

Comparative Analysis of Lifetime Costs: Base Isolated Buildings vs. Conventional Buildings

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Executive summary

Base isolation is a low-damage seismic design approach that can reduce earthquake damage, repair costs, and disruption by separating a building from strong ground shaking. Although base-isolated buildings usually cost more to construct, their lower future earthquake losses may justify the higher upfront cost. This project examined that trade-off for New Zealand buildings.

The project produced two main outputs. First, a New Zealand base-isolated building inventory database was developed. By the end of 2024, 67 base-isolated buildings had been identified, including completed buildings, buildings under construction, and buildings in design. The database records available information on building location, use, structural system, isolation system, and design characteristics. It shows that base isolation is concentrated in Wellington and Christchurch, but is increasingly being used beyond hospitals and nationally significant facilities, including for commercial and residential buildings. This database helped inform the loss study, and serves as a useful reference tool for further research into base isolated building performance.

Second, the project developed a comparative loss and lifetime-cost assessment workflow for fixed-base and base-isolated buildings. Four case-study building sets were analysed: regular 2-storey, 4-storey, and 8-storey steel buildings, and an L-shaped case-study building. Expected earthquake losses were estimated using a FEMA P-58/Pelicun-based workflow and converted into break-even periods for base isolation.

Using an initial base-isolation cost premium of approximately 3%, the simple break-even periods were approximately 7 years for the 2-storey building, 13 years for the 4-storey building, and 12 years for the L-shaped building. The 8-storey building was the main exception, with a much longer break-even period of approximately 85–96 years because the reduction in expected annual loss was small.

Overall, the project shows that base isolation can reduce expected earthquake losses and support natural hazard risk reduction, but its value should be assessed using whole-of-life thinking rather than initial construction cost alone.

Abstract

Base isolation is a low-damage seismic design strategy that can reduce structural and non-structural damage by shifting displacement demand to an isolation layer and reducing superstructure response. However, its wider adoption is influenced by the additional initial construction cost and uncertainty around whole-of-life economic benefit. This study examines the lifetime cost implications of base-isolated buildings in New Zealand through two linked components: a national base-isolated building inventory and a comparative seismic loss assessment.

The inventory identified 67 base-isolated buildings in New Zealand by the end of 2024, including completed buildings, buildings under construction, and buildings in design. The database records available information on building location, use, structural system, isolation system, and design characteristics, providing a basis for understanding current New Zealand practice and informing future performance assessments.

A comparative loss-assessment workflow was then implemented for four case-study building sets: regular 2-storey, 4-storey, and 8-storey steel moment-frame buildings, and an L-shaped case-study building. Each set included fixed-base, base-isolated, and base-isolated-with-pounding variants. Nonlinear response-history analyses using the NZS 1170.5:2004 hazard were used to obtain engineering demand parameters, which were converted into direct economic losses using a FEMA P-58/Pelicun-based workflow. Expected annual losses were then used to estimate break-even periods for base isolation.

Using an assumed base-isolation cost premium of approximately 3%, the 2-storey, 4-storey, and L-shaped buildings had simple break-even periods of approximately 7, 13, and 12 years, respectively. The 8-storey case was an outlier, with a break-even period of approximately 85–96 years because the reduction in expected annual loss was small. The results indicate that base isolation can provide substantial lifetime-cost benefits, but that these benefits are sensitive to building configuration, superstructure drift response, loss-model assumptions, and pounding-related damage mechanisms.

Keywords

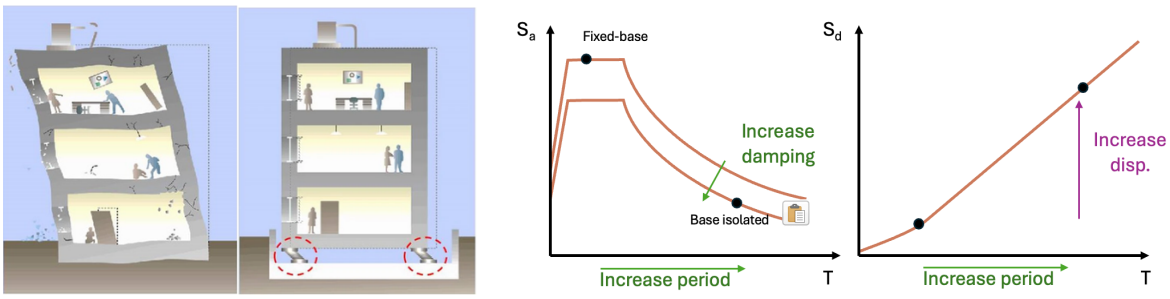
Base isolation; New Zealand buildings; building inventory; FEMA P-58; expected annual loss; life-cycle cost assessment; moat-wall pounding

Introduction

New Zealand seismic resilience context

New Zealand faces a constant risk of damaging earthquakes, and the consequences of earthquake damage extend beyond immediate life-safety concerns. Buildings that protect occupants from collapse may still sustain damage that is costly to repair, disruptive to occupants, and difficult for communities and businesses to recover from. This distinction is important for natural hazard risk reduction because the social and economic impacts of earthquakes are strongly influenced by whether buildings can be repaired, reoccupied, and returned to function within acceptable timeframes.

Conventional seismic design in New Zealand has historically relied on ductile structural response to protect life safety during strong earthquake shaking. This approach is effective in preventing collapse when properly implemented, but it accepts that structural and non-structural components may be damaged during severe shaking. In contrast, low-damage seismic design technologies seek to reduce the physical damage that occurs in the first place. One such technology is base isolation. As illustrated in Figure 1, base isolation is a concept that introduces a flexible isolation layer between the ground and the superstructure. As shown in Figure 1b, base isolation has the effect of lengthening the period of the building, thereby reducing the earthquake forces transmitted into the superstructure. This typically reduces floor accelerations and superstructure storey drifts; thus also reducing damage to structural and non-structural components. The flipside of these reductions is an increase in the building displacement demands (as shown in Figure 1c). However, these are typically concentrated at the isolation level, and in most cases will not be detrimental to the structure if adequate space around the structure is provided.



(a) (b) (c)

Figure 1: (a) Effect of base isolation on building response [1] (b) effect of base isolation on the acceleration and displacement demand spectra.

The practical benefit of base isolation is therefore not limited to improved life safety. Its broader value lies in reducing repair costs, limiting damage to contents and building services, and improving the likelihood that a building can return to function soon after an

earthquake. These outcomes are directly relevant to the Natural Hazards Commission (NHC) Toka Tū Ake because they affect future earthquake losses, community recovery, and the long-term resilience of the built environment.

Adoption barrier and decision-making problem

Despite the technical maturity of base isolation and its successful performance in major earthquakes internationally, adoption in New Zealand has remained relatively limited compared with the total number of buildings constructed in seismically active regions. One reason is that base isolation is simply more expensive. Additional cost may arise from the isolation devices, the isolation plane, moat or rattle-space requirements, foundation and diaphragm detailing, and the additional design effort required provided it still remains an alternative solution under the Building Code [2].

This creates a decision-making problem for building owners, developers, engineers, insurers, and public agencies. The costs of base isolation are very much tangible during design and construction, while the benefits are realised probabilistically over the life of the building through reduced damage, reduced downtime, and improved post-earthquake functionality. For this reason, decisions about base isolation cannot be assessed only through initial construction cost. They require a whole-of-life view that considers the likelihood and consequences of future earthquake damage.

The motivation of this project is therefore to help express the benefits of base isolation in terms that are useful for practical decision-making. This includes direct repair costs, potential downtime-related costs, and the broader implications of reduced damage for building owners, occupants, communities, and NHC.

Research gap

Previous New Zealand studies have shown that base-isolated buildings can offer significant life-cycle cost benefits relative to conventional buildings. However, several gaps remain before these benefits can be translated into robust and generalisable decision-support information.

First, there has been limited consolidated information on the existing base-isolated building stock in New Zealand. Without a structured inventory, it is difficult to identify what types of buildings are being isolated where they are located, which isolation systems are being used, what design parameters are typical, and how design practice has evolved over time. This is particularly important because base isolation is not explicitly covered in NZS 1170.5:2004 [3] in the same way that it is treated in some overseas standards (e.g., ASCE 7 [4]) . In New Zealand, base-isolated buildings are generally designed as alternative

solutions, with reference to project-specific design approaches and the draft NZSEE Seismic Isolation Guideline [5].

Second, the seismic hazard context has changed. The release of the 2022 National Seismic Hazard Model and the associated development of TS 1170.5 have raised questions about how existing base-isolated buildings would perform under revised hazard estimates. For base-isolated buildings, this issue is especially important because increased displacement demand at the isolation layer may affect moat clearance, isolator displacement capacity, and the potential for impact with surrounding moat walls.

Third, there is a need for practical workflows that connect structural response analysis to economic loss estimates. The value of base isolation is often described qualitatively, but decisions by owners, engineers, and risk managers require quantitative estimates of repair cost, downtime, and whole-of-life consequences. This project contributes to that need by developing two linked components: a New Zealand base-isolated building inventory database, and a comparative framework for assessing the lifetime costs of base-isolated and conventional buildings.

Project purpose and objectives

The overall purpose of this project is to improve the evidence available for evaluating the economic value of base-isolated buildings in New Zealand. The project focuses on the relationship between building design choices, seismic response, expected damage, repair cost, and long-term cost implications.

The specific objectives addressed in this report are to:

1. develop a structured database of base-isolated buildings in New Zealand;

2. identify trends in the location, use, structural form, isolation system, and key design parameters of the current base-isolated building stock;

3. use the database to inform the selection and interpretation of representative case-study buildings for comparative loss assessment;

4. develop a workflow for comparing conventional and base-isolated buildings using structural response analysis and loss-estimation methods; and

5. develop and implement a modelling approach to quantify the ‘break-even’ period for choosing a base isolated structure over a fixed-base one.

Scope of this report

This report presents the completed database component of the project and introduces the comparative loss and lifetime-cost assessment framework. The database component summarises the base-isolated building inventory in New Zealand and identifies major trends relevant to future performance and loss assessment. The comparative analysis component, described later in the report, considers selected case-study buildings and evaluates their seismic response and expected losses using numerical modelling and loss-estimation methods.

The report does not provide a complete national loss model for all base-isolated buildings in New Zealand. Instead, it provides an evidence base and case-study methodology that can support future expansion. The findings should therefore be interpreted as a combination of: database-supported observations about the current building stock; case-study evidence from selected archetype buildings; and methodological outputs that establish a basis for more comprehensive future assessments.

The focus is given to the current building inventory, representative isolation-system characteristics, and the comparative performance and cost implications of base-isolated and conventional building systems. Where isolation displacement, moat clearance, or pounding risk is discussed, it is treated as a performance and loss-assessment issue rather than as a device-optimisation study.

Report structure

The report is organised as follows. The next section presents the New Zealand base-isolated building inventory database, including its purpose, data sources, parameters, limitations, and key findings. The following sections then describe the methodology for comparative structural response and loss assessment, present the results of that assessment, and discuss the implications for design practice and future research.

Base-isolated building inventory database

Purpose of the database

A key output of this project was the development of a database summarising the current inventory of base-isolated buildings in New Zealand. The purpose of the database was not simply to produce a list of buildings, but to create a structured evidence base that could support future performance assessment, loss modelling, design guidance, and policy-relevant discussion.

The database serves three main functions within this project. First, it provides a stocktake of where and how base isolation has been adopted in New Zealand. Second, it identifies typical design and structural characteristics that can inform the selection of representative buildings for further analysis. Third, it highlights potential issues for future assessment, including variation in design approaches, moat clearance, isolation period, and the implications of changing seismic hazard estimates.

This database component is particularly important because base isolation practice in New Zealand has developed through project-specific alternative solutions rather than through a single codified design pathway. A consolidated database therefore provides a useful way to understand current practice and identify where further assessment or guidance may be needed.

Database scope and data sources

The database focused on buildings in New Zealand that use base-isolation systems, including elastomeric rubber bearings, curved-surface sliders, flat sliders, or combinations of these devices. The database was developed from technical literature, structural drawings, design feature reports, information provided by structural engineers, and council property files.

For each building, available information was reviewed and entered into a spreadsheet database. The recorded parameters covered four broad categories:

1. general building information, including location, use, status, construction year, and whether the project was a new build or retrofit;

2. design characteristics, including importance level, soil class, design standard information, moat or rattle-space provision, and key design parameters where available;

3. isolator properties, including isolator type, supplier, arrangement, and displacement-related information where available; and

4. superstructure characteristics, including number of storeys, structural material, lateral load-resisting system, floor system, foundation type, footprint, and isolation-plane arrangement.

The database was intended to support trend analysis rather than replace detailed building-specific assessment. Not all parameters were available for every building, and some information may reflect design-stage documentation rather than final as-built conditions. For this reason, missing or uncertain data were retained transparently rather than inferred where the source information was insufficient.

Overview of the identified inventory

By the end of 2024, 67 base-isolated buildings had been identified in New Zealand. The full database is provided in Appendix 5. Of the 67 buildings, 54 were completed, 7 were under construction, and 6 were in the design stage (Figure 2). The inventory shows that base isolation has become more common in New Zealand since the Canterbury earthquake sequence, with further growth following subsequent large earthquakes and increasing interest in low-damage design.

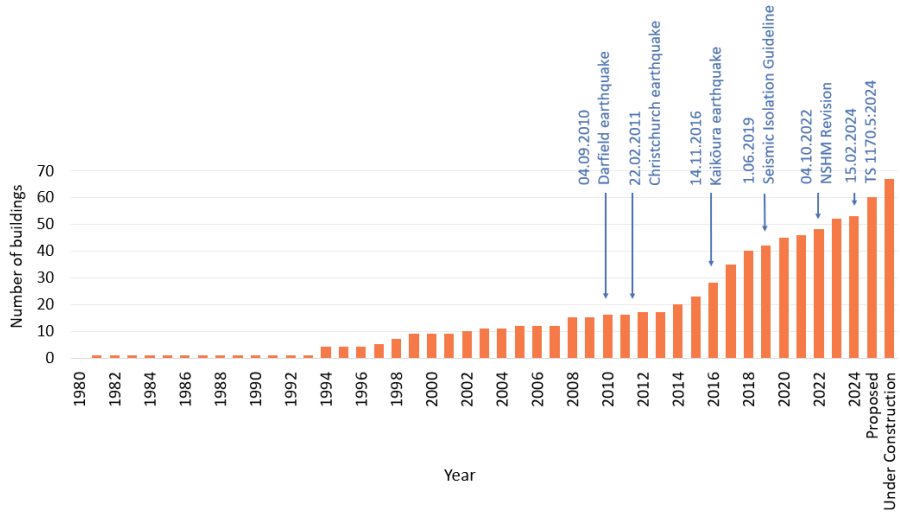


Figure 2: Cumulative number of base isolated buildings in New Zealand.

The identified inventory is strongly concentrated in higher-seismicity regions. As shown in Figure 3, Wellington and Christchurch together account for 73% of the identified base-isolated building stock. This distribution is consistent with the concentration of seismic hazard, the experience of recent damaging earthquakes, and subsequently higher value placed on post-earthquake functionality in these regions. At the same time, the presence of base-isolated buildings outside Wellington and Christchurch suggests that the perceived value of base isolation is not limited to the highest-hazard urban centres.

Most identified projects were new buildings rather than retrofits. New construction accounted for 81% of the database, while retrofits accounted for 19%. This indicates that base isolation is most commonly adopted when it can be integrated into the design from the outset, although the retrofit examples remain important because they demonstrate the potential use of base isolation for existing buildings where conventional strengthening may be difficult, intrusive, or not well-aligned with heritage and functionality objectives.

As shown in Figure 3, the database highlighted that base isolation in New Zealand is no longer confined to hospitals or nationally significant public facilities. Commercial buildings form the largest use category, accounting for 33% of the inventory, followed by hospitals at 27% and residential buildings at 16%. This is an important finding because it suggests that the market for base isolation has broadened beyond critical facilities and is increasingly connected to business continuity, reduced damage, and wider resilience objectives.

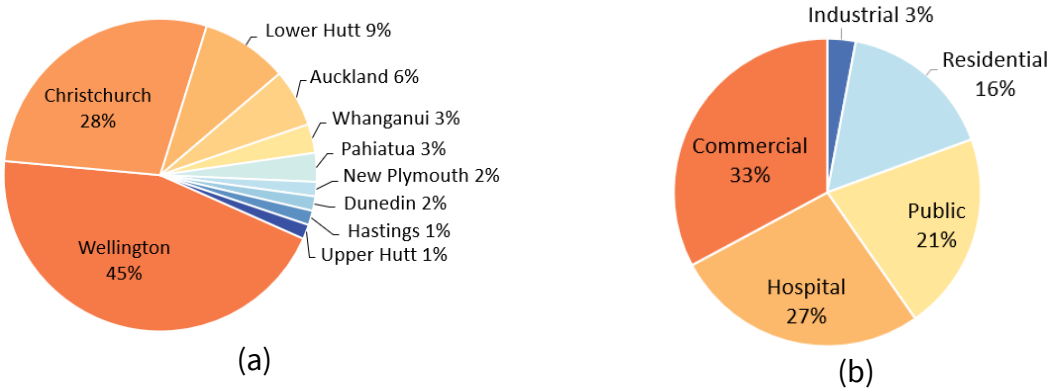


Figure 3: (a) Distribution of base isolated buildings across New Zealand and (b) building use.

Isolation systems and structural characteristics

Lead rubber bearings are the most common isolation device type in the New Zealand building inventory as shown in Figure 4a. The database indicates that lead rubber bearings account for approximately 65% of the identified systems, with many used in combination with sliders. Triple friction pendulum bearings account for approximately 16% of systems; however, use of these is rapidly dominating due large moat-space requirements necessitated by increased seismic hazard.

The inventory also shows that base isolation is applied across a range of structural systems. Reinforced concrete and structural steel are both common, and some buildings use hybrid arrangements combining different systems across directions or storeys. Reinforced concrete walls are a frequently used lateral load-resisting system, while steel moment-resisting frames, braced frames, and mixed systems are also represented.

