-vertical spectral ratio to experimentally estimate the fundamental period (or frequency) of soil-rock vibration from ambient vibration (or microtremor) data was first described by Nakamura (1989). Over the past 30 years, the method has been used worldwide to characterise sites, evaluate the fundamental period of vibration of soil and rock above impedance contrasts (often called T_{Site} or T_0), and even estimate depth to bedrock. NZS1170.5:2004 uses a site period value of 0.6 seconds to distinguish between shallow and deep soil sites, site class C and site class D, respectively. The SESAME European project (2004)provided guidance on the clarity and reliability of microtremor HVSR spectra for estimating T_0. Molnar et al. (2022) provide an overview of the mHVSR method and best practice guidance for testing. The Python package hvsrpy provides screening tools that incorporate this best-practice guidance.

The ambient vibration data from each broadband station were processed using a uniform workflow. Each record was broken into 120-second-long windows. No filtering was applied to the data. The vertical and two horizontal components were transformed into the frequency domain using a Fast Fourier Transform. The geometric mean of the two horizontal components was divided by the vertical component, frequency-by-frequency, to produce an mHVSR spectrum for each time window. The mHVSR spectra are smoothed using the Konno and Ohmachi (1998) method with a smoothing bandwidth constant b of 40.

Assuming the data are log-normally distributed, the median and ±1 lognormal standard deviation mHVSR spectra are evaluated across all time windows for a recording. Additionally, hvsrpy automatically identifies peaks in the mHVSR spectra using the frequency-domain window-rejection algorithm (Cox *et al.*, 2020). The resulting peak-picks and mHVSR spectra were inspected by the analyst for trends. Any additional peaks were noted and searched for within analyst-specified frequency ranges and were attributed to soil or building response.

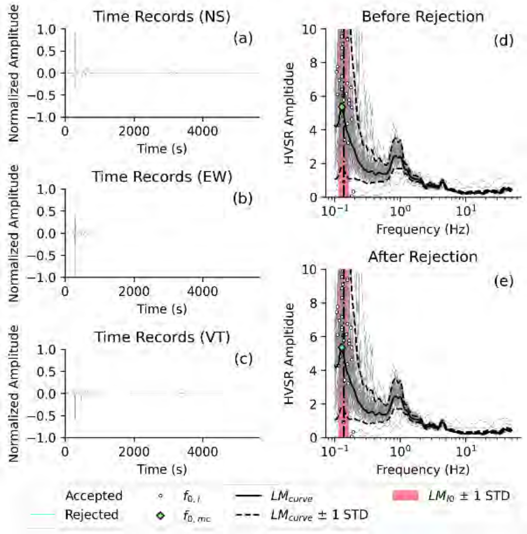
The broadband seismometer station locations and mHVSR spectra are detailed in this appendix. The results are summarised in Table 2, found in Section 3.1 of this report.

C.1 Building H1

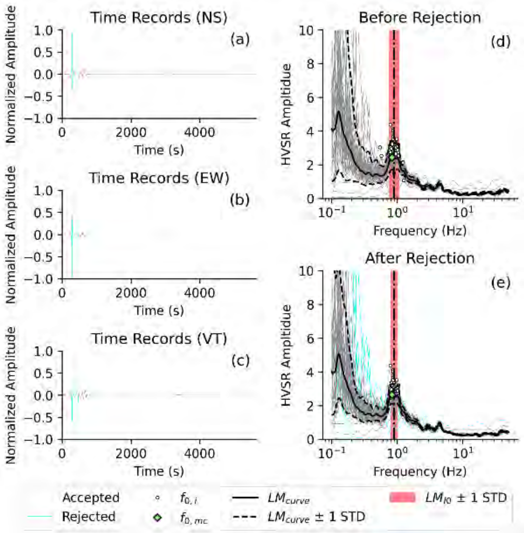
C.1.1 Floor B4 – Foundation Level

Station 01 was placed in the basement of H1, the north arrow on the instrument aligned with the damaged wall (y-direction). Note the long-period and short-period peaks.

(a) Photo of Station 01 in the basement, positive y-direction to the right of the image



(b) $T_{HV,0}$ – Median: 7.24s – σ_{ln}: 0.168

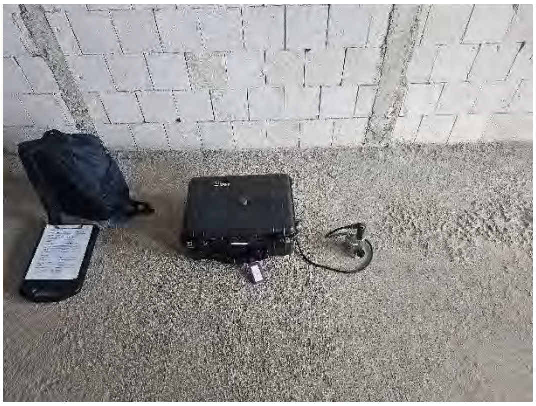


(c) $T_{HV,1}$ – Median: 1.12s – σ_{ln}: 0.104

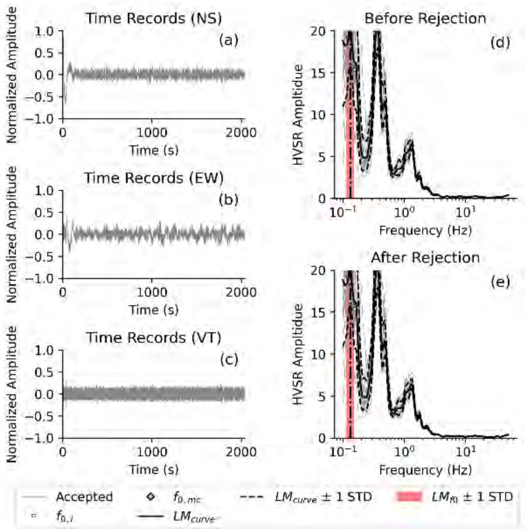
Figure 50: Ambient Vibration Testing Results – Foundation Level (B4) of Building H1

C.1.2 Floor 25 – Topmost Level

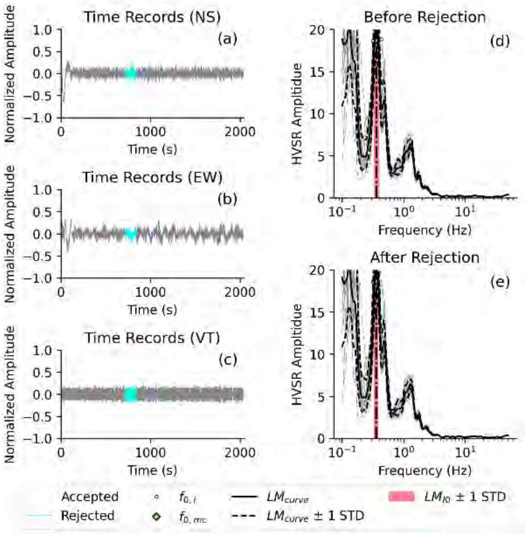
Station 02 was placed on the 25th floor of Building H1, the north arrow on the instrument aligned with the damaged wall (y-direction). THV,1 is associated with the building response. Floor 25 was unfinished and unoccupied.