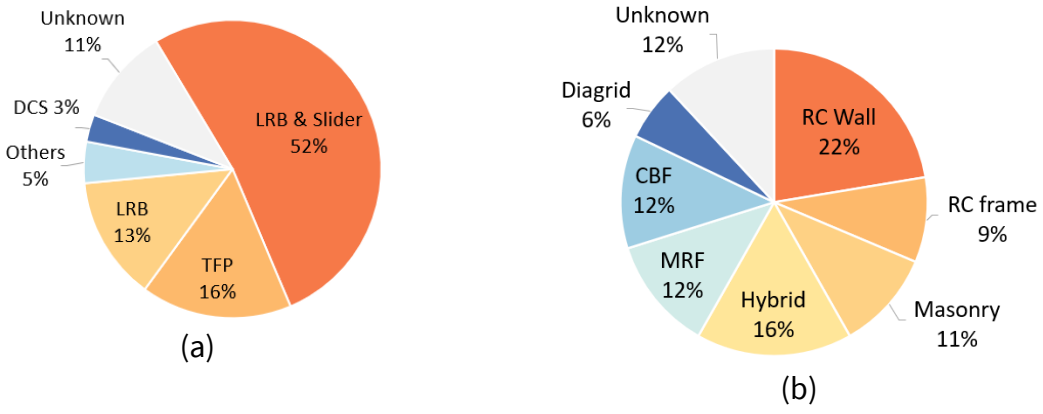


Figure 4: (a) Isolator type distribution and (b) building structural system type.

Relevance to the lifetime-cost assessment

The database has helped identify variation in design parameters used across New Zealand. This provided information that was useful for selecting representative case-study buildings for loss analysis in following sections. The database forms an important foundation for the wider lifetime-cost analysis in this project. It helps ensure that the case-study buildings and assumptions used in subsequent modelling are consistent with what is observed in New Zealand practice. It also identifies the types of parameters that are most relevant for linking building design to loss outcomes, including building use, structural system, floor system, isolation system, moat clearance, and displacement demand.

For NHC, the value of the database is that it supports a more evidence-based understanding of the current base-isolated building stock. This is relevant to natural hazard risk reduction because the benefits and vulnerabilities of base-isolated buildings cannot be assessed without knowing what has actually been built, where it is located, and how it has been designed. The database therefore provides a foundation for future work on building performance, expected losses, retrofit prioritisation, and the role of low-damage design in reducing long-term earthquake risk.

Overall, the database strengthens the evidence base for evaluating base isolation in New Zealand. It supports the subsequent loss and lifetime-cost assessment undertaken in this report and provides a foundation for research, design guidance, and risk-informed decision-making. The next section of this report will describe the methodology adopted in this study to undertake the lifetime-cost assessment, followed by a summary of the results.

Lifetime-Cost Assessment

Overview of project workflow

The overall framework for the comparative analysis between fixed-base and isolated buildings is shown in Figure 5. A total of four building pairs were considered for the comparisons made in this study. Each building pair consisted of a 'conventional' (i.e., fixed-base) design and a corresponding base-isolated design. Both buildings were assigned structural and non-structural member components for loss analysis. The buildings were modelled in 3D and analyzed in the OpenSeesPy software over a suite of ground motions scaled to a range of intensities. From each analysis, the engineering demand parameters (EDPs) were extracted to represent the structural response. The analysis also considered the response of the building with and without the impacts of moat-wall pounding. These EDPs were then used to estimate repair cost to the building using the FEMA P-58 methodology [6]. Implementation of the FEMA P-58 methodology was done through the Pelicun Python package [7]. Finally, the costs were compared on a time-basis to understand the life-cycle cost assessment of base-isolated buildings compared to fixed-base buildings.

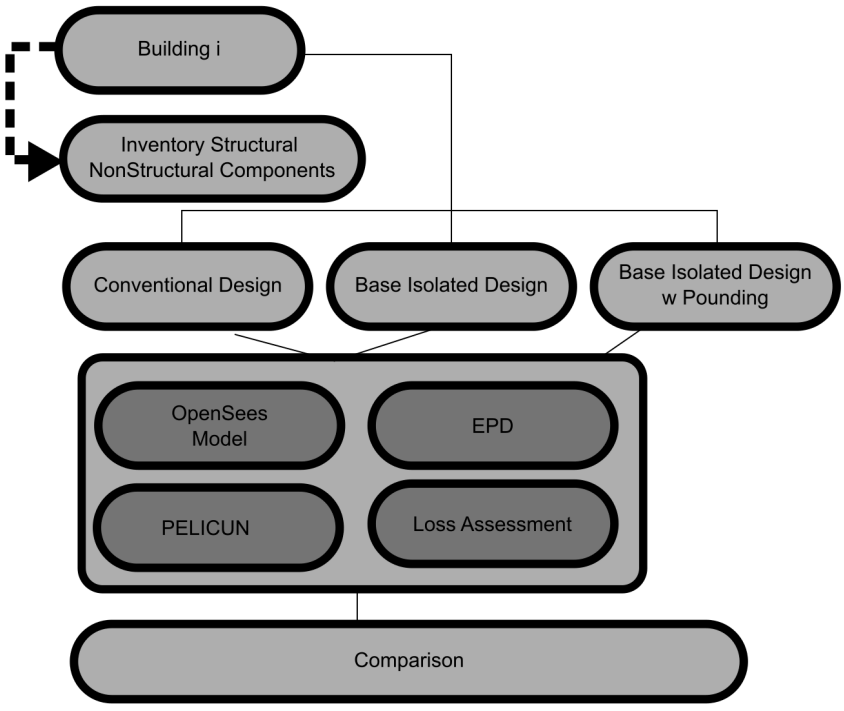


Figure 5: High-level methodology overview for the comparative study.

Building archetype selection and assumptions

A total of 4 building triplets (i.e., fixed-base, base-isolated with and without pounding) were considered in this study. The first was a building, referred to as the 'L-shaped' building, is shown in Figure 6. It is comprised of four storeys above ground and a single-storey basement over the entire building footprint. The basement is primarily carparking and plantrooms are in the basement and on the roof. The building is an Importance Level 2 structure with a retail and commercial use.

The L-shaped building is a steel moment frame, with seismic isolation just under ground floor level. The seismic isolation will cover the entire building footprint, including the courtyard area. The building plan dimensions are approximately 60m x 60m. The building is located in Christchurch on subsoil class D [3] and designed for a moderate ductility demand ($\mu \leq 3$). The base isolation system consists of double concave slider isolators (54 No. of isolators) and a design travel displacement of 500mm.

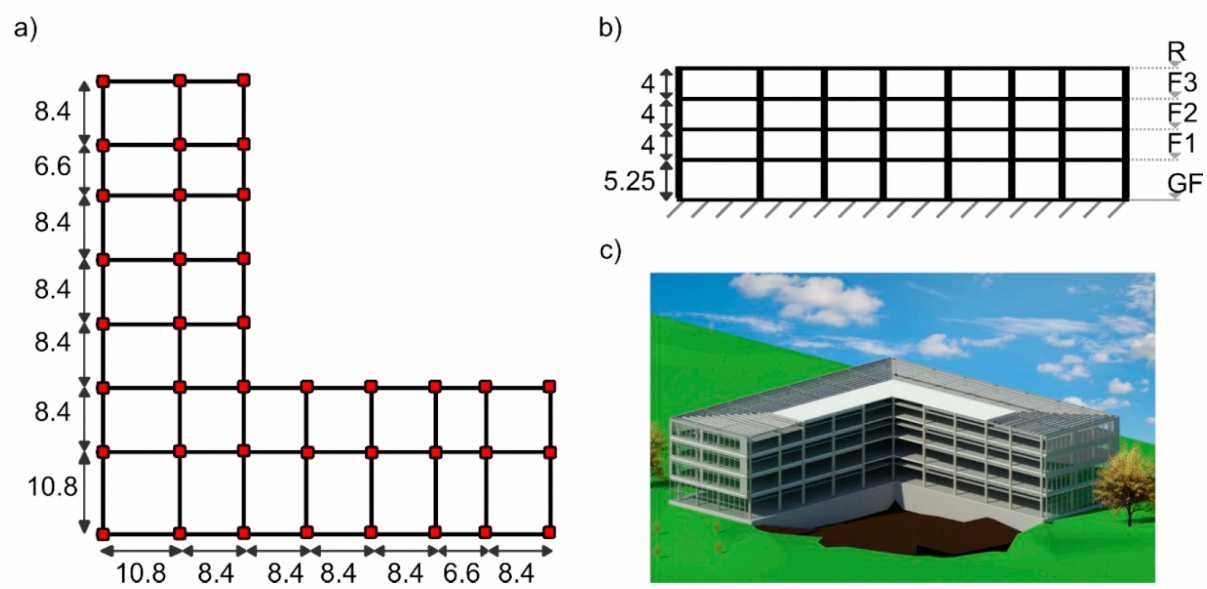


Figure 6: Case-study building (a) plan view, (b) elevation view, and (c) isometric view (units metres) [8]

The remaining buildings, termed '2-storey', '4-storey' and '8 storey' all featured identical design criteria, differing only by the number of storeys. The generic floor plan was 960 m^2 per floor, and sketch of this is floor plan is shown in Figure 7. Each building was designed for Wellington soil class D and as a steel moment-resisting frame in both orthogonal directions. The fixed-base buildings were designed to be governed by the 2.5% storey drift limit, while the base-isolated system properties were designed such that the period at ULS was at least 3 times the fixed-base building period [5], [9]. The floor system was assumed to be ComFloor 60 composite slab (150mm) depth. The base-isolated buildings have grillage beams (1200x1000 mm). The base isolation system was designed to be a

combination of lead rubber bearings (LRBs) and flat-plate sliders. The structural section sizes for the moment-resisting frame for both fixed-base and base-isolated buildings is detailed in Table 1 and Table 2, respectively. The vibrational characteristics of each building is shown in Table 3.

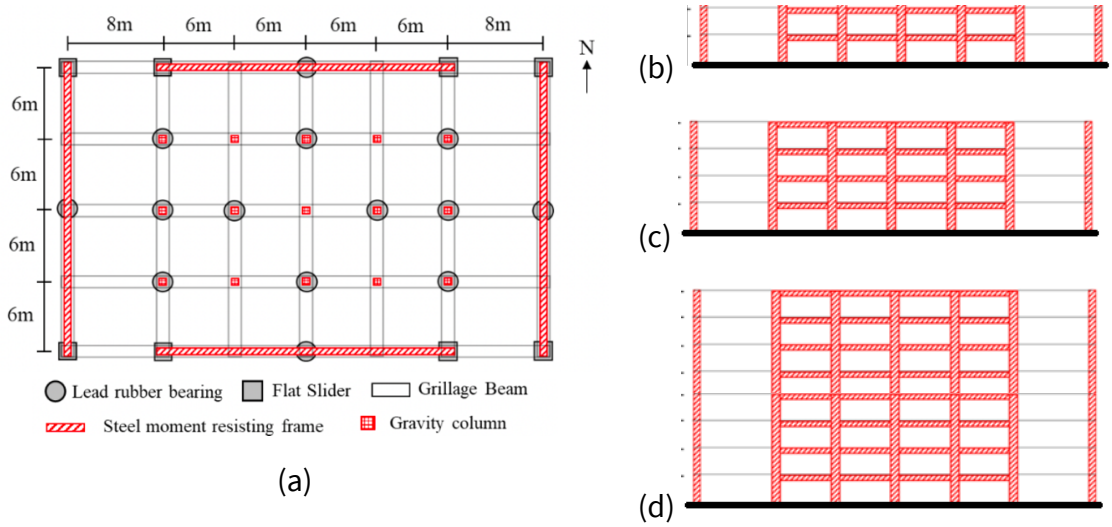


Figure 7: Case study buildings sketches (a) typical floor and isolation level plan (b) 2-storey (c) 4-storey and (d) 8-storey building elevations [9]

Table 1: Section sizes at each floor for the fixed-base buildings.

	2-storey		4 Storey		8 Storey	
Storey	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**
8 (3,6m)					327HCC156	460UB67.1
7 (3,6m)					327HCC332	530UB82
6 (3,6m)					327HCC332	600UB125
5 (3,6m)					400HCC270	700WB130
4 (3,6m)			310UC137	410UB59.7	400HCC270	800WB146
3 (3,6m)			400HCC333	530UB82	500HCC356	800WB146
2 (3,6m)	327HCC156	60UB67.1	400HCC333	600UB125	500HCC356	900WB175
1 (4,5m)	400HCC270	600UB101	500HCC315	900WB175	500HCC485	1000WB215

Table 2: Section sizes at each floor for the base-isolated buildings.

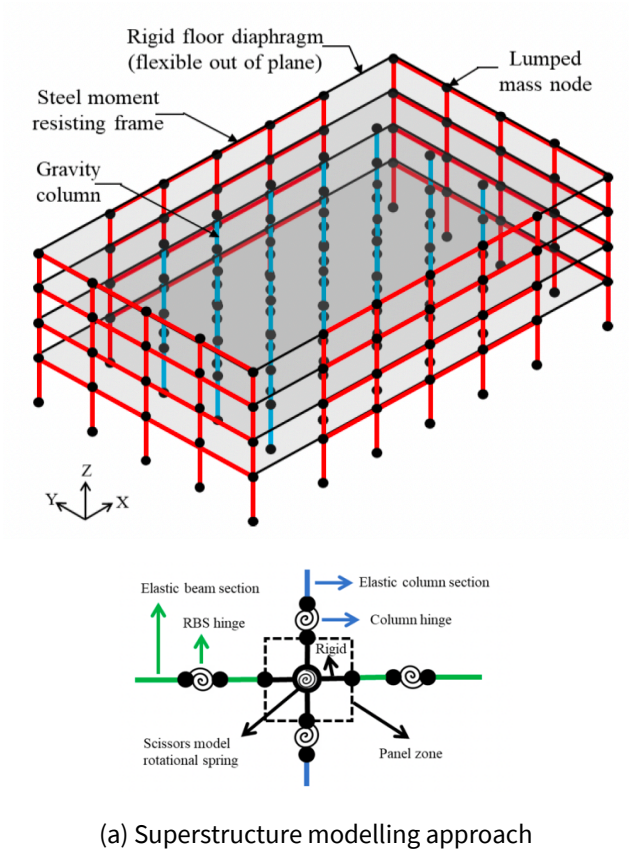
	2-storey		4 Storey		8 Storey	
Storey	**Columns**	**Beams**	**Columns**	**Beams**	**Columns**	**Beams**
8 (3,6m)					327HCC197	530UB82
7 (3,6m)					327HCC197	600UB125
6 (3,6m)					400HCC385	800WB146
5 (3,6m)					400HCC385	900WB175
4 (3,6m)			500WC228	600UB101	500WC383	1000WB215
3 (3,6m)			500WC228	600UB113	500WC383	1000WB215
2 (3,6m)	327HCC207	530UB82	500WC340	700WB150	500HCC485	1000WB215
1 (4,5m)	327HCC207	700WB130	500WC340	700WB150	500HCC485	1000WB215

Table 3: Period characteristics of each test structure.

	Period (s)		Moat gap (mm)	
Storey	**Fixed-base**	**Base-isolated**	**Fixed-base**	**Base-isolated**
L-shaped	1.26	2.08	N/A	650 mm
2-storey	0.9s	2.1s	N/A	500 mm
4-storey	1.1s	2.6s	N/A	650 mm
8-storey	1.7s	3.9s	N/A	700 mm

Conventional and base-isolated building modelling

The buildings were all modelled in 3D using in OpenSeesPy. As shown in Figure 8a, the moment frames were modelled using the lumped-plasticity approach, with the modified Ibarra-Median-Krawinkler deterioration model is used to model the beam and column hinges. Beam-column joints were modelled as rigid zones via rigid elements. The base isolation devices were modelled using a zero-length element approach shown in in Figure 8b and c. This hinge accounts for the translational and rotational degrees of freedom. The horizontal stiffness characteristics of the isolators are modelled using the Steel01 element. Full details of the superstructure and isolation plane modelling approach can be found in references [9], [10].



(a) Superstructure modelling approach

Figure 8: Modelling approach for building in OpenSeesPy [9], [10]

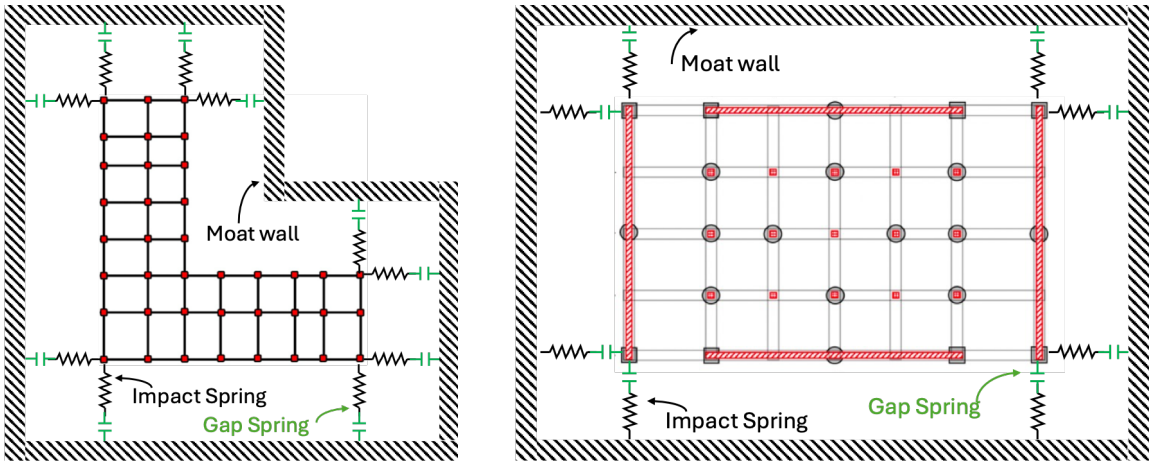
Moat pounding and isolator limit-state modelling

As an additional novelty of this study, potential contact of the structure with the surrounding moat wall was explicitly captured using a gap element, as shown in Figure 8d [10]. In summary, the approach involves the setting two springs in series at each corner of the structures (Figure 9) and in each orthogonal direction: one spring to represent the gap between the structure and the moat wall (ElasticPPGap element), and an additional spring to model the force-deformation characteristics of the impact process (zeroLengthElement). The impact material model was assumed to be linear-elastic; past and ongoing research suggested the response of the structure is not overly sensitive to the selection of the impact model [11]. The impact element stiffness was derived using an assumed moat wall material properties (concrete) and contact area at impact, using Equation 1 and 2 below The moat wall itself was modelled as a rigid node (i.e., ignoring moat wall flexibility).

$$K = \frac{E * A}{L} \qquad \text{Equation 1}$$

$$E = 4700 * \sqrt{fc'} * \left(\frac{\delta}{2400}\right)^{1.5} \qquad \text{Equation 2}$$

Where K is the stiffness of the impact element, A is the contact area during impact/pounding, L is the length of the assumed impacting surface (along the direction of the building), E is the Young's Modulus of concrete, $f'c$ is the concrete compressive strength (assumed as 40 MPa) and δ is the unit weight of concrete.