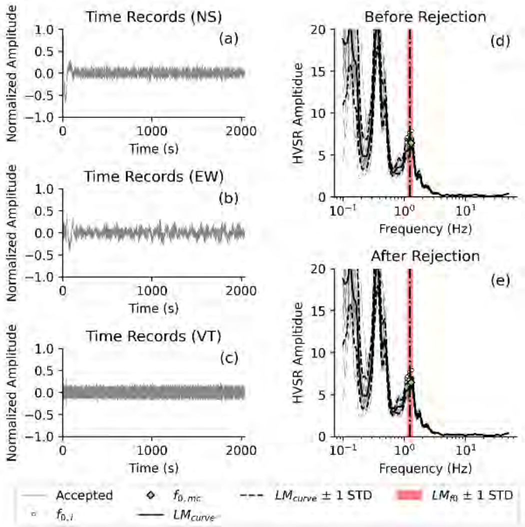
(a) Photo of Station 02 on the 25th floor, positive y-direction to the back of the image



(b) $T_{HV,0}$ – Median: 7.59s – σ_{ln}: 0.116



(c) $T_{HV,1}$ – Median: 2.80s – σ_{ln}: 0.062



(d) $T_{HV,2}$ – Median: 0.81s – σ_{ln}: 0.088

Figure 51: Ambient Vibration Testing Results – Topmost floor (Floor 25) of Building H1

C.2 Building H2

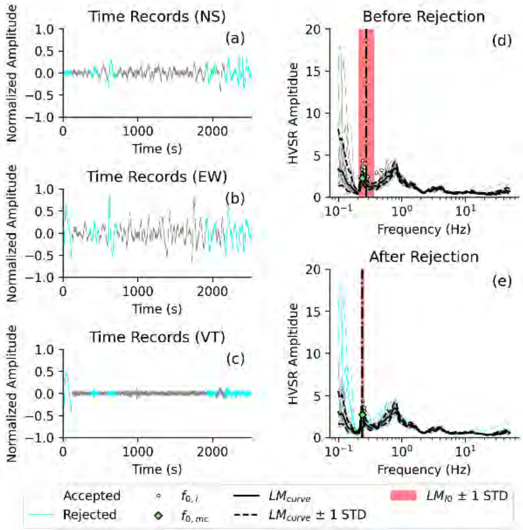
Building H2 was actively under construction, and the site was visited concurrently with Building H1, and thus not temporarily instrumented with a broadband seismometer.

C.3 Building R1

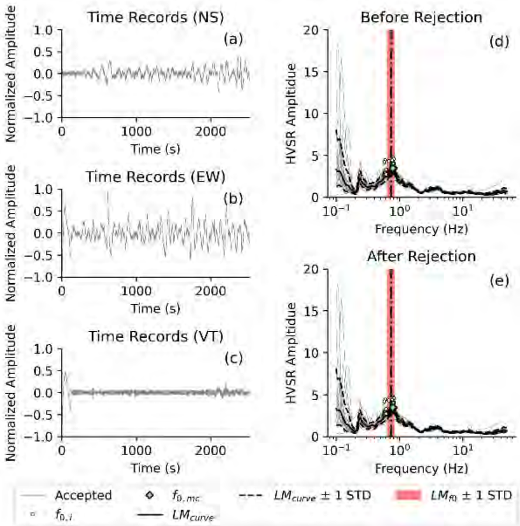
C.3.1 Ground Level

Station 01 was placed on the ground outside Building R1. The North component of the seismometer was oriented in the horizontal direction parallel to the long wall, but closest to true North.

(a) Photo of Station 01 placed on the ground outside of building R1, near rolling doors to the loading bay



(b) $T_{HV,0}$ – Median: 4.18s – σ_{ln}: 0.03

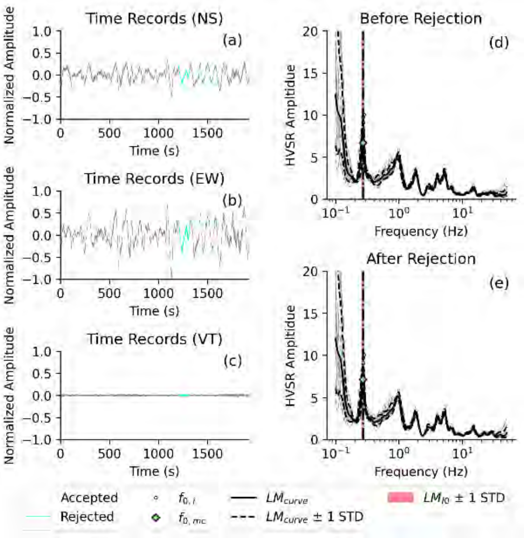


(c) $T_{HV,1}$ – Median: 1.37s – σ_{ln}: 0.12

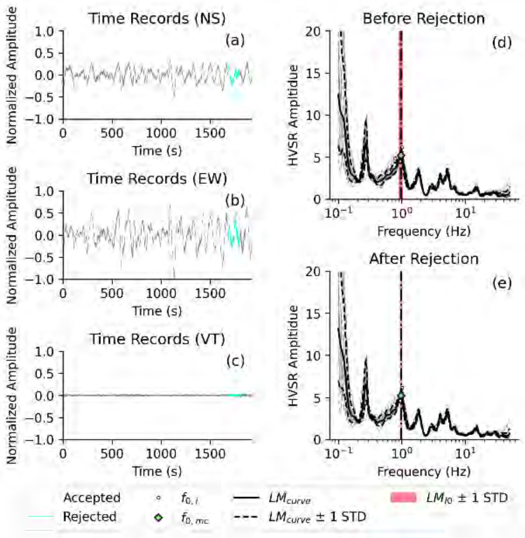
Figure 52: Ambient Vibration Testing Results – Ground Level of Building R1

C.3.2 Floor 08

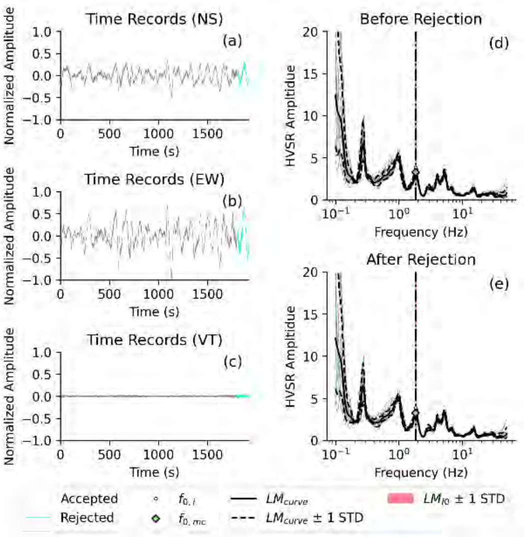
Station 02 was placed on the 8th floor of the residential building. The lift of this building was not in operation during the repair of the 9th Floor and was inaccessible during the delegation's visit. The North component of the seismometer was oriented in the horizontal direction parallel to the long wall, but closest to true North. A picture was not taken of the sensor placement.



(a) $T_{HV,0}$ – Median: 3.70s – σ_{ln}: 0.02



(b) $T_{HV,1}$ – Median: 1.03s – σ_{ln}: 0.02



(c) $T_{HV,2}$ – Median: 0.54s – σ_{ln}: 0.02

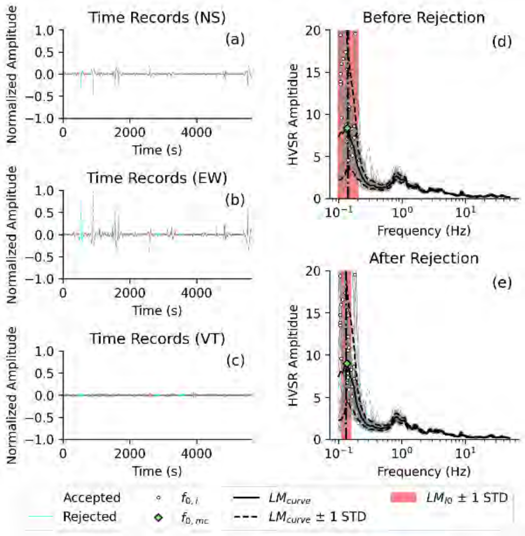
Figure 53: Ambient Vibration Testing Results – Floor 08 of Building R1

C.4 Building R2

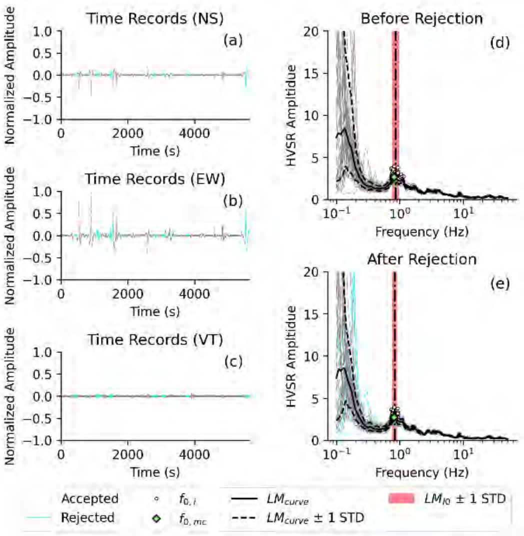
C.4.1 Floor B1 – Foundation Level

Station 01 was placed in the basement, control room of building R2. The North component of the seismometer was oriented in the horizontal direction parallel to a wall, but closest to true North.