(a) L-shaped building (b) 2, 4 and 8-storey buildings

Figure 9: Modelling concept for moat wall and impact springs.

Structural and non-structural components and contents assumptions

The structural and non-structural components (NSCs) considered in the loss analysis have been mostly adapted from [12], and are listed below. These have been broadly categorized as Group B (structural components) Group C (drift sensitive non-structural components) and Group D (acceleration sensitive non-structural components). The exact quantities and settings are listed in Appendix 1.

Group B: Structural:

1) Steel beam-column connections
2) Steel column base plate connection

Group C: Drift sensitive non-structural:

1) Partitions
2) Interior and exterior glazing
3) Precast concrete cladding
4) Stairwells
5) Suspended ceilings

Group D: Acceleration sensitive non-structural:

1) Elevators
2) Water and Sewerage Piping

3) Chiller and Cooling Tower
4) HVAC system
5) Fire sprinkler system
6) Transformers

Note, the base isolators and foundation systems were not considered in the loss analysis study. In both cases, the loss fragility data for these components is not sufficient in literature. However, in the case of isolators, empirical data from past earthquakes have suggested little to no damage when subjected to earthquakes up to MCE-level intensity [13], [14].

The fragility functions for different components were similarly adopted from [12], as these were tailored to the New Zealand context. This includes the repair/replacement costs used in that study.

FEMA P-58 direct-loss assessment

Damage states and associated losses were directly calculated via the FEMA P-58 database of components fragilities (or adapted fragilities as specified in the previous section). The exception to this case was when a building was deemed to have reached a collapse limit state or a 'unrepairable' limit state. The former was defined as non-convergence of numerical analysis due to excessive deformations, or analyses that resulted in a maximum storey drift that exceeded 5%. The latter was identified as cases where the median cost of the building exceeded 60% of the initial build cost or the residual drift ratio exceeded 1% (residual drift was estimated using the procedure given in [6]). In both collapse and unrepairable scenarios, the cost associated with 'loss' associated was taken as a initial construction cost of the building.

Replacement costs were estimated using a planning-level Wellington commercial office replacement rate of NZ$4,500/m^2 applied to the gross floor area of each building. The base-isolated cases were assigned an additional 3% isolation-system premium, representing the cost of lead-rubber bearings, flat sliders, isolation-plane grillage, installation, testing, detailing, and associated design/coordination allowances [15]. The resulting total building costs are listed in Table 4.

Table 4: Construction costs assumed for each structure.

	Construction Cost (in $millions)	
Building	**Fixed-base**	**Base-isolated**
L-shaped	$52.3	$53.8
2-storey	$7.7	$7.9
4-storey	$15.4	$15.9
8-storey	$30.7	$31.6

Ground motion selection

The ground motion records used to perform the time-history analysis were scaled to match the NZS 1170.5:2004 hazard. The ground motions were scaled for Wellington city with a site soil class D. The selected *IM*s used for conditional scaling were Spectral Acceleration (S_a) at periods: 0.5 sec, 1.0 sec, 2.0 sec, and 4.0 sec. For each *IM*, three separates Uniform Hazard Spectra were used to represent Probability of Exceedance in 50 years (PoE_{50yrs}) as 63%, 10%, and 0.5%. In addition to the three suites of 20 ground motion, six supplementary ones were acquired by applying a scale factor to the corresponding base hazard level, as shown in Table 5, resulting in a total of nine different hazard levels. A similar process was repeated for the other three selected IM, SA (0.5 sec), SA (2.0 sec), and SA (4.0 sec) to achieve nine different sets of 20 earthquake records which are not presented for brevity. The PoE_{50yrs} (%) of all nine hazard levels are presented in Table 5, where the corresponding value of SA (1.0s) as well as the calculated scale factor. The acceleration spectra for both orthogonal directions for the records conditionally scaled to 1.0 s is shown in Figure 10 for the Wellington location (i.e., pertinent to building 2-storey, 4-storey and 8-storey).

Table 5: Adopted hazard levels information.

PoE_{50yrs} (%)	Return Period (years)	SA (1.0 sec) (g)	Scale Factor*	Base Hazard Level
80	30 (SLS)	0.141	0.68	PoE_{50yrs} = 63%
63	50	0.207	1	PoE_{50yrs} = 63%
50	75	0.283	1.366	PoE_{50yrs} = 63%
20	225	0.617	0.652	PoE_{50yrs} = 10%
10	475 (ULS)	0.945	1	PoE_{50yrs} = 10%
5.0	975	1.346	1.424	PoE_{50yrs} = 10%
2.0	2,475 (MCE)	1.97	0.62	PoE_{50yrs} = 0.5%
0.5	9,975	3.18	1	PoE_{50yrs} = 0.5%
0.2	24,975	4.217	1.326	PoE_{50yrs} = 0.5%

* Scale factors are found by dividing the SA (1.0 sec) of the hazard level to the SA (1.0 sec) of the respective base hazard level.

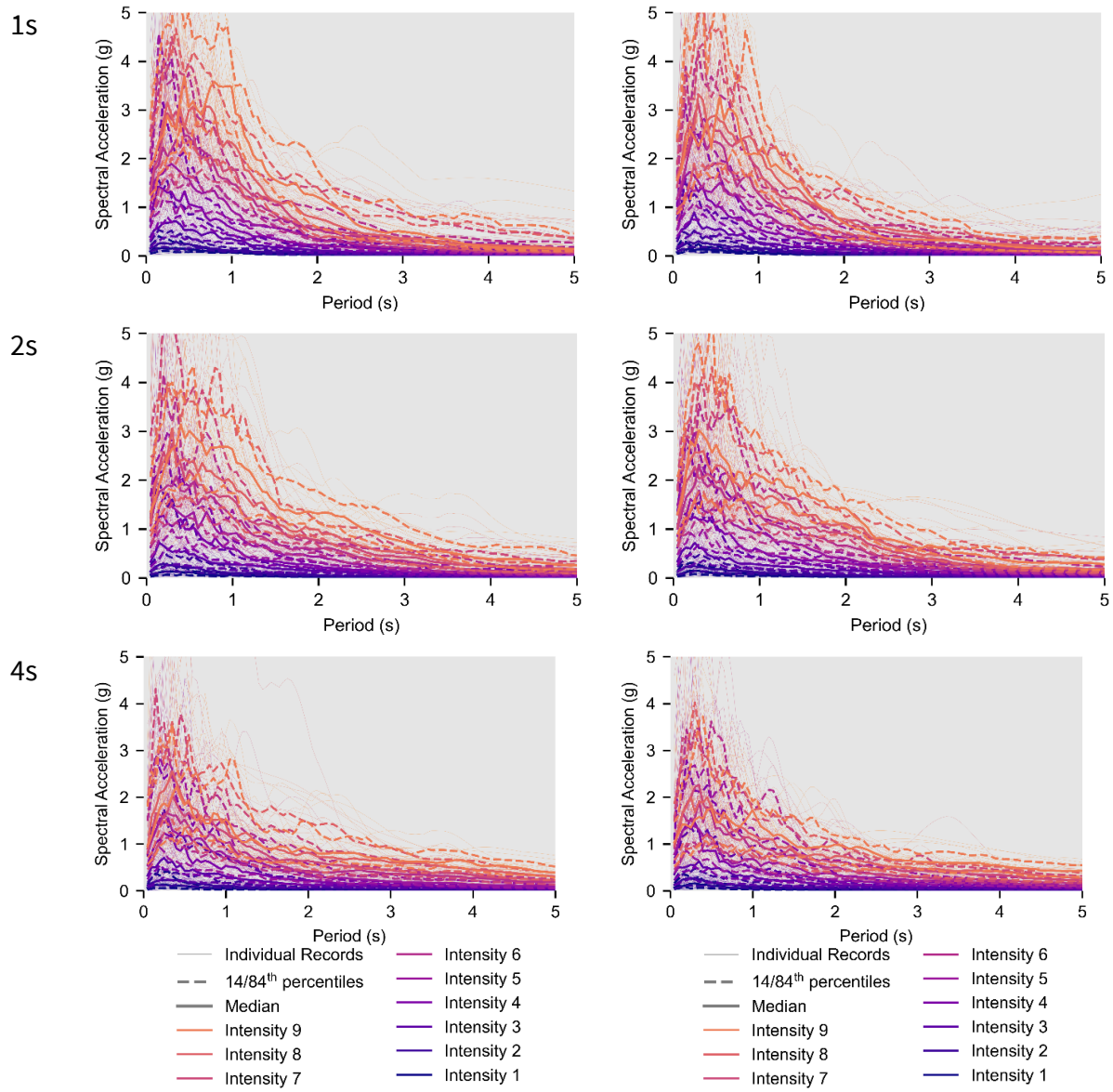


Figure 10: Acceleration spectra for ground motions used in this study (right – Dir 1, and left – Dir 2)

Uncertainty and limitations

Since the number of records used in this study is limited to 20, it is necessary to consider uncertainty in presentation of results. These demands are used to create simulated demands to be able to complete a higher number of iterations than the number of seismic records using the methodology suggested in FEMA P-58 Appendix G. The larger number of demands capture the relationship between demands and follow a log-normal distribution. The methodology also includes the uncertainty β_m: the dispersion to account for the quality of construction (β_c) and the analytical model (β_q). For this study, an average quality is assumed with a value of 0.2 for both β_c and β_c. The total dispersion β_m is then calculated using Equation 3.

$$\beta_m = \sqrt{\beta_c^2 + \beta_q^2}$$ Equation 3

Pelicun allows to include different scenarios. The result from each realization can fall into a repairable, irreparable, and collapse scenario. The irreparable scenario uses a median and dispersion related to the residual drifts. Pelicun uses the procedures recommended in FEMA P-58 based on the yield drift ratios to convert them to residual drifts.

Results and discussion

Results of building archetype analysis

The peak storey drift and floor accelerations distributions are shown in Figure 11 for SLS, ULS and MCE Stripes for the 2-storey building (data for the other buildings is omitted here for brevity, but can be found in Appendix 2). The results are shown for the 3 cases of fixed-base, base-isolated and base-isolated with pounding variations. At SLS the performance of the three building variations very similar. At ULS demands, the effect of base isolation on reducing the superstructure storey drift and floor accelerations is clearly visible; while the fixed-base building experiences mean drifts of 2.5%, base isolated variations maintain drifts at around 0.5%. The effect of pounding does not become significant until MCE level events – this is expected as the moat gap was sized to this earthquake intensity level. Here, it can be seen that the effect of moat-wall pounding results in dramatic amplifications of accelerations (up to 10 times higher), and this transmits into the superstructure to result in higher storey drifts too; increasing from a mean of 1-2% without pounding to over 4% with pounding.

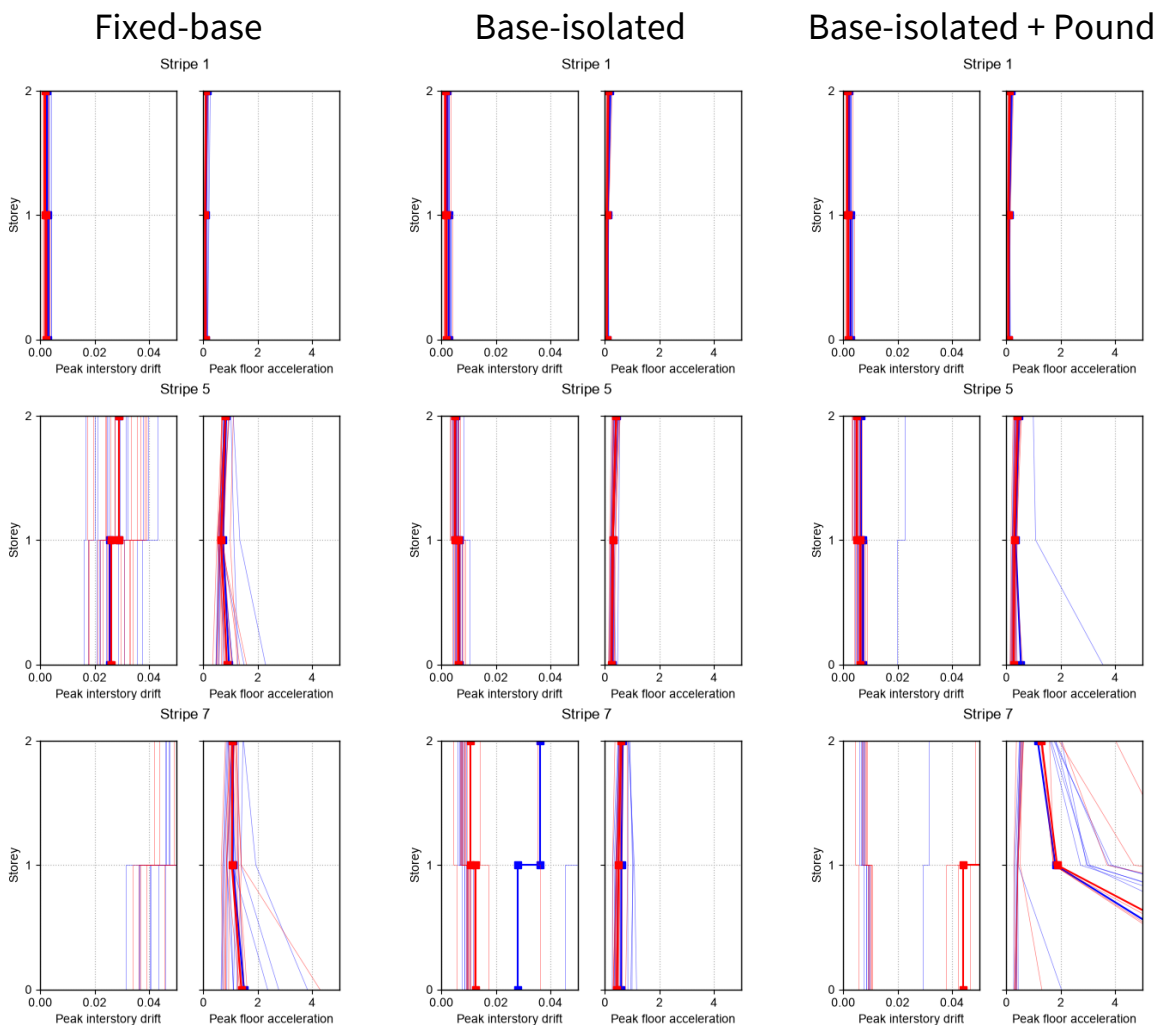


Figure 11: Mean of peak storey drift and floor acceleration distributions up the height for 2-storey building for SLS (left) ULS (middle) and MCE (right) demands. (blue and red correspond to two orthogonal directions, while dashed lines indicate ±1 standard deviation)

The collapse scenario was based on the number of analyses that exceeded the limit state of a maximum inter-storey drift of 0.05 rad. An example of how collapse cases and collapse fragility were determined is shown in Figure 12 for the 2-storey fixed-base building. Collapse cases are indicated as blue data points lying above the red dashed line. The remainder plots for all other buildings are provided in the Appendix 3.

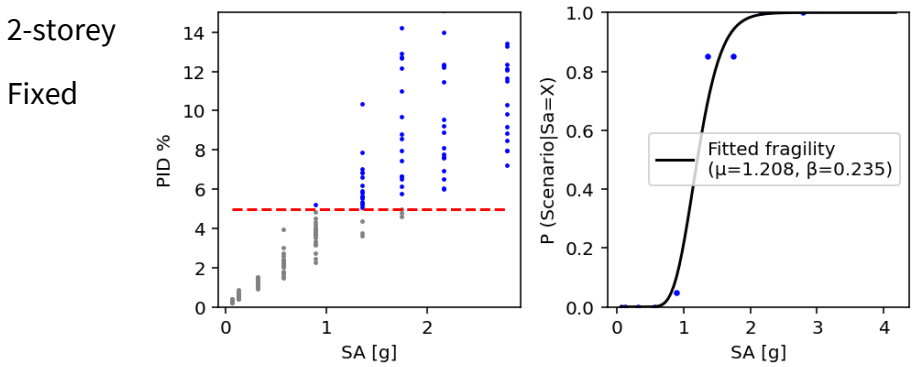


Figure 12: Peak storey drift achieved for each ground motion, and associated collapse fragility

Expected Losses

Figure 13 compares the expected loss in NZD as a function of annual frequency of exceedance (AFE) for the fixed-base, base-isolated, and base-isolated-with-pounding cases. The expected loss values include repair losses for repairable realisations and replacement-level losses for cases classified as collapsed or irreparable. The shaded bands represent the 16th and 84th percentile loss estimates and provide an indication of the dispersion in the loss results at each intensity level.

For the 2-storey building, base isolation produces a clear reduction in expected loss across nearly the full AFE range. The fixed-base case increases rapidly from low-to-moderate intensities and reaches replacement-level losses at the upper stripes, while the base-isolated case remains substantially lower. The base-isolated-with-pounding case follows the no-pounding isolated case at lower intensities and diverges only at higher intensities, where isolation displacement demand is large enough for moat-wall impact to occur.

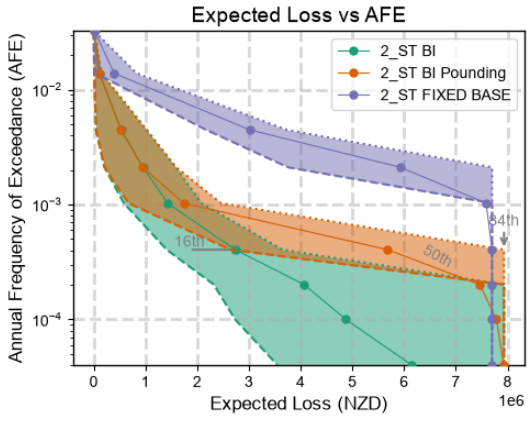
For the 4-storey building the overall trends are similar to the 2-storey; however, at the highest intensity levels, the distinction between the fixed base and isolated cases reduces as collapse and irreparability become more influential. The base-isolated-with-pounding losses are very similar to the base-isolated case. However, it should be noted that the current loss analysis does not explicitly include damage to moat walls, isolation devices, foundations, or other isolation-plane components.