(a) Photo of Station 01 in the basement security room.



(b) $T_{HV,0}$ – Median: 7.56s – σ_{ln}: 0.16



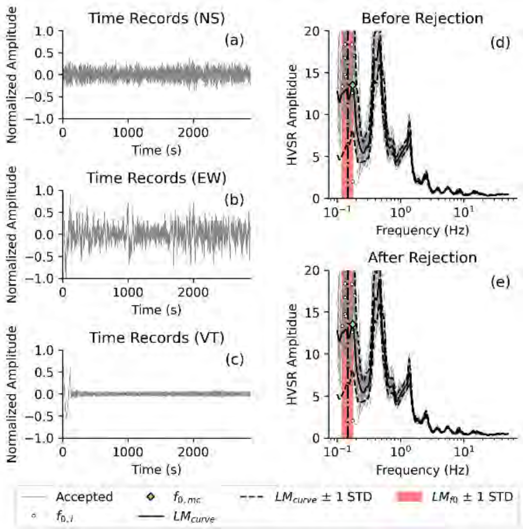
(c) $T_{HV,1}$ – Median: 1.21s – σ_{ln}: 0.07

Figure 54: Ambient Vibration Testing Results – Foundation Level (B1) of Building R2

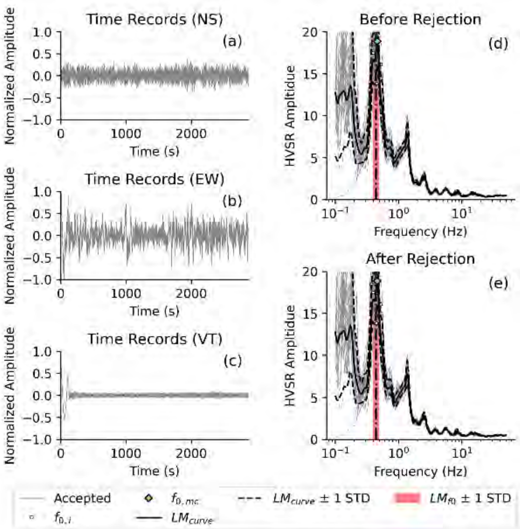
C.4.2 Floor 44 – Topmost Level

Station 02 was initially placed on the 44th floor of building R2, the top of the tower. The North component of the seismometer was oriented in the horizontal direction parallel to a wall, but closest to true North.

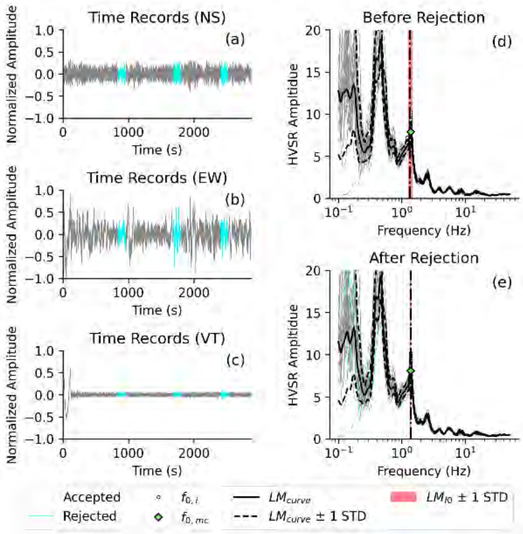
(a) Photo of Station 02 on the 44th floor



(b) $T_{HV,0}$ – Median: 6.86s – σ_{ln}: 0.19



(c) $T_{HV,1}$ – Median: 2.27s – σ_{ln}: 0.09



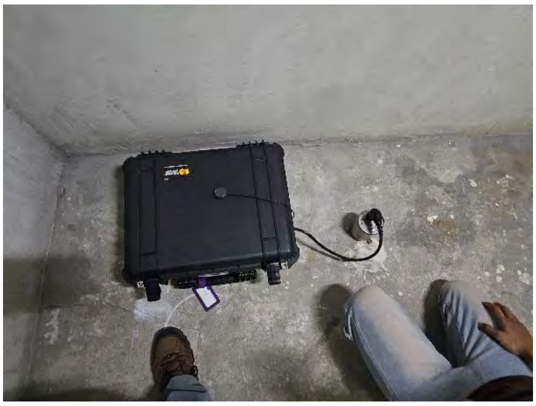
(d) $T_{HV,2}$ – Median: 0.73s – σ_{ln}: 0.00

Figure 55: Ambient Vibration Testing Results – Topmost floor (Floor 44) of Building R2

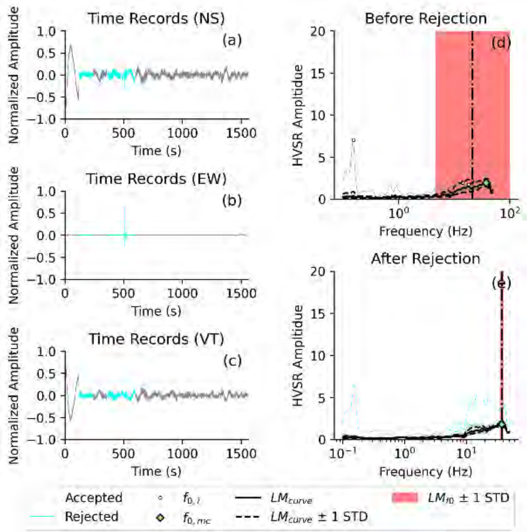
C.4.3 Floor 25 – Second Stepback

Station 02 was moved to the 25th floor of building R2 at the base of the second stepback. The sensor was located in the stairwell. The North component of the seismometer was oriented horizontally, parallel to a wall, but closest to true North.

The data indicate no clear peak in the mHVSR spectra.



(a) Photo of Station 02 on the 25th floor



(b) No clear, long period peak

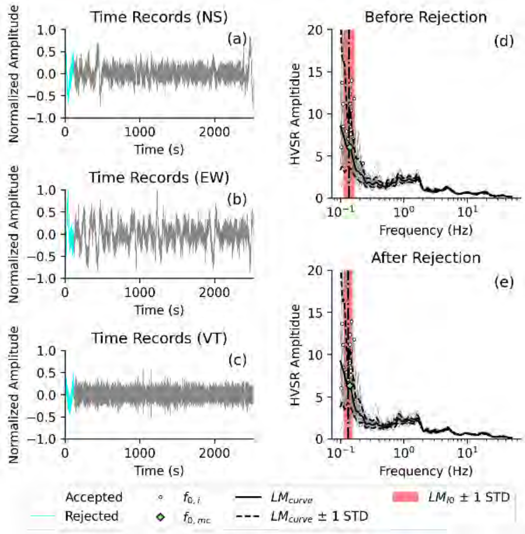
Figure 56: Ambient Vibration Testing Results – Base of second setback (Floor 25) of Building R2

C.5 Building R3

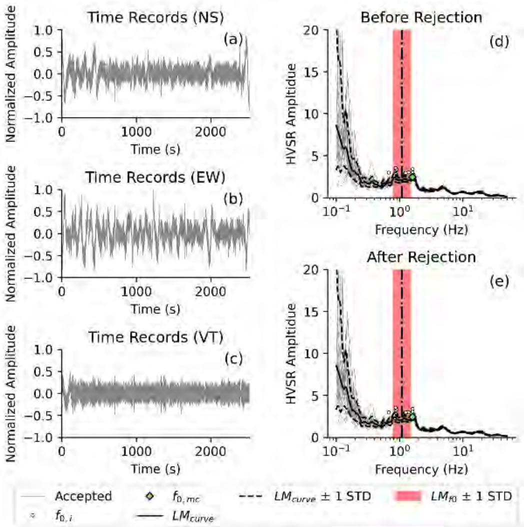
C.5.1 Ground Floor – Foundation Level

Station 01 was placed on the ground floor of building R3 under the security/reception desk. The North component of the seismometer was oriented in the horizontal direction parallel to a wall, but closest to true North.