The 8-storey building is the main exception. The base-isolated case reduces expected loss at intermediate intensities, particularly between stripes 3 and 6, but this benefit is partly offset by higher losses at the first two stripes (i.e., 1/30 and 1/50 year return periods). These low-intensity stripes have high AFE and therefore strongly influence the expected annual loss. Although the base-isolated model reduces floor accelerations relative to the fixed-base model (by approximately 30-40%), it produces higher upper-storey drifts (by 30-40%) at low intensities, particularly near storeys 7 and 8. This presents an interesting situation where the earthquake intensity is not sufficient to activate the base-isolation

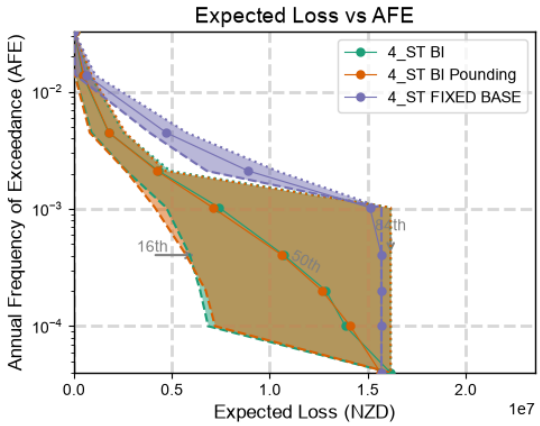
effect, resulting in essentially an overall more flexible building. At intermediate intensities the base-isolated case performs better. At the high-intensity end, the base-isolated 8-storey case receives some additional economic benefit from reduced collapse probability; however, once both systems are near replacement-level loss, the base-isolated case can also appear more expensive because its replacement cost is higher.

The L-shaped case-study building provides some contrast to the regular 2-, 4-, and 8-storey building set. Figure 13d shows that the fixed-base L-shaped building has substantially higher expected losses than the base-isolated cases over nearly the full AFE range. Unlike the 8-storey study, in the L-shaped building, the base-isolated cases reduce both peak floor acceleration and peak inter-storey drift over most stripes. Furthermore, the number of collapse cases is significantly less for the base-isolated building compared to the 2-8 storey studies. These two observations will have substantial impact on the losses examined in the next section. Finally, the base-isolated-with-pounding case follows the base-isolated case closely at low and moderate intensities, but diverges at the higher intensity stripes where pounding is more likely.

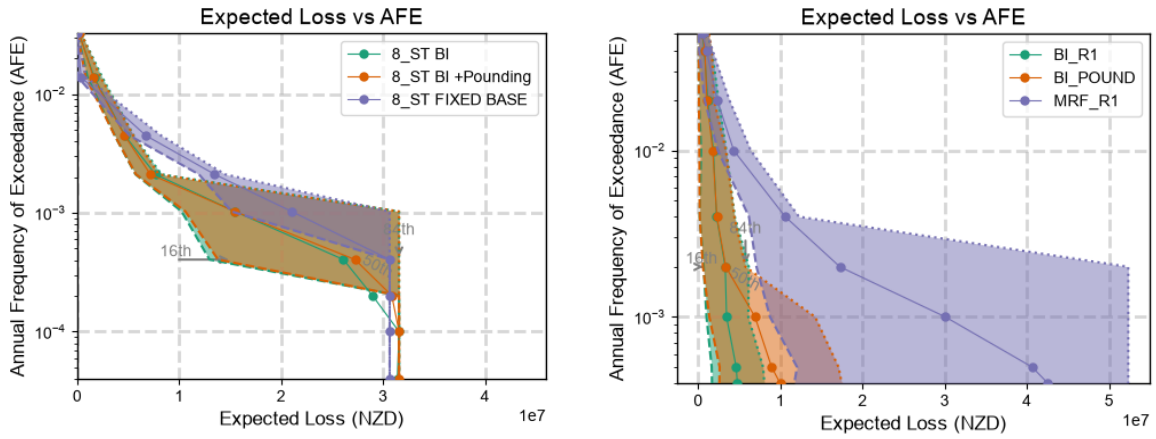
Overall, the expected-loss results show clear direct-loss benefits from base isolation for the 2-storey, 4-storey, and L-shaped buildings. The 8-storey building still benefits at intermediate intensities, but its annualised loss reduction is weakened by low-intensity upper-storey drift-sensitive losses and by the sensitivity of the result to the selected steel connection fragilities. This makes the 8-storey result more sensitive to the design intricacies (i.e., tuning of isolation system), and also the modelling assumptions, particularly the superstructure drift response and the selected drift-sensitive steel connection fragilities.



(a) 2-storey



(b) 4-storey



(c) 8-storey (d) L-shaped

Figure 13: Expected loss versus annual probability of exceedance for all building cases, and showing the 16th and 84th percentile uncertainty bands (i.e., approximately ±1 standard deviation).

Loss disaggregation

In this section the aggregated cost losses shown in the previous section are disaggregated to understand what components contribute the most. The contributions of each component to total losses is categorized into the Groups described earlier (B, C and D), and also arranged by the floor on which the loss is being incurred. The resulting data is shown in Figure 14 for SLS, ULS and MCE intensities for the 4-storey case only as trends are generally similar across the 4 building cases. Data for every intensity and for every building case can be found in the Appendix 4. Note, structural joints, ceilings, partitions, chillers, fire sprinklers and elevator have been additionally singled out in the plot legends as these contribute the most to losses.

Across all buildings, the dominant contributors at SLS are generally drift-sensitive structural components, particularly the steel joint components. These correspond to traditional steel moment-frame joint fragilities and are triggered by peak inter-storey drift ratio. Acceleration-sensitive components, including elevator and other non-structural components, contribute to the total loss but are generally secondary in the low-stripe comparisons considered here. The difference between the fixed-base and base-isolated cases are generally negligible.

At ULS, considerable differences are seen between the fixed-base and isolated buildings, with the fixed-base building incurring roughly 2-3 times higher losses. Meanwhile, the effect of pounding does not produce a difference between the two isolated cases; namely because the displacement demands are low at this earthquake intensity level and so no pounding occurs.

The trends changes again at MCE level demands, where pounding will be expected. The fixed-base building still results in the highest overall losses, and still 2-3 times higher than the base isolated building with unlimited moat gap. The base-isolated building with pounding consideration produces significantly larger losses than when pounding is neglected. These losses are mostly concentrated in the first floor and are mostly attributed to acceleration-sensitive components. This results in losses in the first floor of approximately 3 times that when pounding is ignored.

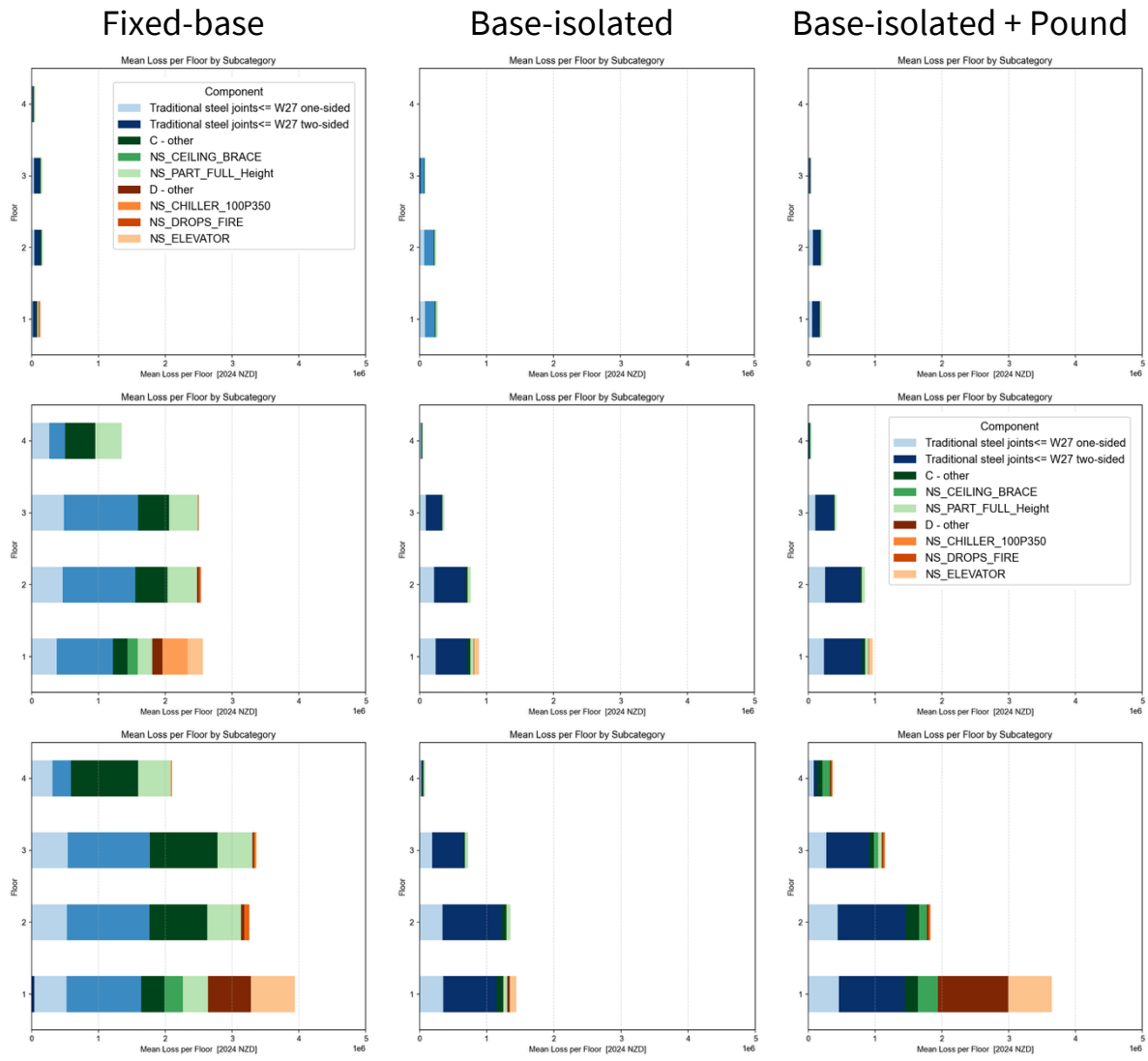


Figure 14: Expected loss categorised by component Group and by floor for SLS (left) ULS (middle) and MCE (right) demands for the 4-storey building.

The 8-storey building is the main exception to the otherwise consistent trend. As discussed in the previous section, the base isolated building results in higher expected losses than the fixed-base case, and the loss disaggregation in Figure 27 (Appendix 4) shows that this is caused primarily by upper-storey drift-sensitive steel joint losses. It is reiterated here that the poor low-intensity performance of the 8-storey base-isolated case is not caused by acceleration-sensitive non-structural components (which would typically

affect building usability). Instead, the loss penalty is caused by higher upper-storey inter-storey drifts in the base-isolated model. It is debatable whether such small drifts would truly result in a necessity of repair of structural joints that would results in the 'loss'.

Overall, the loss disaggregation indicates that the main economic results are controlled by drift-sensitive structural repair losses rather than by acceleration-sensitive non-structural losses. For the 2-storey, 4-storey and L-shaped buildings, base isolation reduces these drift-sensitive losses and produces a clear economic benefit. For the 8-storey building, the isolated model reduces accelerations but produces higher low-intensity upper-storey drifts, causing large drift-sensitive connection losses that offset much of the expected economic benefit. This suggests that the 8-storey result should be interpreted with care and should be discussed as being sensitive to the isolation design and corresponding superstructure drift response.

Downtime and total-loss comparison

Discount Rate

The expected annual losses were converted into cumulative present-value costs to compare the upfront cost premium of base isolation with the expected reduction in future earthquake losses. Discounting was applied because future repair losses do not have the same economic value as costs incurred at the time of construction. This discount rate represents the difference between interest and inflation rates [16]. The net present value (NPV) at each time instance is calculated using the formula provided in FEMA P-58 and presented in Equation 4, where A is a future expenditure, t is a period of time in years, and r is the discount rate. For this study, a range of discount rates has been used to estimate the payback period. Note that there are many other methods to calculate the NPV in literature.

$$NPV = \frac{A}{(1+r)^t} \qquad \text{Equation 4}$$

Break-even point comparison

The seismic losses were calculated for a range of intensities using Pelicun and including the collapse/non-repairable cases identified in postprocessing. The postprocessing for each stripe included (1) fitting a lognormal distribution; (2) obtain the median of each stripe (also the 14th and 84th percentiles); (3) using the median values are used in conjunction with the annual frequency of exceedance of each stripe to calculate the

average annual expected loss (AAL), and (4) convert the future economic losses to present values. Equation 5 was used to obtain the average annual loss [17].

$$AAL = \sum_{i=1}^{N} \Delta\lambda_i \cdot L_i \qquad \text{Equation 5}$$

Where N is number of stripes, λ_i is the APoE for stripe i and, L_i is the expected loss for stripe i. This computation represents the area under the curve in Figure 13.

The expected annual loss results were converted into cumulative present-value cost curves to compare the initial cost premium of base isolation with the expected reduction in future earthquake losses. Figure 15, Figure 16, Figure 17 and Figure 18 show the cumulative cost of the fixed-base, base-isolated, and base-isolated-with-pounding cases over time. The plotted values include the initial cost premium of base isolation and the cumulative discounted expected annual loss. The intersection between the fixed-base and base-isolated curves represents the break-even time, where the additional initial cost of base isolation is offset by lower expected earthquake losses. Each figure also includes cases where a 2.0% and 6.0% discount rate has been applied. The tabulated data of the annual losses and break-even periods is provided in Table 6.

For the 2-storey, 4-storey, and L-shaped buildings, the base-isolated case has a relatively short break-even period ranging from 7-13 years. For the base-isolated-with-pounding cases, the break-even period is only slightly longer (by 1 year at most). From this it can be gleaned that the occurrence of pounding does not render base-isolation inefficient because the losses at the intensity levels at which pounding occurs contribute negligibly to the overall annualised loss. Overall these result supports the premise that, for low-rise buildings to mid-rise buildings, the higher upfront cost of base isolation can be offset over a realistic building ownership or service-life timeframe through reduced earthquake repair and replacement losses.

The 8-storey regular building is the exception in this study in that its break-even period is much longer at around 95 years. Although the isolated case still reduces expected annual loss, the annual saving is only approximately $10k/year. This is small relative to the initial base-isolation premium of approximately $1.0M, producing the longer break-even period. As discussed in the preceding sections on expected loss and loss disaggregation, the weak economic benefit is caused by unexpectedly high low-intensity drift-sensitive losses in the base-isolated model, especially at the upper storeys. These frequent-event losses offset much of the benefit obtained at intermediate intensities. This behaviour is a function of the original building design – specifically likely due to the inherent flexibility of the resulting superstructure in designing to 2.5% storey drift limit. Typically, superstructures of base-isolated buildings are designed to much smaller drifts – in the order of 1.0-1.5% at

ULS [18] – so the losses observed at lower drifts for the 8-storey building are perhaps not representative of what would be expected in a real-building.

Applying discounting lengthens the break-even period for the 2-storey, 4-storey, and L-shaped buildings, and removes the break-even point for the 8-storey building. This occurs because future loss savings are valued less as the break-even period becomes longer. With a 6% discount rate, the break-even period for the 2-storey, 4-storey and L-shaped buildings increases by about 1.5 times, compared with a 0% discount, as shown in Table 6. In general, the effect of applying a discount rate will increase as the break-even period increases, This is because longer-break-even period allow for more time for the discount to accumulate significant effect (compound interest effect). If larger base-isolation premiums are assumed (i.e., above 3%), the break-even periods will also be longer, and subsequently the effect of discounting will become more pronounced.

Across the case studies, the shortest break-even periods occur where base isolation produces consistent reductions in both component-level repair loss and replacement-level outcomes. This is observed most clearly in the 2-storey regular building and the L-shaped building. The 4-storey regular building also shows a clear benefit, but with a longer break-even period because the initial premium is larger relative to the annual saving. The 8-storey regular building remains the least favourable case because its base-isolated configuration does not achieve the same reduction in annualised loss and because frequent low-intensity drift-sensitive losses reduce the net economic benefit. It should be noted that the assumptions on the base-isolation premium proportionally affects the break-even points reported here. While a 3% premium has been used here, this number can be as high as 10% in some cases [19], which would roughly triple the break-even periods. However, even so, a roughly 30 year break-even point (in the case of a 10% premium) would still be an attractive outcome given building life-times are generally much greater than this value. Furthermore, as base isolation adoption grows the technology isolation component will become more affordable and expertise will be less confined to specialist firms. These factors will collectively make base-isolation an even more attractive design option from an economic perspective.

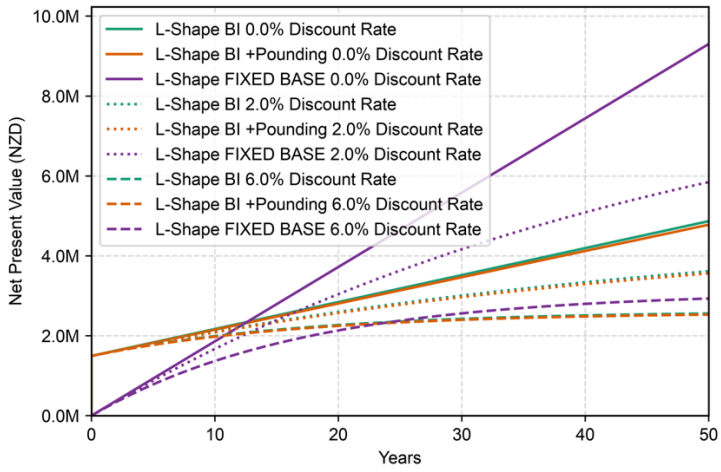


Figure 15: Cumulative annual cost of base-isolated vs fixed-base building for building L-shaped building.