(a) Photo of Station 01 on ground floor under security/reception desk.



(b) $T_{HV,0}$ – Median: 7.48s – σ_{ln}: 0.14



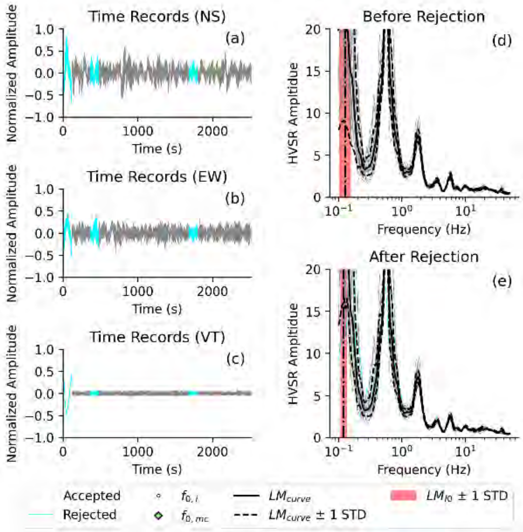
(c) $T_{HV,1}$ – Median: 0.92s – σ_{ln}: 0.31

Figure 57: Ambient Vibration Testing Results – Foundation Level (Ground Floor) of Building R3

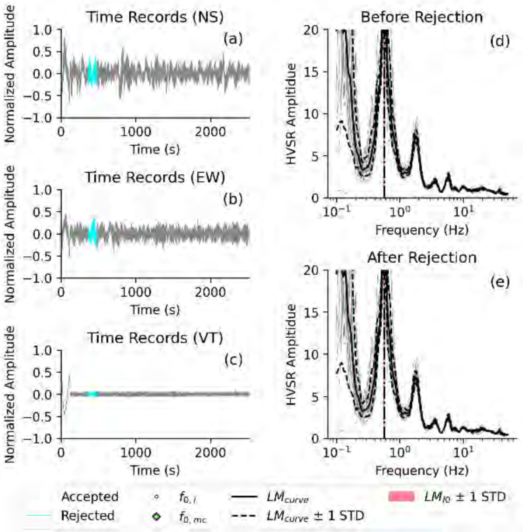
C.5.2 Floor 29 – Topmost Level

Station 02 was placed above story 29 of R3, but below the roof. The North component of the seismometer was oriented in the horizontal direction parallel to the core wall, but closest to true North.

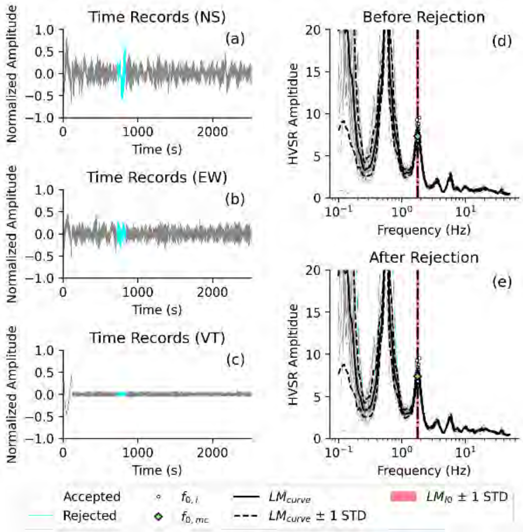
(a) Photo of Station 02 under the roof of building R3



(b) $T_{HV,0}$ – Median: 8.31s – σ_{ln}: 0.09



(c) $T_{HV,1}$ – Median: 1.76s – σ_{ln}: 0.02



(d) $T_{HV,2}$ – Median: 0.56 – σ_{ln}: 0.03

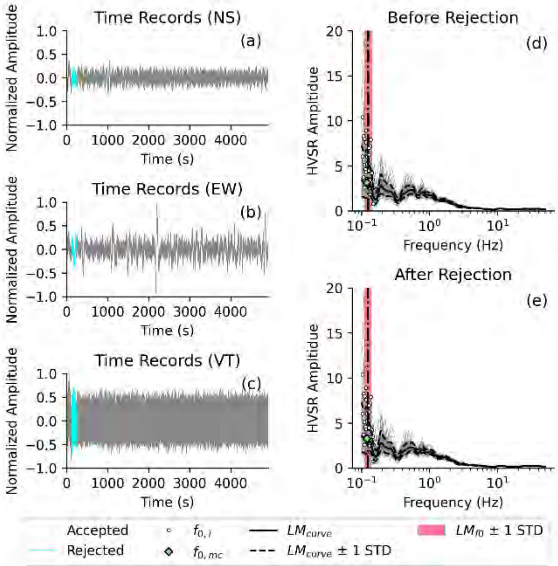
Figure 58: Ambient Vibration Testing Results – Topmost floor (Floor 29) of Building R3

C.6 Building R4

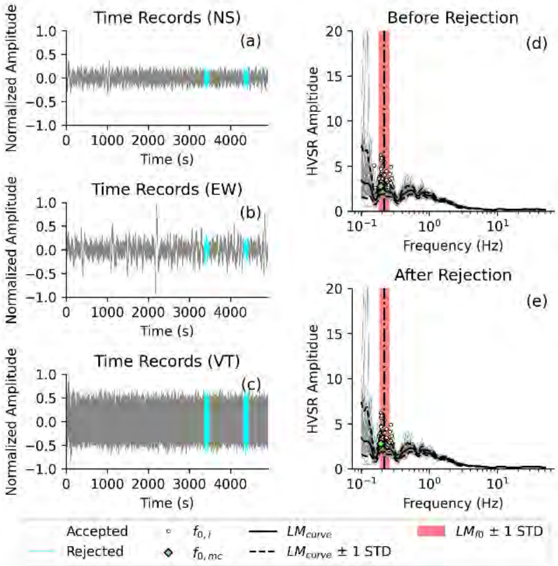
C.6.1 Floor B1 – Foundation Level

Station 01 was placed in B1 on a concrete slab. This was in a plant room. The instrument was oriented parallel to the building axis, closest to true North.

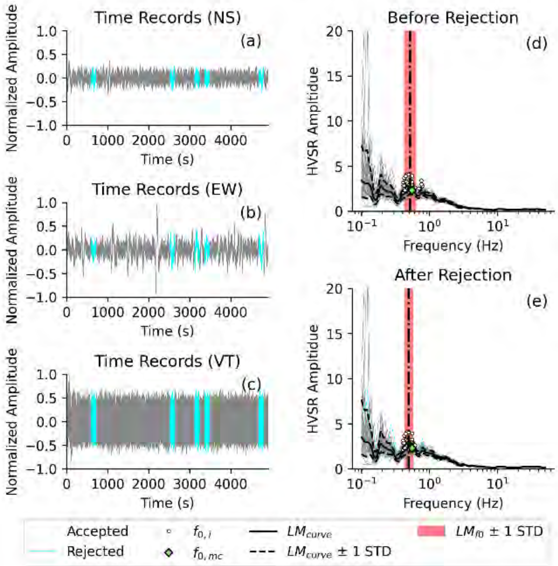
(a) *Photo of Station 01 placed on B1*



(b) *$T_{HV,0}$ – Median: 7.80s – σ_{ln}: 0.14*



(c) *$T_{HV,1}$ – Median: 4.58s – σ_{ln}: 0.14*



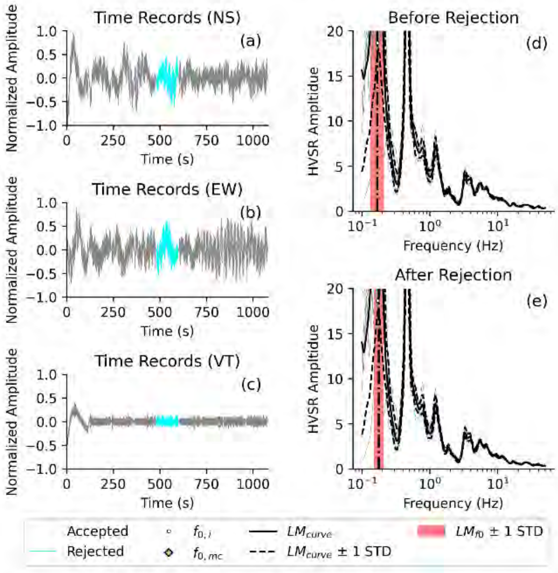
(d) *$T_{HV,2}$ – Median: 2.03s – σ_{ln}: 0.12*

Figure 59: Ambient Vibration Testing Results – Foundation Level (B1) of R4

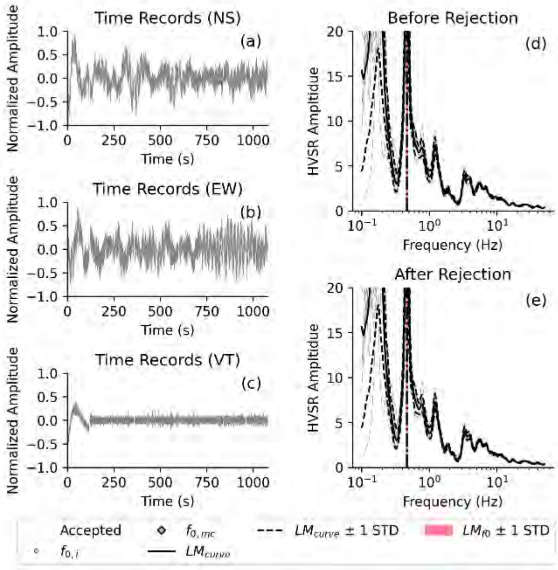
C.6.2 Floor 76 – Topmost Level

Station 02 was placed on F76 in the storage room underneath the roof observation deck. The observation deck was open to the public and would be interfered with during recording. The instrument was oriented parallel to the building axis, closest to true North.

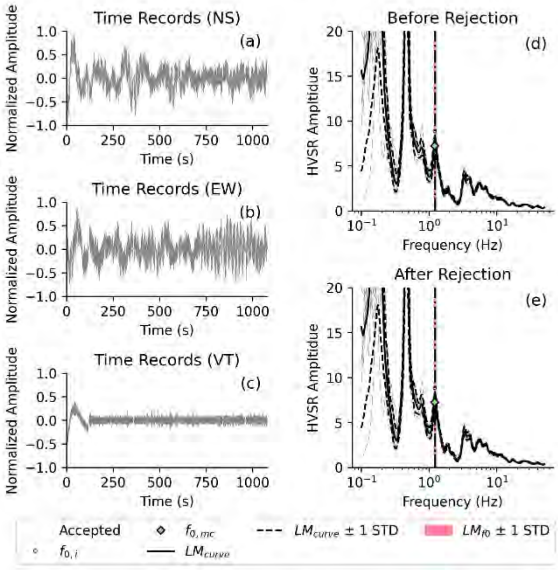
(a) Photo of Station 01 in the storage room on Floor 76



(b) $T_{HV,0}$ – Median: 5.66s – σ_{ln}: 0.14



(c) $T_{HV,1}$ – Median: 2.15 – σ_{ln}: 0.01



(d) $T_{HV,2}$ – Median: 0.82 – σ_{ln}: 0.02

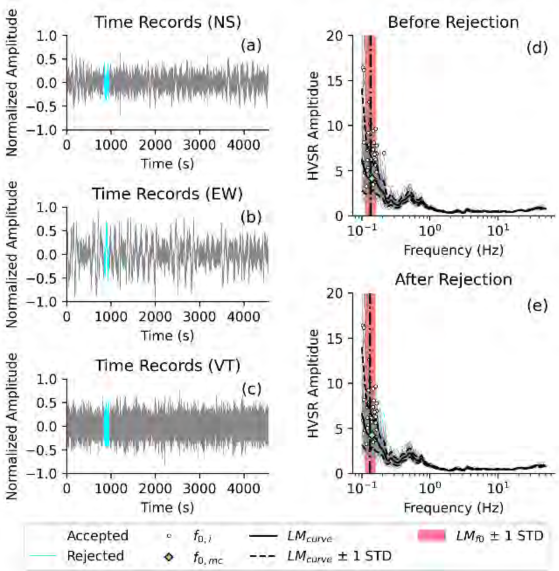
Figure 60: Ambient Vibration Testing Results – Topmost floor (Floor 76) of Building R4

C.7 Building C1

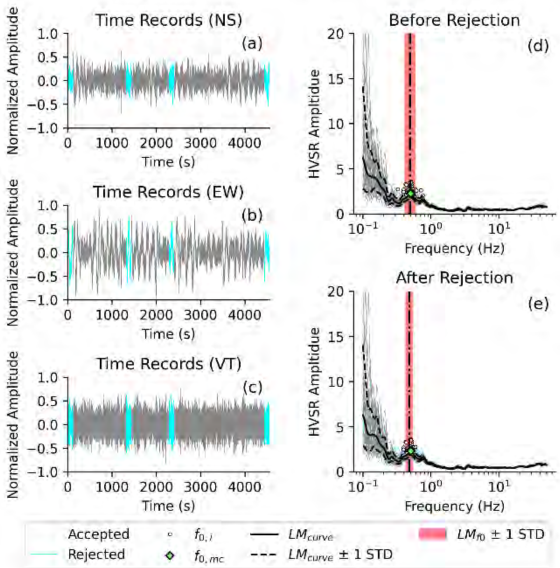
C.7.1 Floor B4 – Foundation Level

Station 04 was placed on concrete on B4 of Building C1 under tower #4. The sensor was aligned with the N-S axis of the building.

(a) *Photo of Station 04 placed on B4*



(b) *$T_{HV,0}$ – Median: 7.52s – σ_{ln}: 0.16*



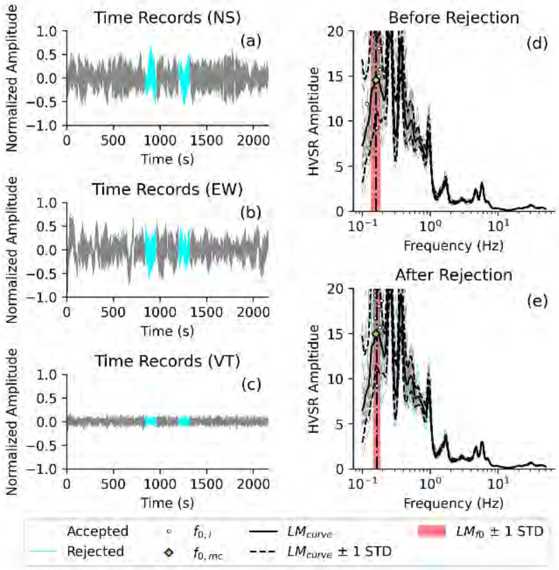
(c) *$T_{HV,1}$ – Median: 2.07s – σ_{ln}: 0.11*