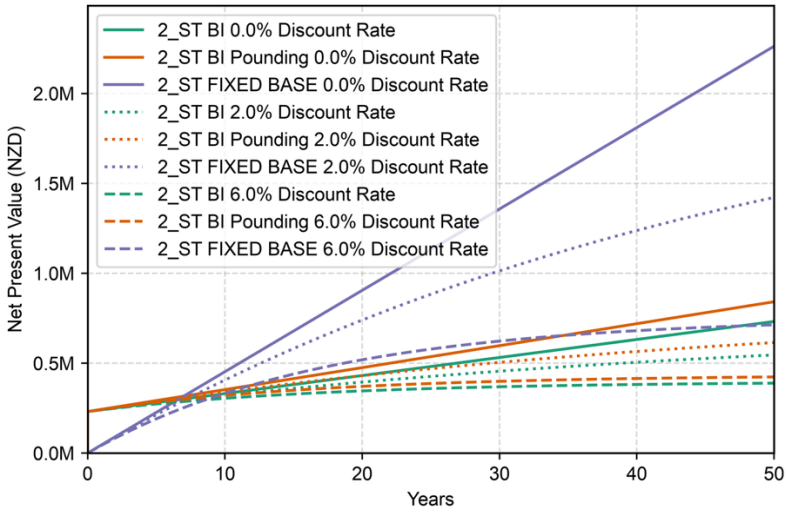


Figure 16: Cumulative annual cost of base-isolated vs fixed-base building for building 2-storey.

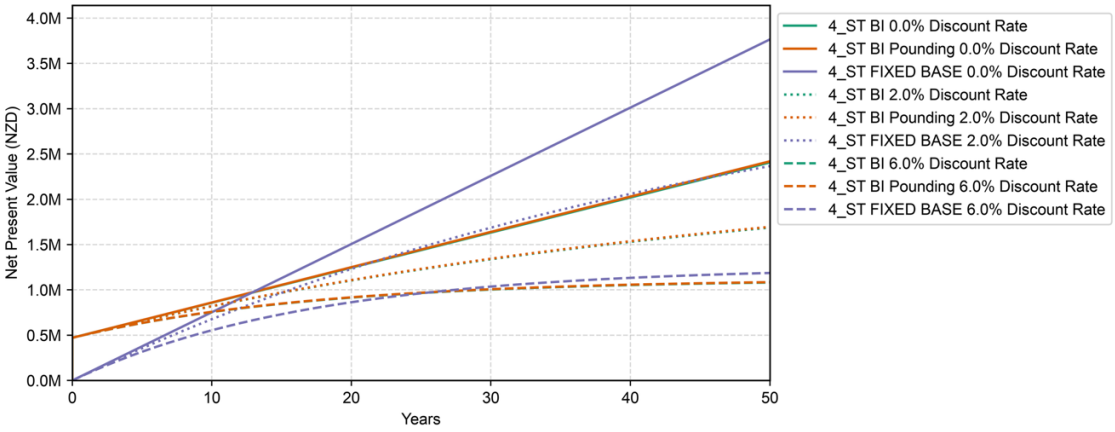


Figure 17: Cumulative annual cost of base-isolated vs fixed-base building for building 4-storey.

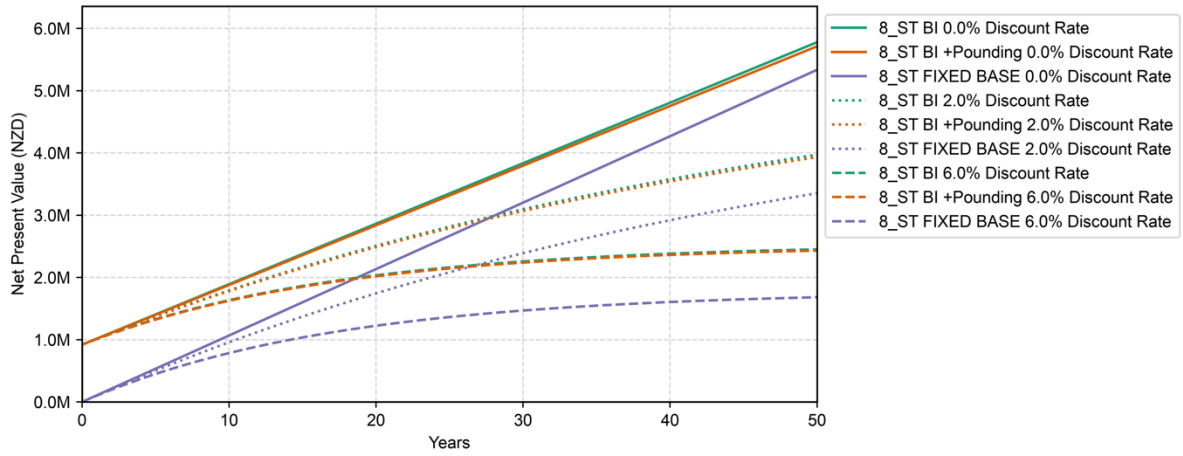


Figure 18: Cumulative annual cost of base-isolated vs fixed-base building for building 8-storey.

Table 6: Annualised loss and break even period summary for each test structure.

Building	Annualised Loss ($1,000s/year)			Break-even (yrs)		With 6% Discount	
	Fixed	BI	BI+Pound	BI	BI+Pound	BI	BI+Pound
2-storey	45	10	12	6.5	7.0	9	9
4-storey	75	39	39	12.9	13.0	22	22
8-storey	107	97	96	96.5	86.0	None	None
L-shaped	193	64	65	11.6	11.7	17.8	17.9

Conclusions

It is generally understood that base isolation can reduce earthquake-induced damage and improve post-earthquake functionality, but its adoption is often constrained by higher upfront construction costs. This project examined whether the higher initial cost of base isolation can be justified by reduced earthquake losses over the life of a building. The work produced two linked outputs: a New Zealand base-isolated building inventory database, and a comparative loss and lifetime-cost assessment for fixed-base and base-isolated case-study buildings.

Base-isolated building inventory

The database provides a structured record of base-isolated buildings identified in New Zealand. By the end of 2024, 67 base-isolated buildings had been identified, including completed buildings, buildings under construction, and buildings in design. The database records information on building location, use, status, structural system, isolation system, and key design characteristics where available.

The database shows that base isolation in New Zealand is concentrated in high-seismicity regions, particularly Wellington and Christchurch. It also shows that base isolation is no longer limited to hospitals or nationally significant public facilities. Commercial and residential buildings now form a substantial part of the identified inventory, indicating that the perceived value of base isolation has expanded beyond life-safety and emergency-response functions to include business continuity, repairability, and reduced disruption. This database provides an evidence base for understanding how base isolation has been adopted in New Zealand and gives future researchers, engineers, and policy agencies a stronger foundation for assessing the performance and risk profile of the existing base-isolated building stock.

Loss and lifetime-cost assessment

The building case-study loss analyses showed that base isolation can substantially reduce expected earthquake losses, but the size of the benefit depends strongly on the building configuration, the superstructure response, and the assumed initial cost premium.

For the 2-storey, 4-storey, and L-shaped buildings, base isolation produced clear reductions in expected annual loss. Using a base-isolation cost premium of approximately 3%, the simple break-even periods were approximately 7 years for the 2-storey building, 13 years for the 4-storey building, and 12 years for the L-shaped building. These results indicate that, for some low- to mid-rise buildings, the additional upfront cost of base isolation can be offset within a realistic building service-life timeframe.

The 8-storey building was the main exception. Although base isolation reduced losses at some intermediate intensity levels, the annualised saving was small, giving a much longer simple break-even period of approximately 85–96 years depending on whether pounding was included. Under discounted assumptions, no break-even occurred within the assessed time horizon. This result was not primarily caused by the 3% premium assumption, but by the relatively small reduction in expected annual loss. In this model, low-intensity upper-storey drift-sensitive steel connection losses reduced the economic benefit of the isolated configuration.

The effect of moat-wall pounding was visible in the structural response results, particularly at high intensities where acceleration demands increased. However, pounding had only a modest effect on expected annual direct loss in the current analysis because of the very low exceedance probability of events that will give rise to pounding. It should be noted, however, that the present loss model does not explicitly include moat-wall damage, isolator damage, foundation damage, isolation-plane repair, or downtime consequences, all of which could increase the practical significance of pounding.

The overall conclusion is that base isolation can provide a strong economic and resilience benefit where it produces consistent reductions in expected annual loss, but this benefit still necessitates proper tuning of the isolation system during design.

Research benefits and future directions

This project provides several benefits for natural hazard risk reduction, engineering practice, and future research in New Zealand. These are summarized below:

Research benefits

1) The project developed a structured inventory of base-isolated buildings in New Zealand. This provides a stronger basis for understanding what has been built and where future performance or risk assessment should focus.
2) The lifetime-cost assessment showed that base isolation can substantially reduce expected annual earthquake losses. These results help translate the benefits of base isolation into economic terms that are more useful for owners, engineers, insurers, and public agencies.
3) The project demonstrated that the additional upfront cost was offset by lower expected future earthquake losses within realistic timeframes provided the isolation system was well tuned.

4) The project developed an approach for modelling pounding effects explicitly in numerical models.
5) The analysis that includes pounding showed that impact into the moat has a minor effect on loss in the current model.

Capability development

The project supported student research, development of OpenSeesPy and Pelicun-based workflows, and collaboration between researchers and practising engineers. These outcomes contribute to New Zealand capability in low-damage seismic design, loss assessment, and resilience-based decision-making.

Relevance to NHC risk-reduction objectives

The findings are directly demonstrate the effect of base isolation on reducing future earthquake losses and improving post-earthquake functionality. Thereby, the project provides supporting information on how transition to low-damage technology could lower NZ's seismic risk exposure.

Future directions

In development of the building archetypes, the models and executing loss analysis, several assumptions were made. Refining these assumptions, and widening the scope of this work would increase the validity of the results. Specifically future work should:

1) repeat the loss and break-even assessment using the 2022 New Zealand National Seismic Hazard Model [20] and emerging TS 1170.5 hazard requirements.
2) assess a broader range of building types and design criteria to include a wider range of design criteria for the superstructure, building heights, structural systems, occupancy types, soil conditions, and isolation-system configurations.
3) include downtime, loss of function, business interruption, reoccupancy delays, and disruption to occupants or tenants in the loss analysis
4) include potential for isolation-plane, moat-wall, foundation, and isolator damage during pounding simulation
5) maintain and expand the base-isolated building database
6) consider mechanisms where insurance discounts could be used to incentivise adoption of base isolated technology.

Publications and research outputs

Output Type*	Date Published/ Completed	Author/s	Title	DOI or Link
Conference Paper	01/04/2025	Chen, T	An overview of the base isolated building inventory in New Zealand	(sent to NHC)
Conference Paper	01/04/2025	Wei, Y	Effect of impact model on the response of base-isolated building during moat wall pounding	(sent to NHC)
Conference Paper	01/09/2025	Wei, Y	Consequences of moat wall pounding in base-isolated buildings under bidirectional excitations	(sent to NHC)
Journal Paper	01/06/2026	Wei, Y., Shegay A., Henry R.,	Effect of impact models on the response of base-isolated 1 buildings during moat	(submitted to journal)
Presentation	20/03/2026	Shegay. A	Cost-effectiveness study of base isolation compared with conventional structures	(sent to NHC)

Communications and dissemination

Presentations:

Event*	Date	Presenter	Title
NZSEE Conference 2025	15/04/2025	Tina Chen	An overview of the base isolated building inventory in New Zealand
NZSEE Conference 2025	15/04/2025	Yuanyuan Wei	Effect of impact model on the response of base-isolated building during moat wall pounding
World Conference on Seismic Isolation	17/09/2026	Alex Shegay	Consequences of moat wall pounding in base-isolated buildings under bidirectional excitations
Japan-NZ Workshop on Cost-effective retrofit #1	17/01/2026	Alex Shegay	Investigation of effective and cost-efficient building retrofit strategies using modern technologies
Resilience Highlights Report	03/03/2026	NHC	Resilience Highlights Report 2025
Japan-NZ Workshop on Cost-effective retrofit #2	20/03/2026	Alex Shegay	Cost-effectiveness study of base isolation compared with conventional structures

Research team

Alex Shegay, *Senior Lecturer, University of Auckland*: Overall project coordination, student supervision, data analysis and interpretation.

Rosa Gonzalez, *Graduate Engineer, Holmes Consulting Ltd*: Numerical modelling, loss analysis and data compilation. Involved from 01 July 2025 to 01 Feb 2026.

Claire Dong, *Graduate Engineer, Holmes Consulting Ltd*: Development and sharing of building models. Involved throughout 2025 for quarterly meetings.

Student details

Name:	Tina Chen	**Status:**	Completed
Institution:	University of Auckland	**Primary Supervisor:**	Alex Shegay
Start Date:	01/03/2024	**End Date:**	01/11/2024
Degree Type:	Hons	**NHC Funded:**	Partially
Project Title:	Typical structural characteristics of base isolated buildings in New Zealand		

Name:	Cara Wang	**Status:**	Completed
Institution:	University of Auckland	**Primary Supervisor:**	Alex Shegay
Start Date:	01/03/2024	**End Date:**	01/11/2024
Degree Type:	Hons	**NHC Funded:**	Partially
Project Title:	Typical structural characteristics of base isolated buildings in New Zealand		

Name:	Yuanyuan Wei	**Status:**	In progress
Institution:	University of Auckland	**Primary Supervisor:**	Alex Shegay
Start Date:	01/09/2023	**End Date:**	01/09/2027
Degree Type:	PhD	**NHC Funded:**	No
Project Title:	Effect of moat-wall pounding on the response of base-isolated structures		

References

[1] Y. Nakamura and K. Okada, “Review on seismic isolation and response control methods of buildings in Japan,” *Geoenvironmental Disasters*, vol. 6, no. 1, p. 7, Jun. 2019, doi: 10.1186/s40677-019-0123-y.

[2] *Building Regulations 1992*. New Zealand: http://www.legislation.govt.nz/regulation/public/1992/0150/latest/DLM162570.html, 1992. [Online]. Available: http://www.legislation.govt.nz/regulation/public/1992/0150/latest/DLM162570.html

[3] NZS 1170.5:2004, *Structural Design Actions Part 5: Earthquake actions - New Zealand*. Wellington, New Zealand: Standards New Zealand, 2016.

[4] American Society of Civil Engineers, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Reston, VA: American Society of Civil Engineers, 2021. doi: 10.1061/9780784415788.

[5] New Zealand Society for Earthquake Engineering, *Guideline for the Design of Seismic Isolation Systems for Buildings (Draft for Trial use)*, 2019. [Online]. Available: https://www.nzsee.org.nz/wp-content/uploads/2019/06/2825-Seismic-Isolation-Guidelines-Digital.pdf

[6] FEMA, “FEMA P-58 - Seismic Performance Assessment of Buildings - Methodology,” FEMA, Washington D.C., U.S., 2012.

[7] A. Zsarnoczay *et al.*, “NHERI-SimCenter/pelicun: v3.3,” *Zenodo CERN Eur. Organ. Fpeor Nucl. Res.*, Mar. 2024, doi: 10.5281/zenodo.10896145.

[8] R. E. Gonzalez, “Building the Carbon Case for Resilient Design,” p, 2024.

[9] C. Dong, T. Sullivan, and D. Pettinga, “Investigating the impacts of design ductility values and importance levels on the performance of base-isolated buildings in New Zealand,” *Bull. N. Z. Soc. Earthq. Eng.*, vol. 58, no. 3, pp. 169–186, 2025.

[10] Y. Wei, A. Shegay, and R. Henry, “Effect of impact model on the response of base-isolated building during moat wall pounding,” in *Proceedings of the New Zealand Society for Earthquake Engineering (NZSEE) Annual Conference*, Auckland, New Zealand, 2025.

[11] A. Masroor and G. Mosqueda, “Seismic Response of Base Isolated Buildings Considering Pounding to Moat Walls,” Technical Report MCEER-13-0003, 2013.

[12] T. Z. Yeow, “Building-Specific Seismic Resilience Assessment Frameworks Considering Content Sliding and Injury,” Ph.D. Thesis, University of Canterbury, 2017.

[13] A. Kuang *et al.*, “Christchurch Women’s Hospital: Performance Analysis of the Base-Isolation System during the Series of Canterbury Earthquakes 2011–2012,” *J. Perform. Constr. Facil.*, vol. 30, no. 4, p. 04015096, 2016, doi: 10.1061/(ASCE)CF.1943-5509.0000846.

[14] S.-J. Wang, C.-H. Yu, S.-K. Peng, Y.-S. Yang, and T.-T. Kod, “Performance of Seismically Isolated Buildings in Taiwan During the ML 7.2 Hualien Earthquake on April 3, 2024,”

Earthq. Eng. Struct. Dyn., vol. n/a, no. n/a, 2026, doi: https://doi.org/10.1002/eqe.70189.

[15] A. Charleson and N. Allaf, “Costs of Base-isolation and Earthquake Insurance in New Zealand,” in *New Zealand Society for Earthquake Engineering Conference*, 2012.

[16] B. A. Bradley, R. P. Dhakal, M. Cubrinovski, G. A. MacRae, and D. S. Lee, “Seismic loss estimation for efficient decision making,” *Bull. N. Z. Soc. Earthq. Eng.*, vol. 42, no. 2, pp. 96–110, Jun. 2009, doi: 10.5459/bnzsee.42.2.96-110.

[17] N. Jayaram, N. Shome, and M. Rahnama, “Development of earthquake vulnerability functions for tall buildings,” *Earthq. Eng. Struct. Dyn.*, vol. 41, no. 11, pp. 1495–1514, Sep. 2012, doi: 10.1002/eqe.2231.

[18] T. Chen and A. Shegay, “An overview of the base isolated building inventory in new zealand,” in *Proceedings of the New Zealand Society for Earthquake Engineering (NZSEE) Annual Conference*, Auckland, New Zealand, 2025.