Figure 61: Ambient Vibration Testing Results – Foundation Level of Building C1

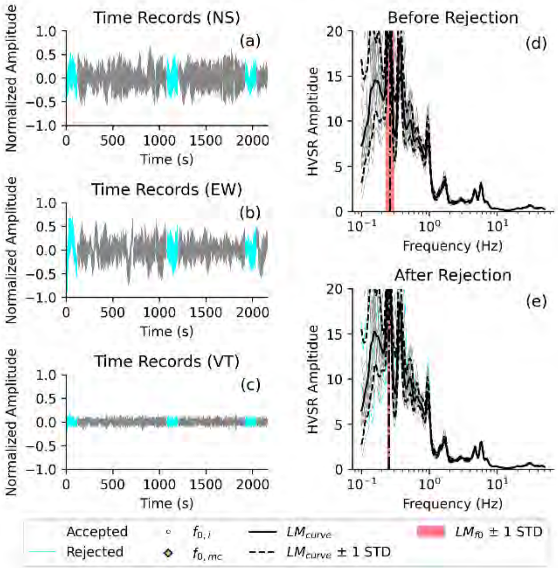
C.7.2 Floor 50 – Topmost Level

Station 05 was placed on floor 50 of Building C1 (tower #4) in the stairwell to the helipad. The sensor was aligned with the N-S axis of the building.

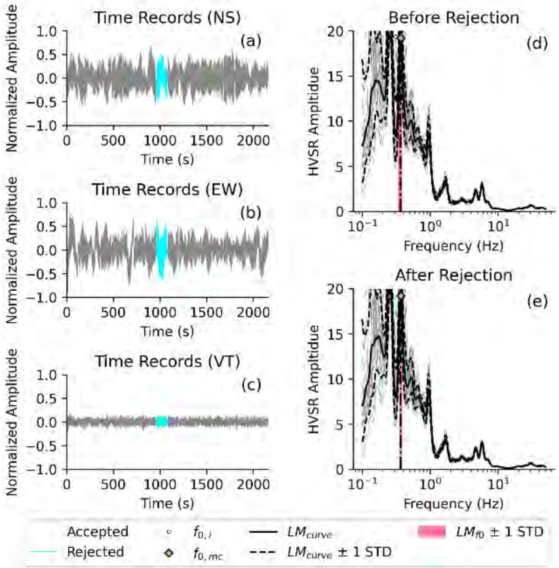
(a) Photo of Station 05 placed on F50



(b) $T_{HV,0}$ – Median: 6.31s – σ_{ln}: 0.14



(c) $T_{HV,1}$ – Median: 3.95s – σ_{ln}: 0.02



(d) $T_{HV,2}$ – Median: 2.71 – σ_{ln}: 0.02

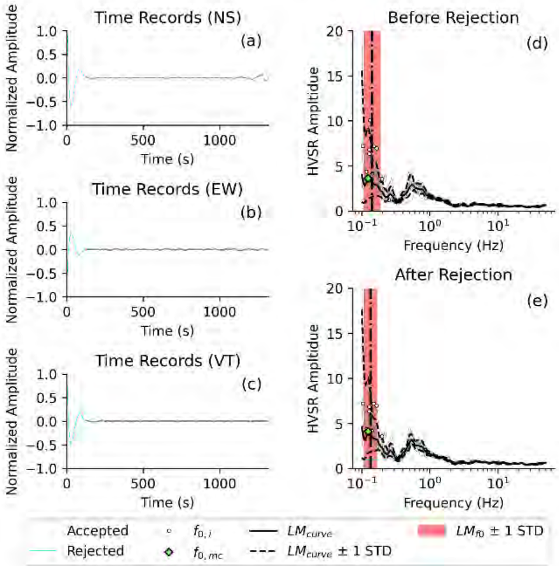
Figure 62: Ambient Vibration Testing Results – Topmost floor (Floor 50) of Building C1

C.8 Building C2

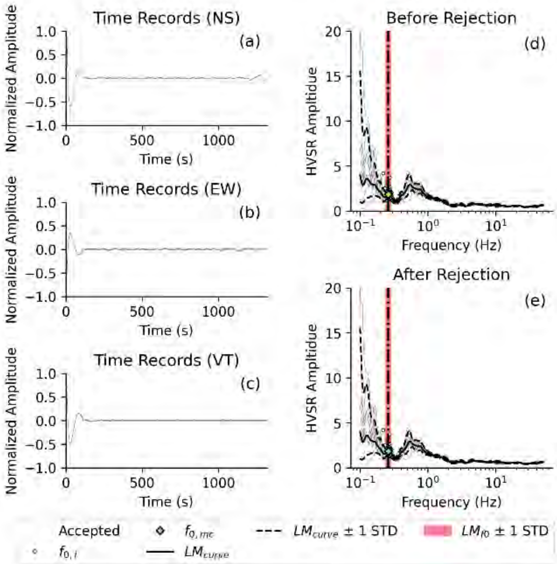
C.8.1 Floor B2 – Foundation Level

Station 01 was placed on concrete on B2 of Building C2. The sensor was aligned parallel to the N-S axis of the building. The room was a storage locker.

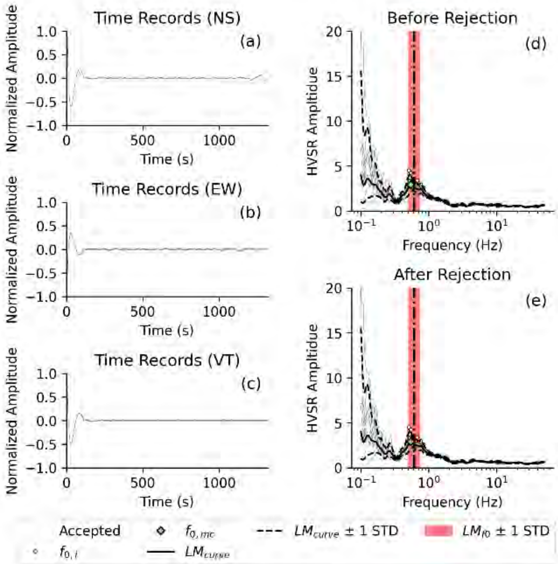
(a) Photo of Station 01 on B2 of Building C2



(b) $T_{HV,0}$ – Median: 7.46s – σ_{ln}: 0.2



(c) $T_{HV,1}$ – Median: 3.86 – σ_{ln}: 0.09



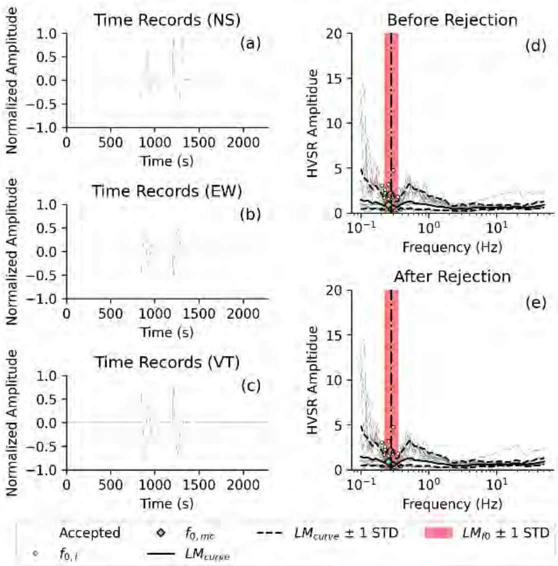
(d) $T_{HV,2}$ – Median: 1.64s – σ_{ln}: 0.19

Figure 63: Ambient Vibration Testing Results – Foundation Level of Building C2

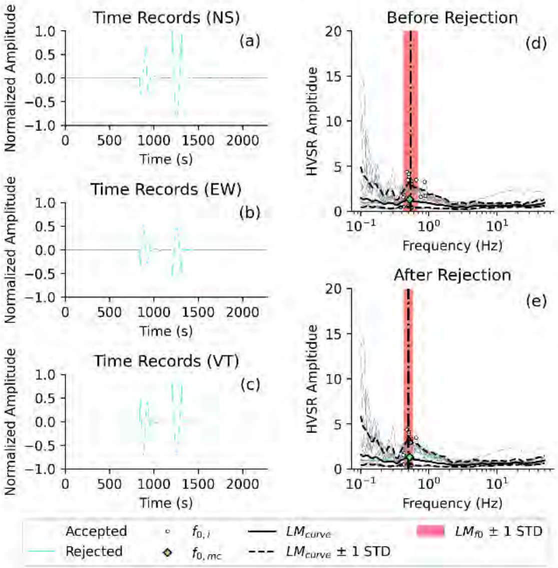
C.8.2 Floor 35 – Topmost Level

Station 02 was placed on floor 35 of Building C2. The sensor was aligned parallel to the N-S axis of the building. The room was a fan room. The data is of poor quality, potentially due to poor sensor coupling and the operation of the commercial-grade fans in the room.

(a) Photo of Station 02 placed on Floor 35



(b) $T_{HV,0}$ – Median: 3.59s – σ_{ln}: 0.21



(c) $T_{HV,1}$ – Median: 1.99s – σ_{ln}: 0.14

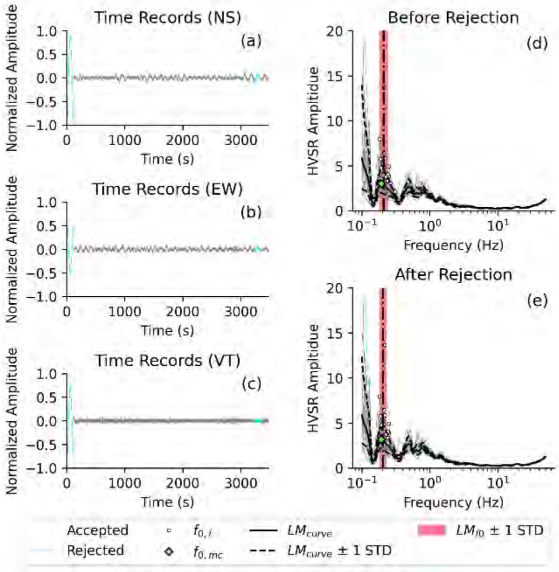
Figure 64: Ambient Vibration Testing Results – Topmost floor (Floor 35) of Building C2

C.9 Building C3

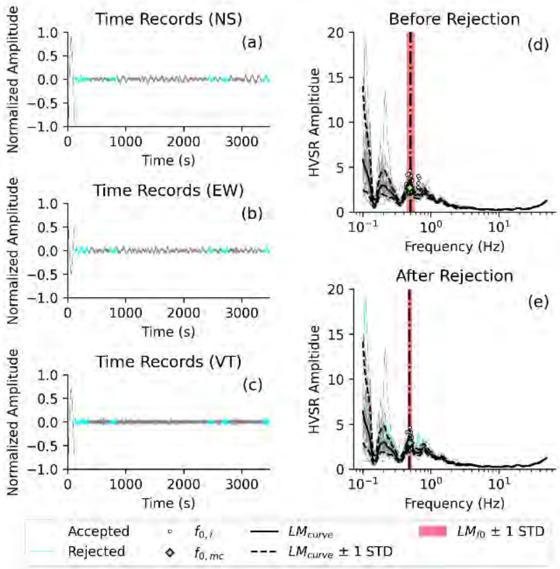
C.9.1 Floor B4 – Foundation Level

Stations 01 and 04 were placed on floor B4 of building C3, in a utility/fan room. The sensors were oriented along the building axis, near the N-S direction, with the North arrow pointing approximately to the North.

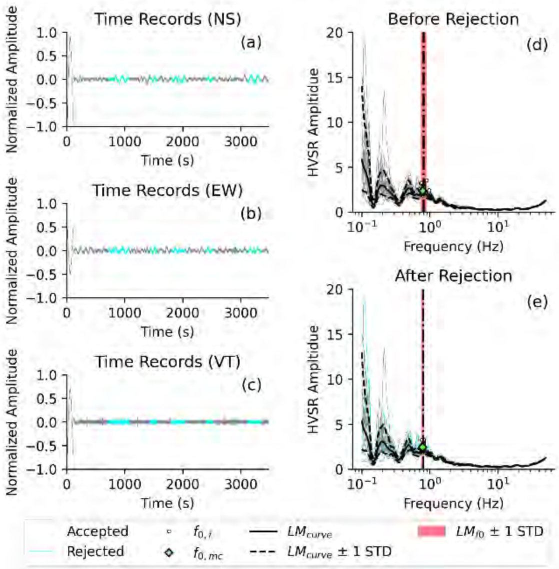
(a) Photo of Stations 01 and 04 on basement level 4 of C3



(b) $T_{HV,0}$ – Median: 4.87s – σ_{ln}: 0.1



(c) $T_{HV,1}$ – Median: 2.04s – σ_{ln}: 0.04



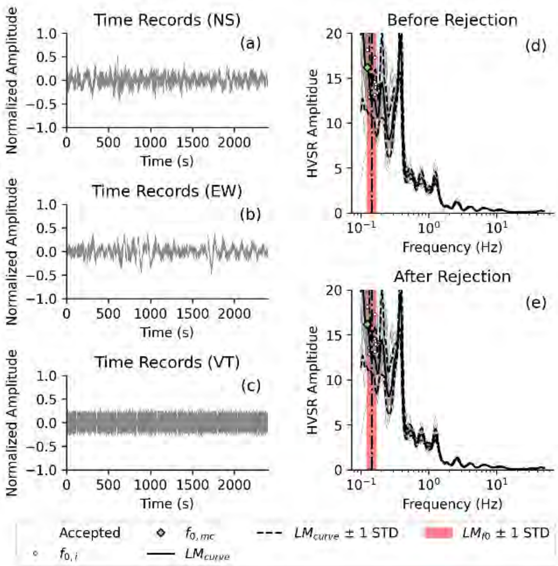
(d) $T_{HV,2}$ – Median: 1.26 – σ_{ln}: 0.02

Figure 65: Ambient Vibration Testing Results – Foundation Level of Building C3

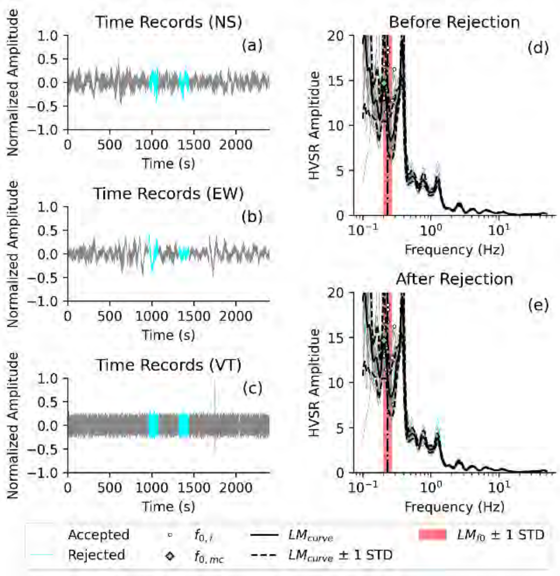
C.9.2 Floor 59 – Topmost Level

Stations 02 and 05 were placed on level 59 of building C3, in a utility room. The sensors were oriented along the building axis near the N-S direction, North arrow pointing approximately North.