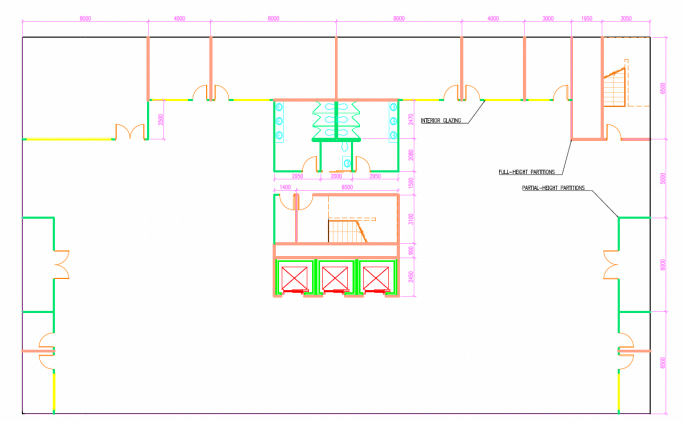
[19] Insurance Council of New Zealand, “Why insurers prefer base-isolated structures with stable foundations for commercial buildings in earthquake zones,” 2019.

[20] M. Gerstenberger *et al.*, “New Zealand National Seismic Hazard Model 2022 revision : model, hazard and process overview,” GNS Science, GNS Science Report 2022/57, 2022. doi: http://doi.org/10.21420/TB83-7X19.

Appendix 1: Components in loss analysis

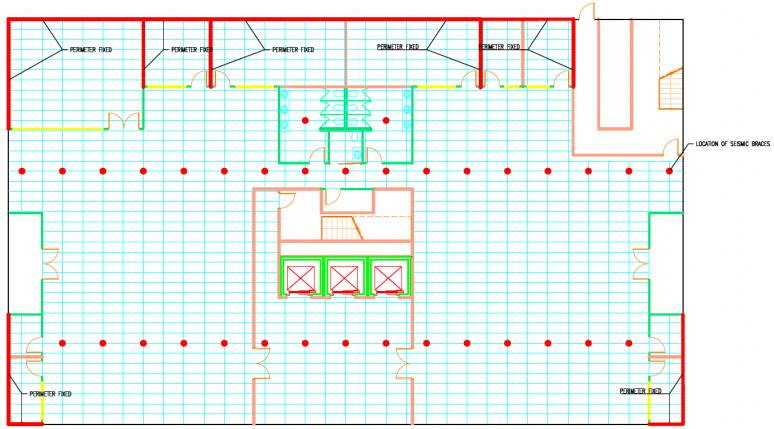
The table below summarizes the components (structural and non-structural) considered in the FEMA P-58 loss-analysis. The components are identical for fixed base and base-isolated buildings.

FEMA P-58 Code	Description	Theta_0	Units	Location	Direction
B.10.35.050	Traditional steel joints<= W27 one-sided	4	ea	1,2	1,2
B.10.35.051	Traditional steel joints<= W27 two-sided	12	ea	1,2	1,2
B.10.35.054	Steel column base	10	ea	1	1,2
C.10.11.050	NS_PART_FULL_Height	14	ea	1,2	1
C.10.11.050	NS_PART_FULL_Height	35	ea	1,2	2
C.10.11.051	NS_PART_PART_Height	18	ea	1,2	1
C.10.11.051	NS_PART_PART_Height	35	ea	1,2	2
C.10.11.052	NS_PART_INTERIOR_GLAZING	5	ea	1,2	2
C.10.11.052	NS_PART_INTERIOR_GLAZING	17	ea	1,2	1
C.10.11.053	NS_CLAD_GLAZING	104	ea	1,2	1
C.10.11.054	NS_CLAD_PRECAST	12	ea	1,2	2
C.20.11.050	NS_STAIRS	2	ea	1,2	1,2
C.30.32.050	NS_CEILING_4t2p5	2	ea	1,2	0
C.30.32.051	NS_CEILING_4t3	2	ea	1,2	0
C.30.32.052	NS_CEILING_4t4	2	ea	1,2	0
C.30.32.053	NS_CEILING_4t8	2	ea	1,2	0
C.30.32.054	NS_CEILING_6p5t2	2	ea	1,2	0
C.30.32.055	NS_CEILING_6p5t8	2	ea	1,2	0
C.30.32.056	NS_CEILING_BRACE	35	ea	1,2	0
D.10.14.050	NS_ELEVATOR	3	ea	1	0
D.20.21.050	NS_WATER_PIPE	2	ea	1,2	0
D.20.21.051	NS_SANITARY_PIPE	2	ea	1,2	0
D.30.41.050	NS_CHILLER_100P350	2	ea	1	0
D.30.41.052	NS_TOWER_100P350	2	ea	1	0
D.30.41.054	NS_SECTION_HVAC	2	ea	1,2	0
D.30.41.055	NS_DROPS_DIFFUSERS_HVAC	11	ea	1,2	0
D.30.41.056	NS_COILS_HVAC	11	ea	1,2	0
D.30.41.057	NS_BOXES_HVAC	5	ea	1,2	0
D.30.41.058	NS_AVA_1T2P5cfm	2	ea	1	0
D.30.41.059	NS_AVA_2p5kT40kcfm	2	ea	1	0
D.40.11.050	NS_DROPS_FIRE	11	ea	1,2	0
D.40.11.051	NS_PIPES_FIRE	2	ea	1,2	0
D.50.11.050	NS_TRANSFORMERS_100P350	2	ea	1	0



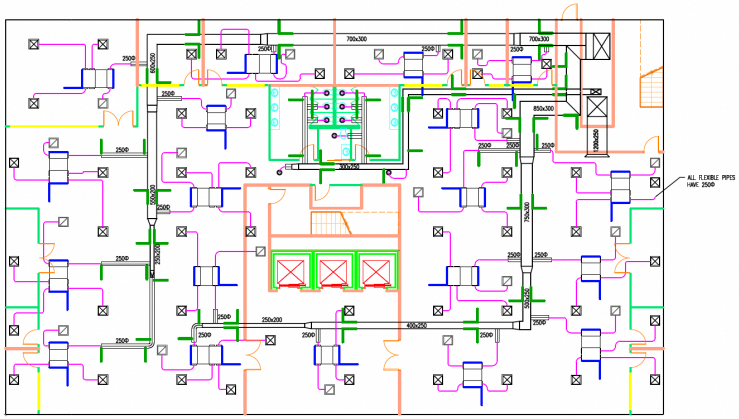
ROOM AND PARTITION LAYOUT FOR UPPER FLOORS
(SCALE 1:200, UNITS IN MM)

(a) Partition wall layout



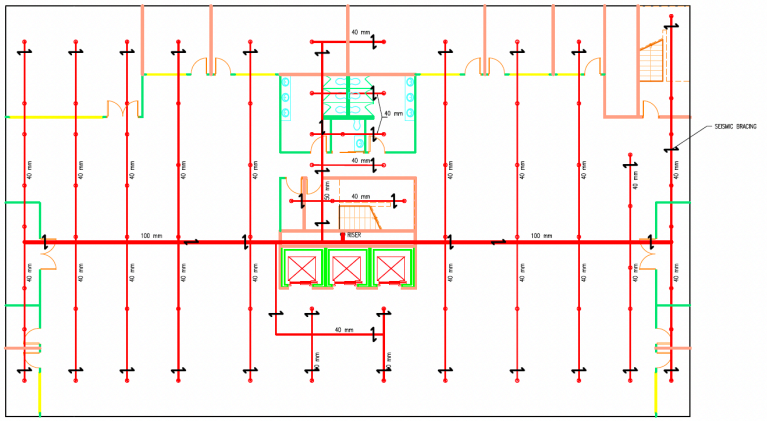
CEILING LAYOUT FOR GROUND FLOOR
(SCALE 1:200, UNITS IN MM)

(b) Ceiling layout



HVAC LAYOUT FOR GROUND FLOOR
(SCALE 1:200, UNITS IN MM)

(c) HVAC layout



SPRINKLER LAYOUT FOR UPPER FLOORS
(SCALE 1:200, UNITS IN MM)

(d) Fire sprinkler layout

Figure 19: Typical NSC layout quantities assumed in a building [12].

Appendix 2: Engineering Demand Parameters

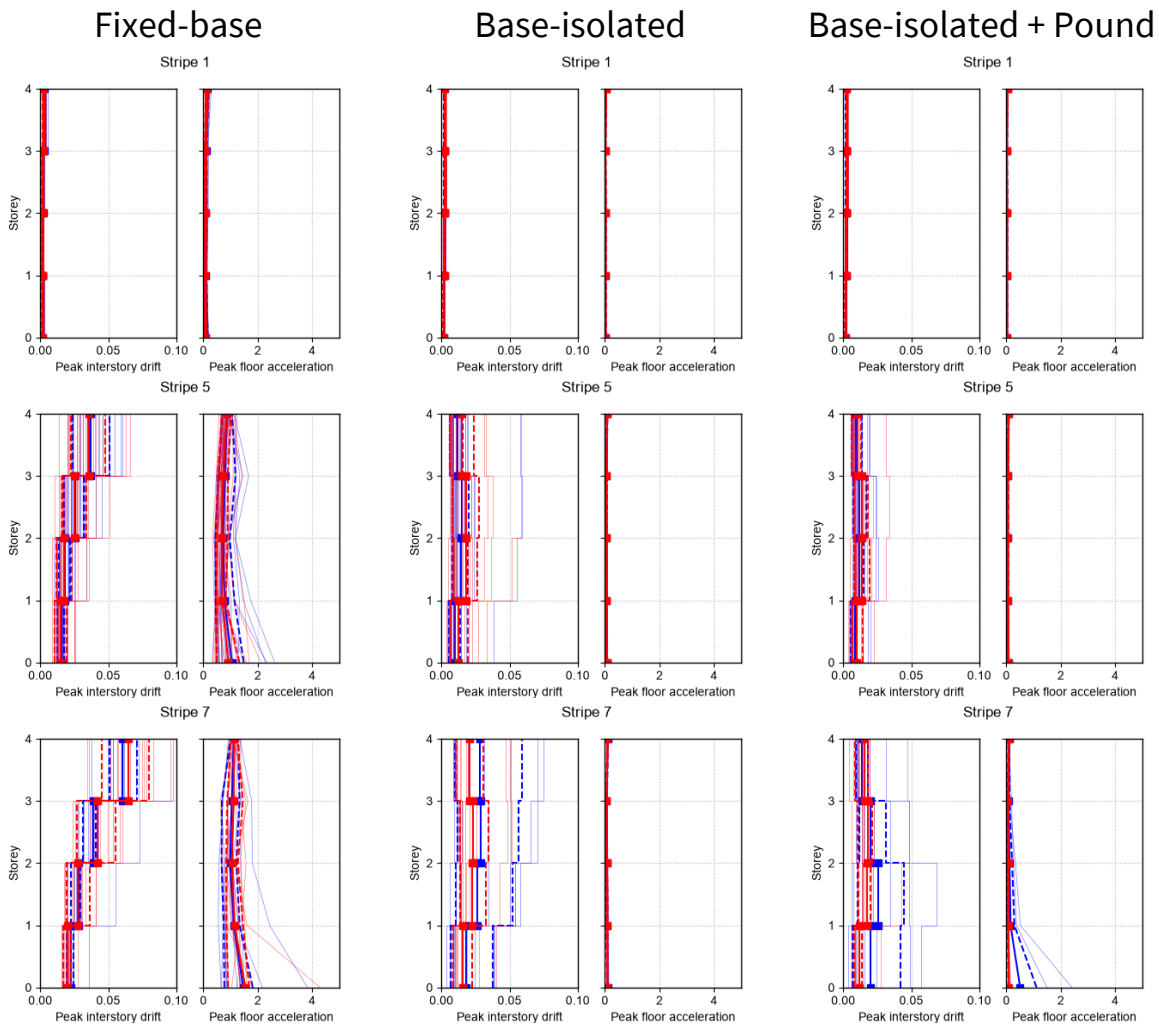


Figure 20: Mean of peak storey drift and floor acceleration distributions up the height for 4-storey building for SLS (left) ULS (middle) and MCE (right) demands. (blue and red correspond to two orthogonal directions, while dashed lines indicate ±1 standard deviation)

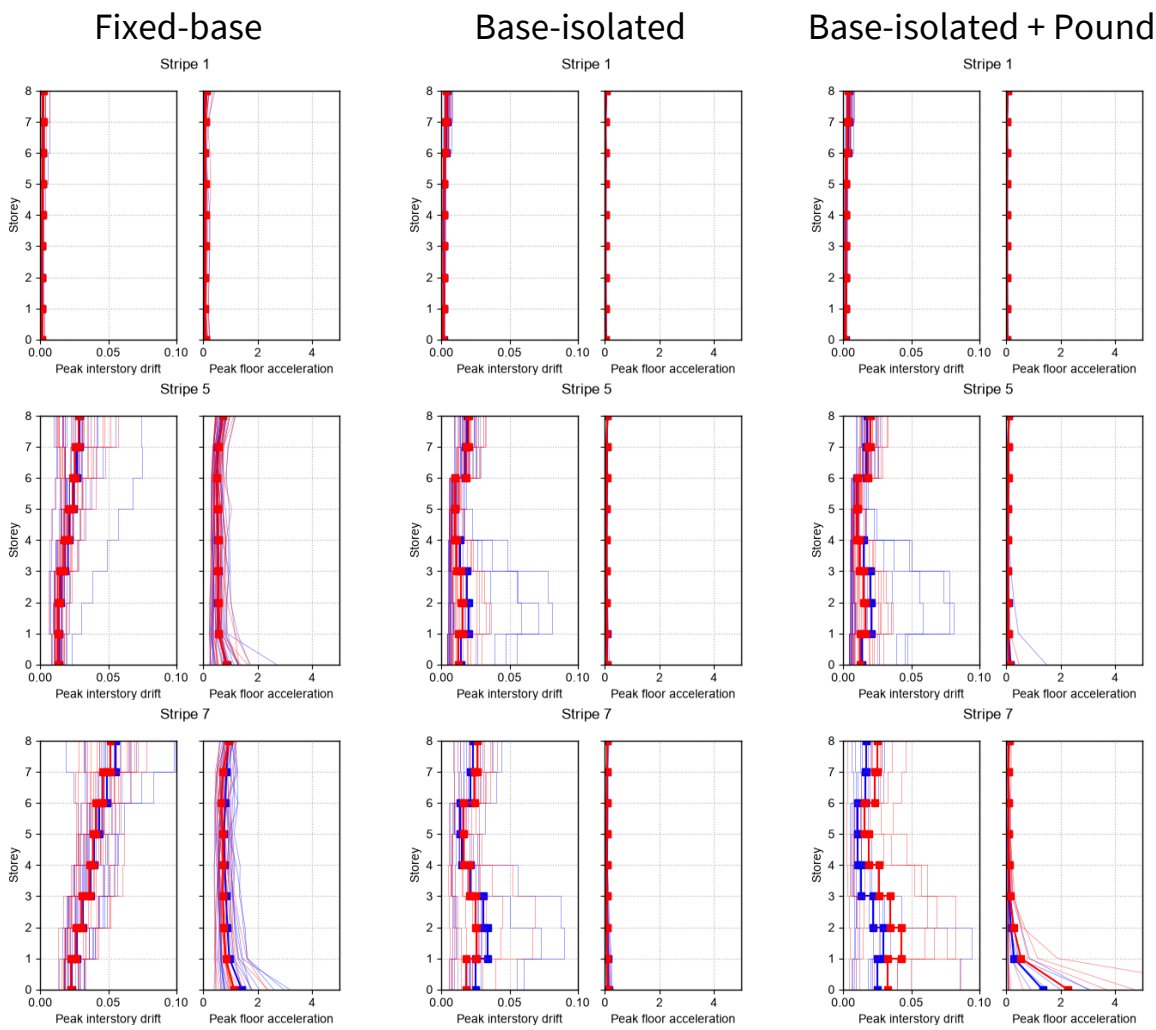


Figure 21: Mean of peak storey drift and floor acceleration distributions up the height for 8-storey building for SLS (left) ULS (middle) and MCE (right) demands. (blue and red correspond to two orthogonal directions, while dashed lines indicate ±1 standard deviation)

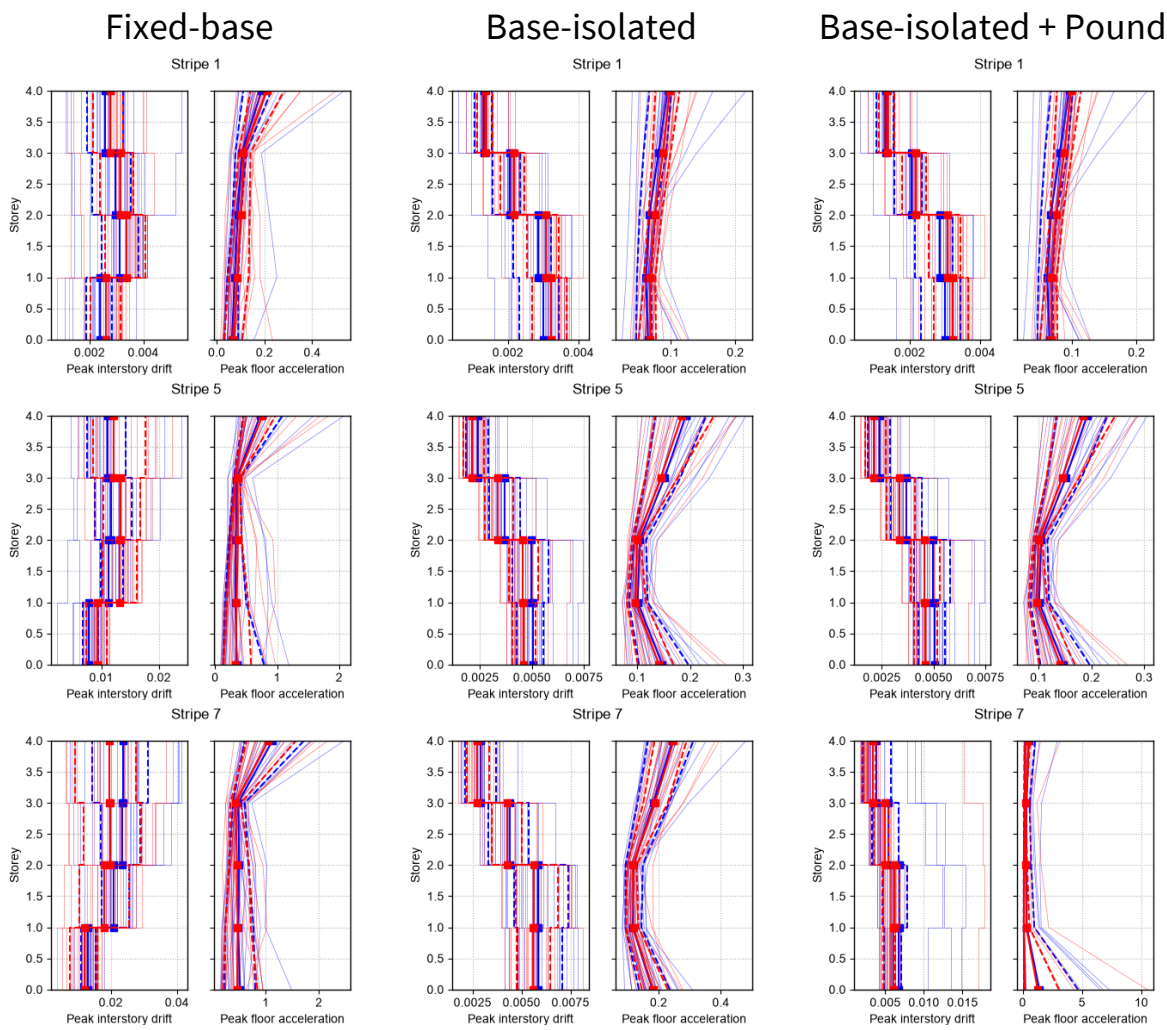
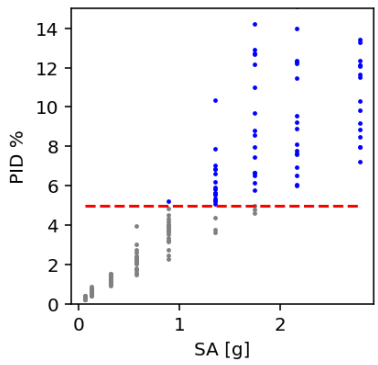