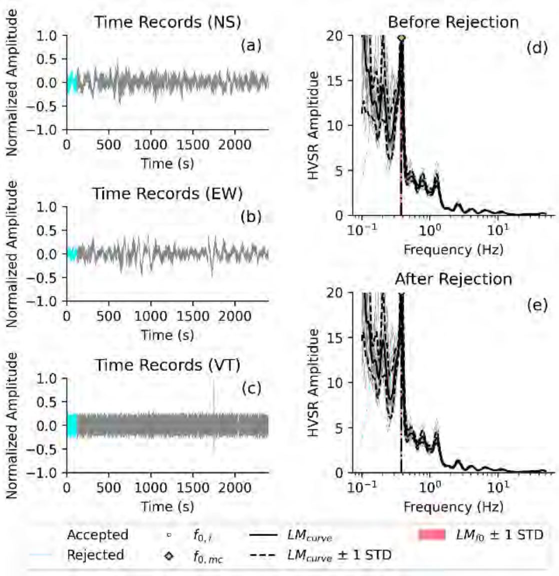
(a) Photo of Stations 02 and 05 on floor 59 of building C3



(b) $T_{HV,0}$ – Median: 6.93s – σ_{ln}: 0.14



(c) $T_{HV,1}$ – Median: 4.36s – σ_{ln}: 0.12



(d) $T_{HV,2}$ – Median: 2.63 – σ_{ln}: 0.01

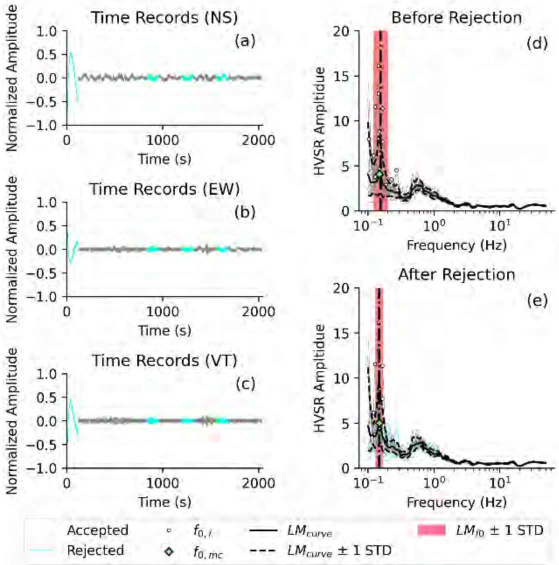
Figure 66: Ambient Vibration Testing Results – Topmost floor (Floor 59) of Building C3

C.10 Building C4

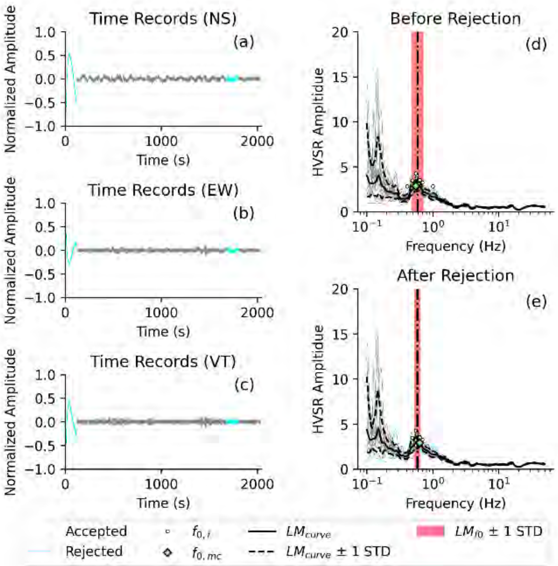
C.10.1 Floor B5 – Foundation Level

Station 01 was placed on concrete on B5 of Building C4. The sensor was oriented parallel to the N-S axis of the building, with the North arrow pointing to the North.

(a) *Photo of Station 01 on B5 of Building C4*



(b) *$T_{HV,0}$ – Median: 6.76s – σ_{ln}: 0.12*



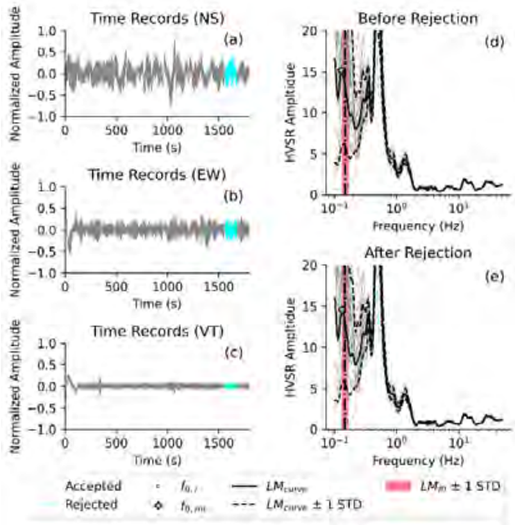
(c) *$T_{HV,1}$ – Median: 0.59s – σ_{ln}: 0.09*

Figure 67: Ambient Vibration Testing Results – Foundation Level of Building C4

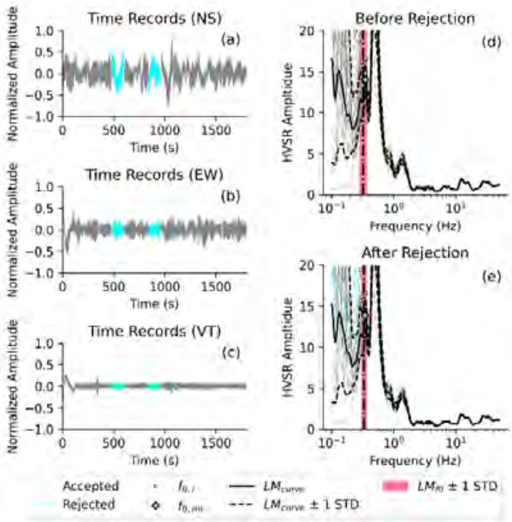
C.10.2 Floor 29 – Topmost Level

Station 02 was placed on floor 29 of Building C4. The sensor was oriented parallel to the N-S axis of the building, with the North arrow pointing to the North.

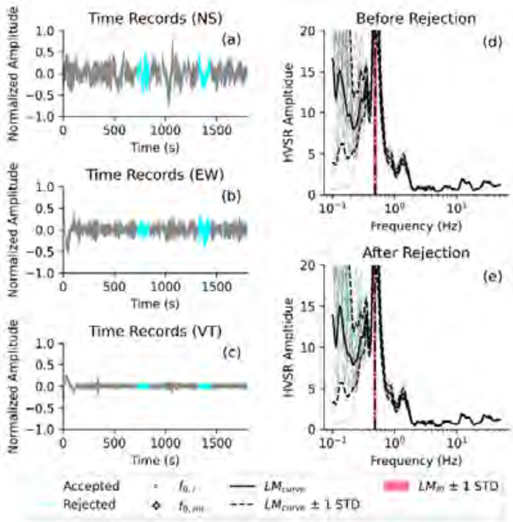
(a) Photo of Station 02 on the 29th floor of building C4



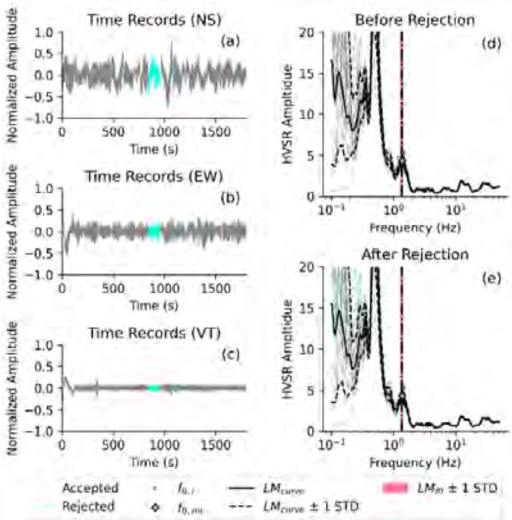
(b) $T_{HV,0}$ – Median: 6.83s – σ_{ln}: 0.09



(c) $T_{HV,1}$ – Median: 2.99s – σ_{ln}: 0.08



(d) $T_{HV,2}$ – Median: 2.08 – σ_{ln}: 0.03



(e) $T_{HV,3}$ – Median: 0.74s – σ_{ln}: 0.03

Figure 68: Ambient Vibration Testing Results – Topmost floor (Floor 29) of Building C4