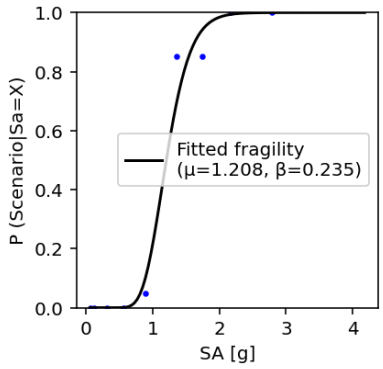
Figure 22: Mean of peak storey drift and floor acceleration distributions up the height for the L-shaped building for SLS (left) ULS (middle) and MCE (right) demands. (blue and red correspond to two orthogonal directions, while dashed lines indicate ±1 standard deviation)

Appendix 3: Collapse cases

2-storey

Fixed

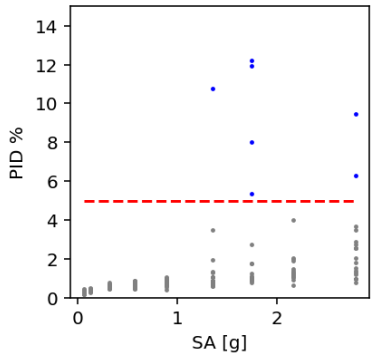


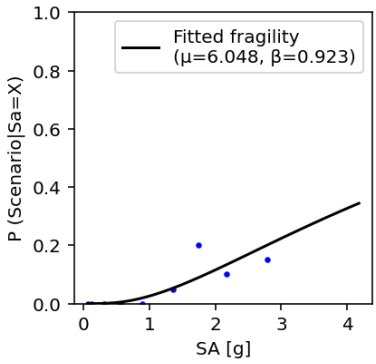


2-storey

Base

Isolated





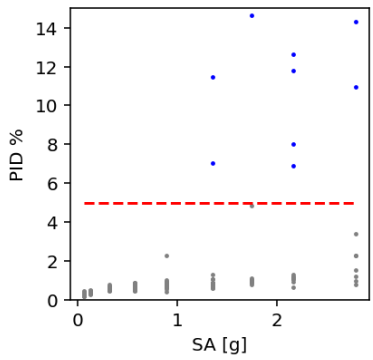
2-storey

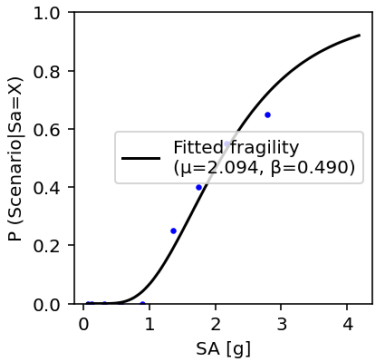
Base

Isolated

+

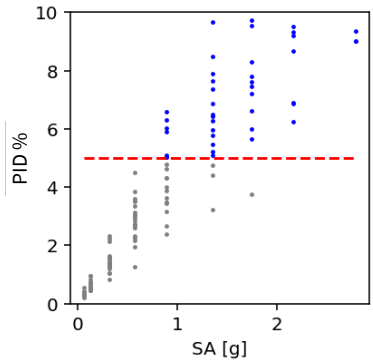
Pounding

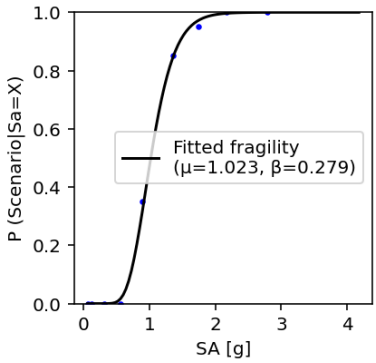




4-storey

Fixed

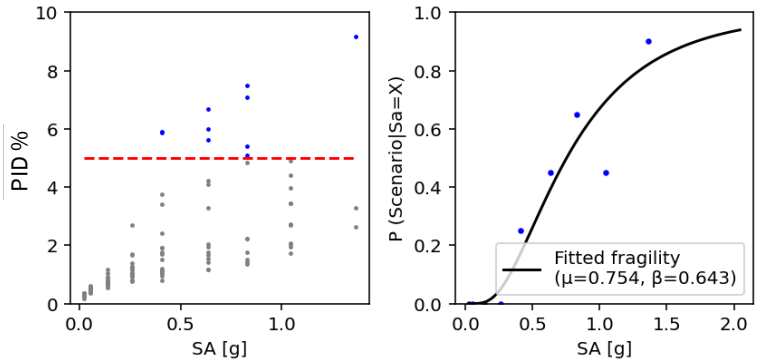




4-storey

Base

Isolated



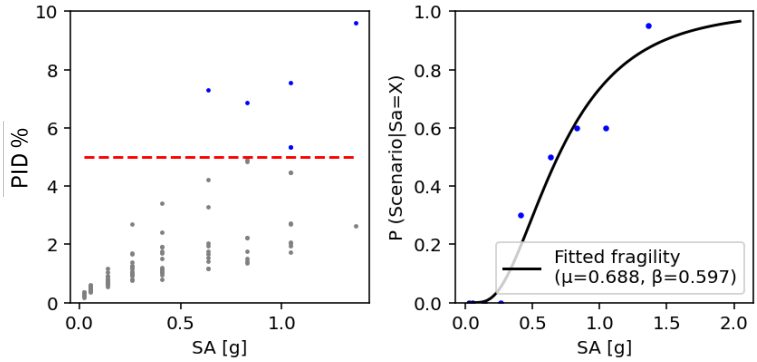
4-storey

Base

Isolated

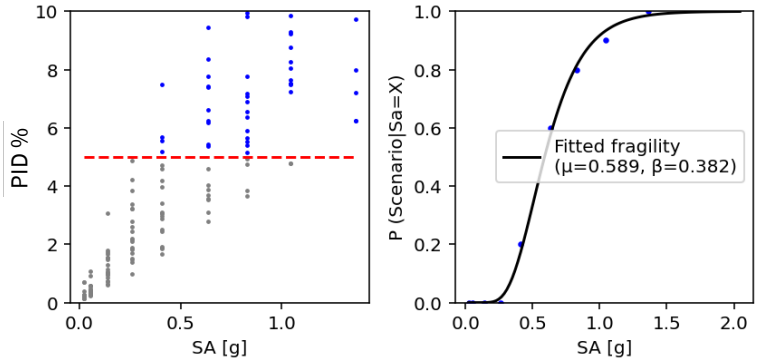
+

Pounding



8-store

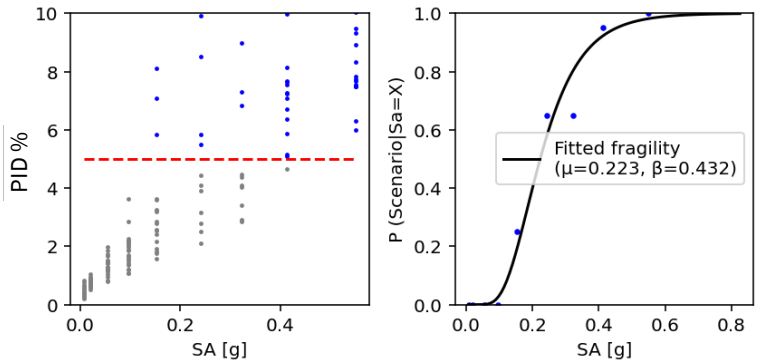
Fixed



8- storey

Base

Isolated



8-storey
Base
Isolated
+
Pounding

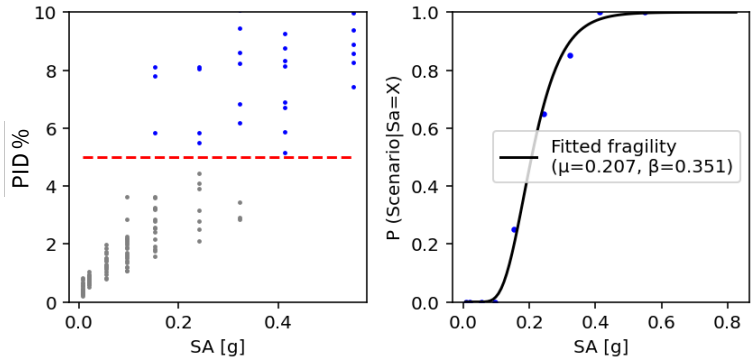


Figure 23: Peak storey drift achieved for each ground motion, and associated collapse fragility

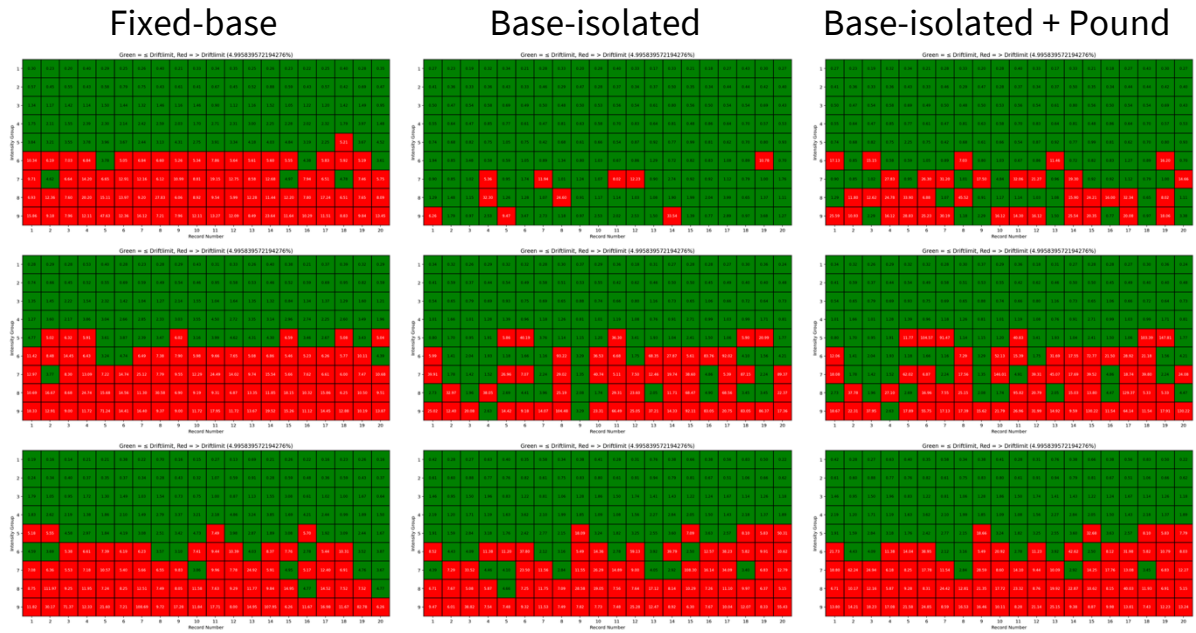
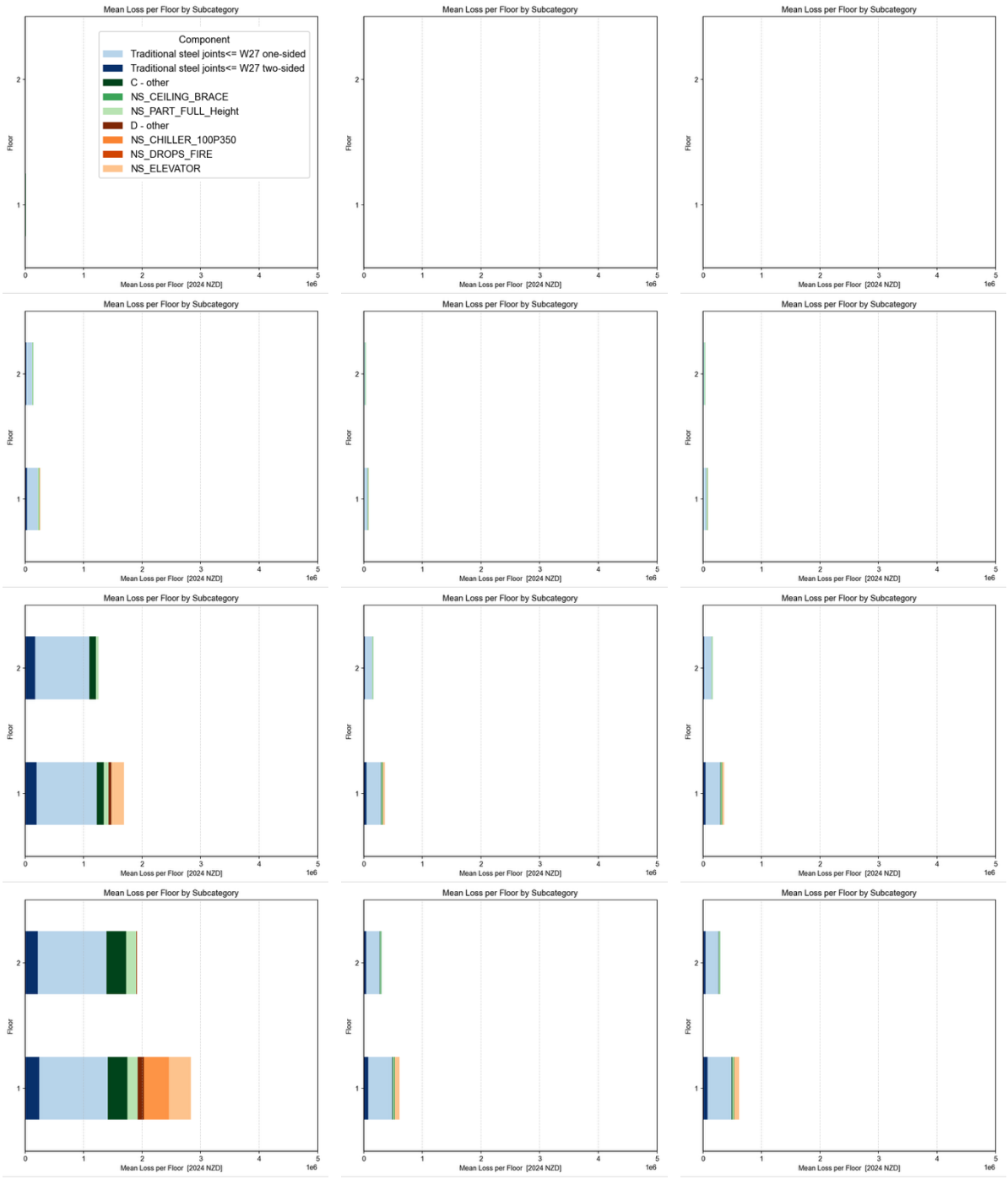


Figure 24: Peak drift for each ground motion in the analysis suite. Green cells indicate drift below 5%, while red cells indicate where drift exceeds 5% (i.e., collapse case). The top, middle and bottom row correspond to the 2-storey, 4-storey and 8-storey buildings.

Appendix 4: Component loss disaggregation

Fixed-base | Base-isolated | Base-isolated + Pound



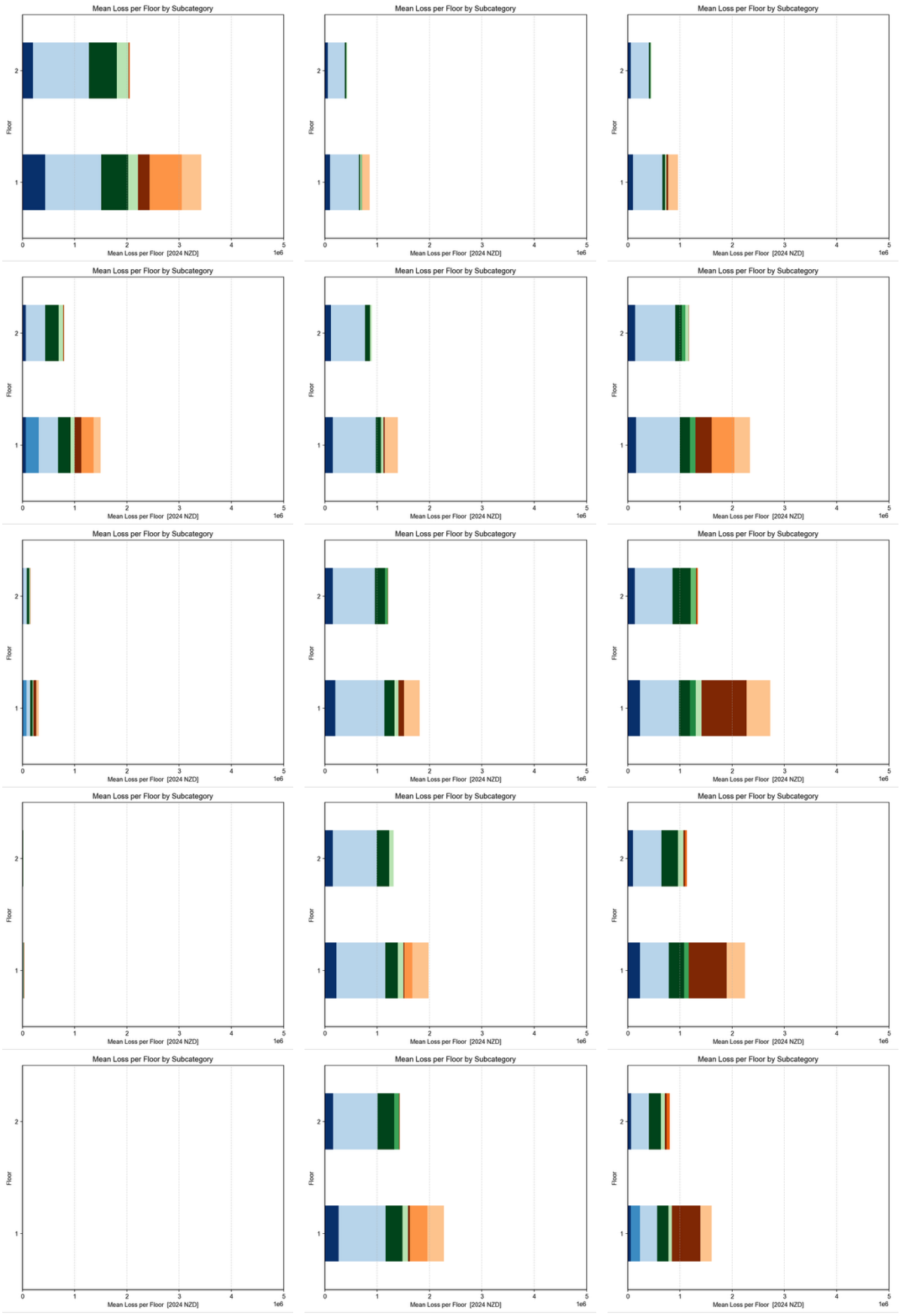
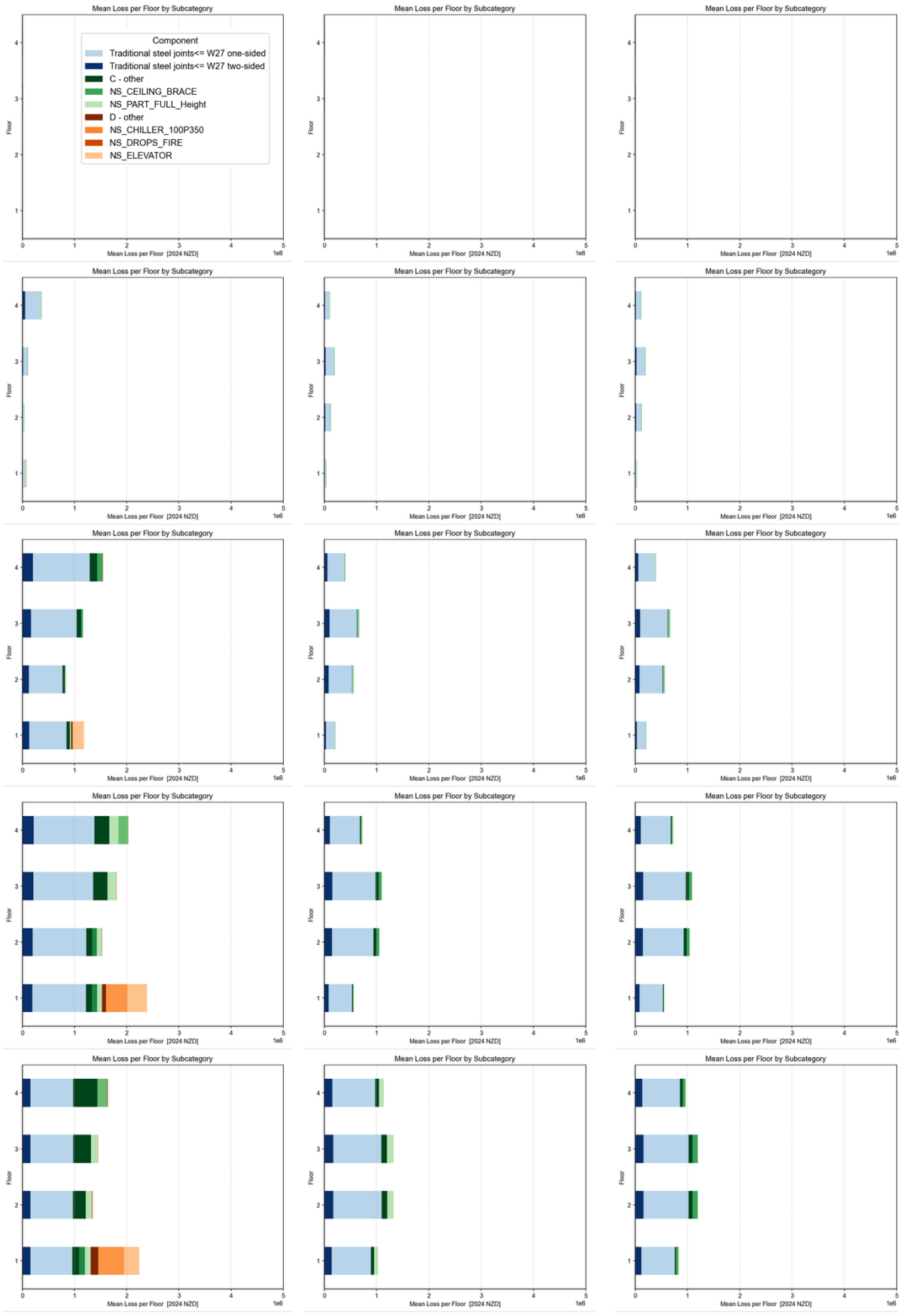


Figure 25: Expected loss for case 2-storey categorised by component Group and by floor.

Fixed-base | Base-isolated | Base-isolated + Pound



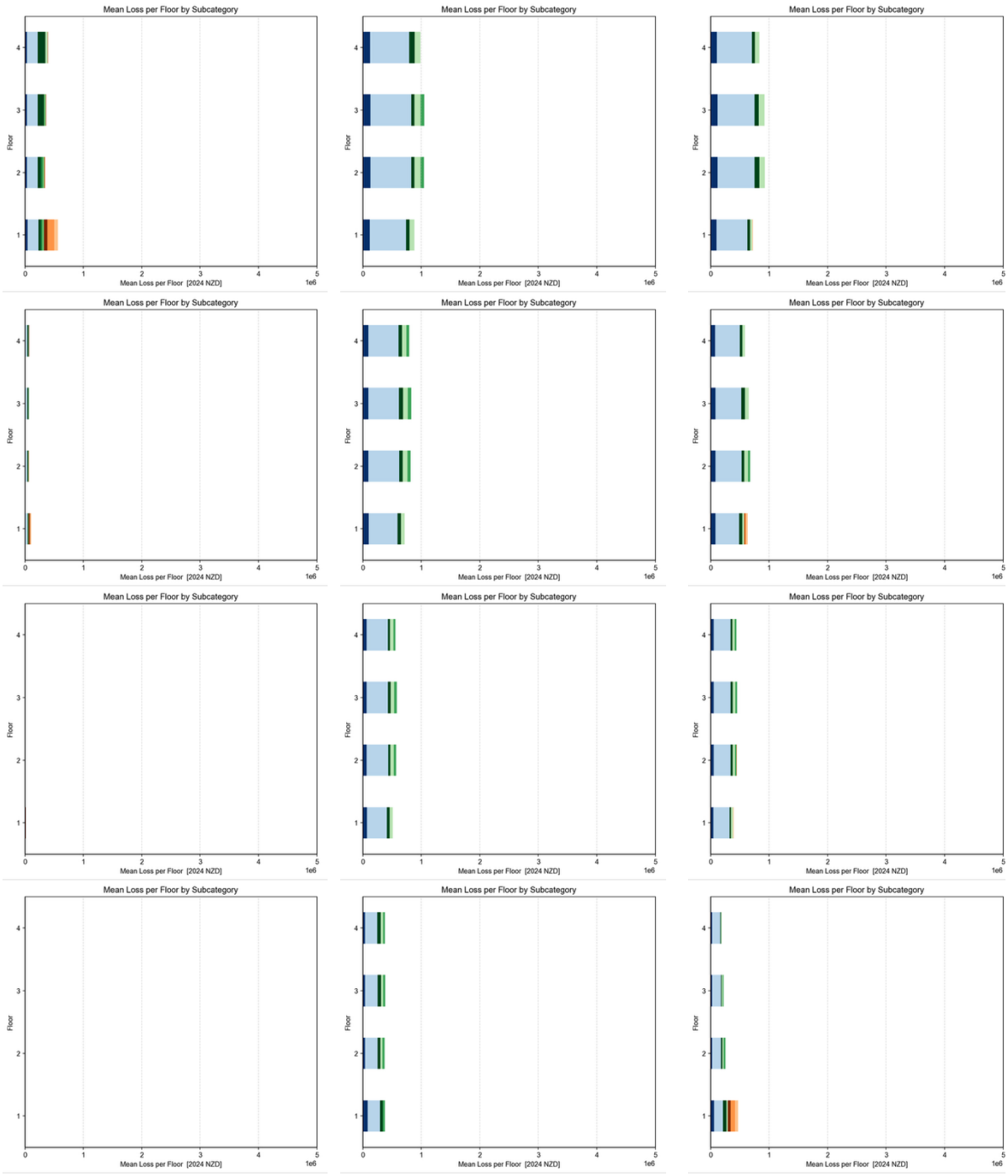


Figure 26: Expected loss for case 4-storey categorised by component Group and by floor.

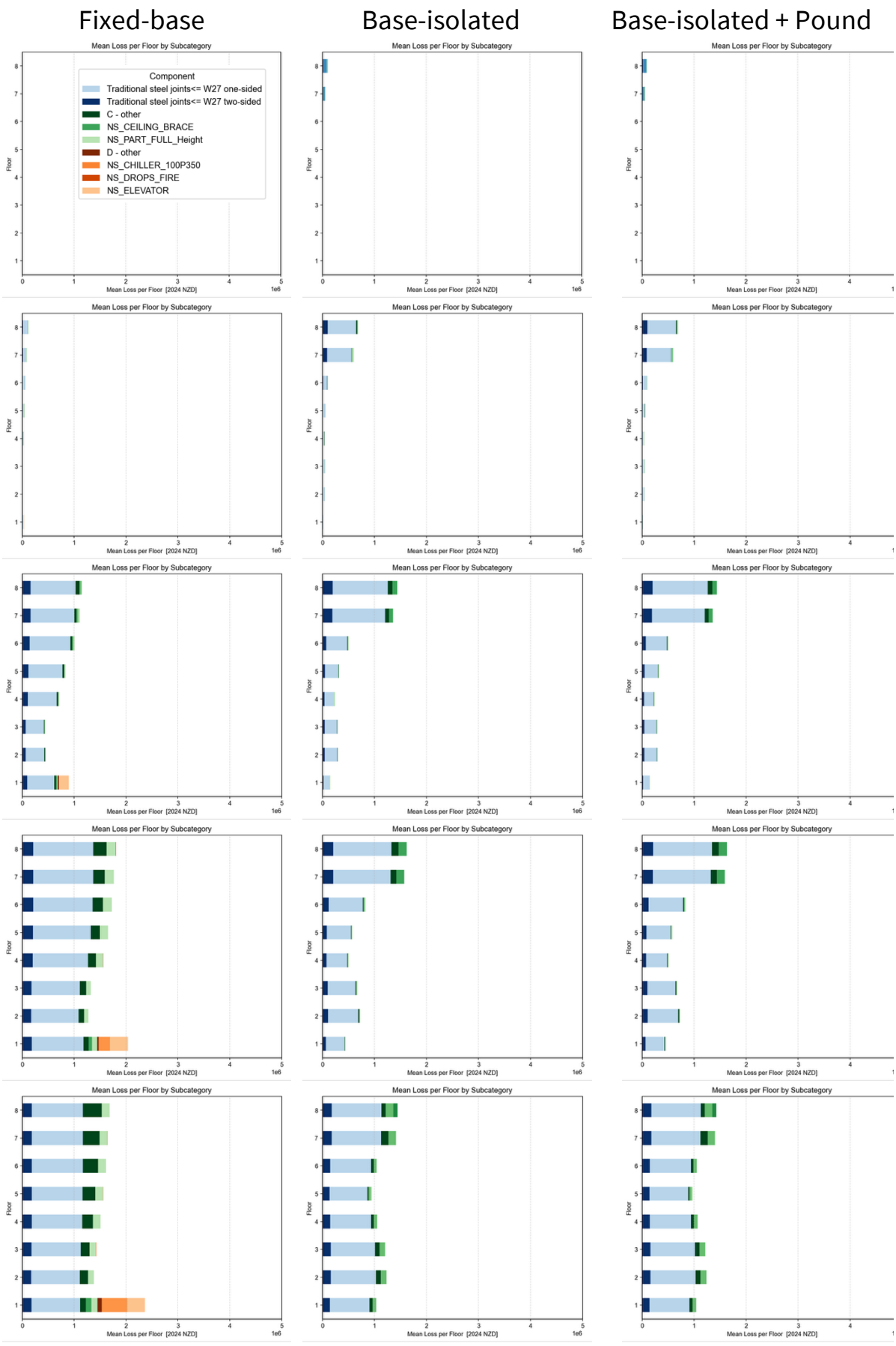
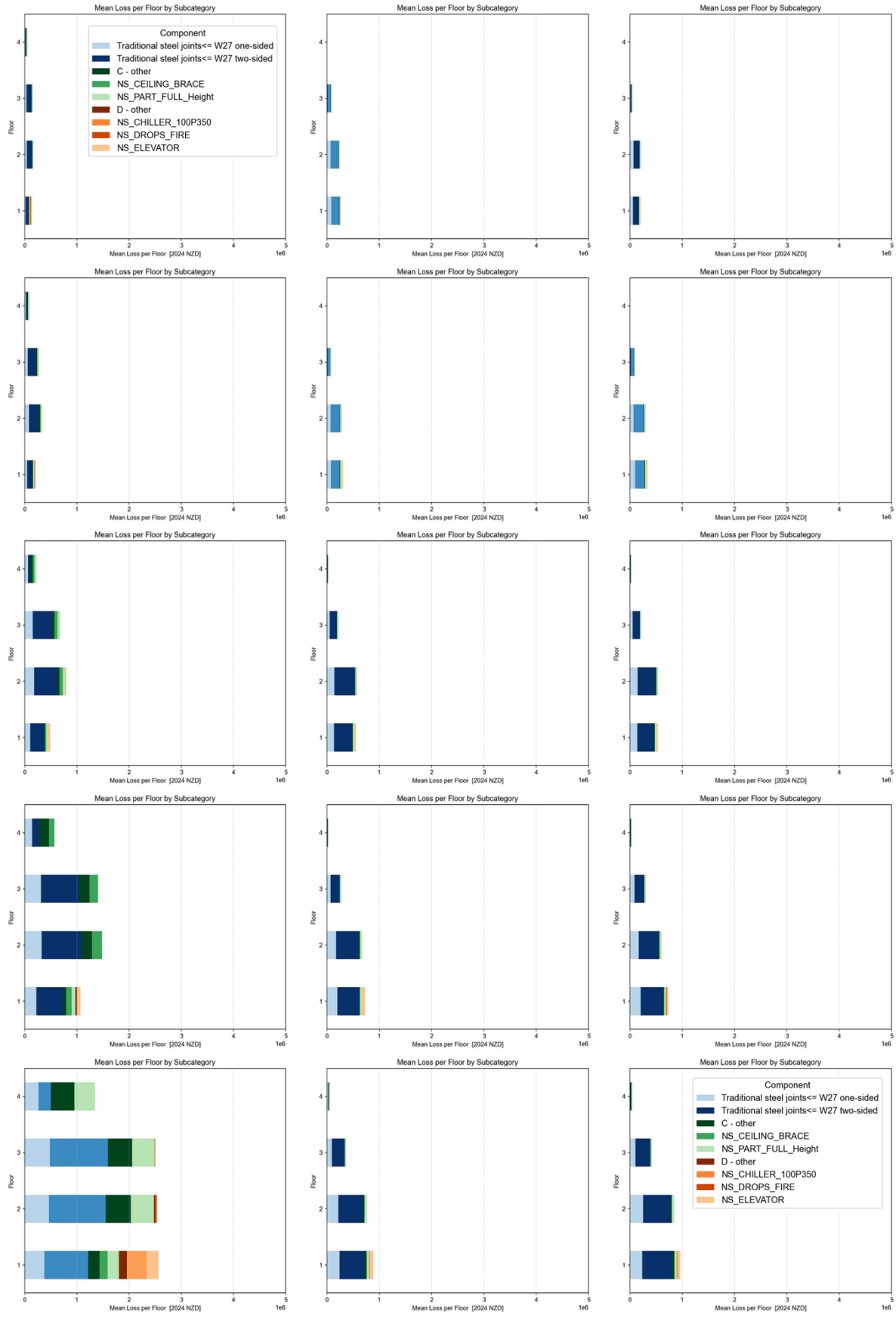

Fixed-base
Base-isolated
Base-isolated + Pound
Mean Loss per Floor by Subcategory
Component
Traditional steel joints<= W27 one-sided
Traditional steel joints<= W27 two-sided
C - other
NS_CEILING_BRACE
NS_PART_FULL_Height
D - other
NS_CHILLER_100P350
NS_DROPS_FIRE
NS_ELEVATOR
Floor
Mean Loss per Floor [2024 NZD]
1e6



Figure 27: Expected loss for case 8-storey categorised by component Group and by floor.

Fixed-base | Base-isolated | Base-isolated + Pound



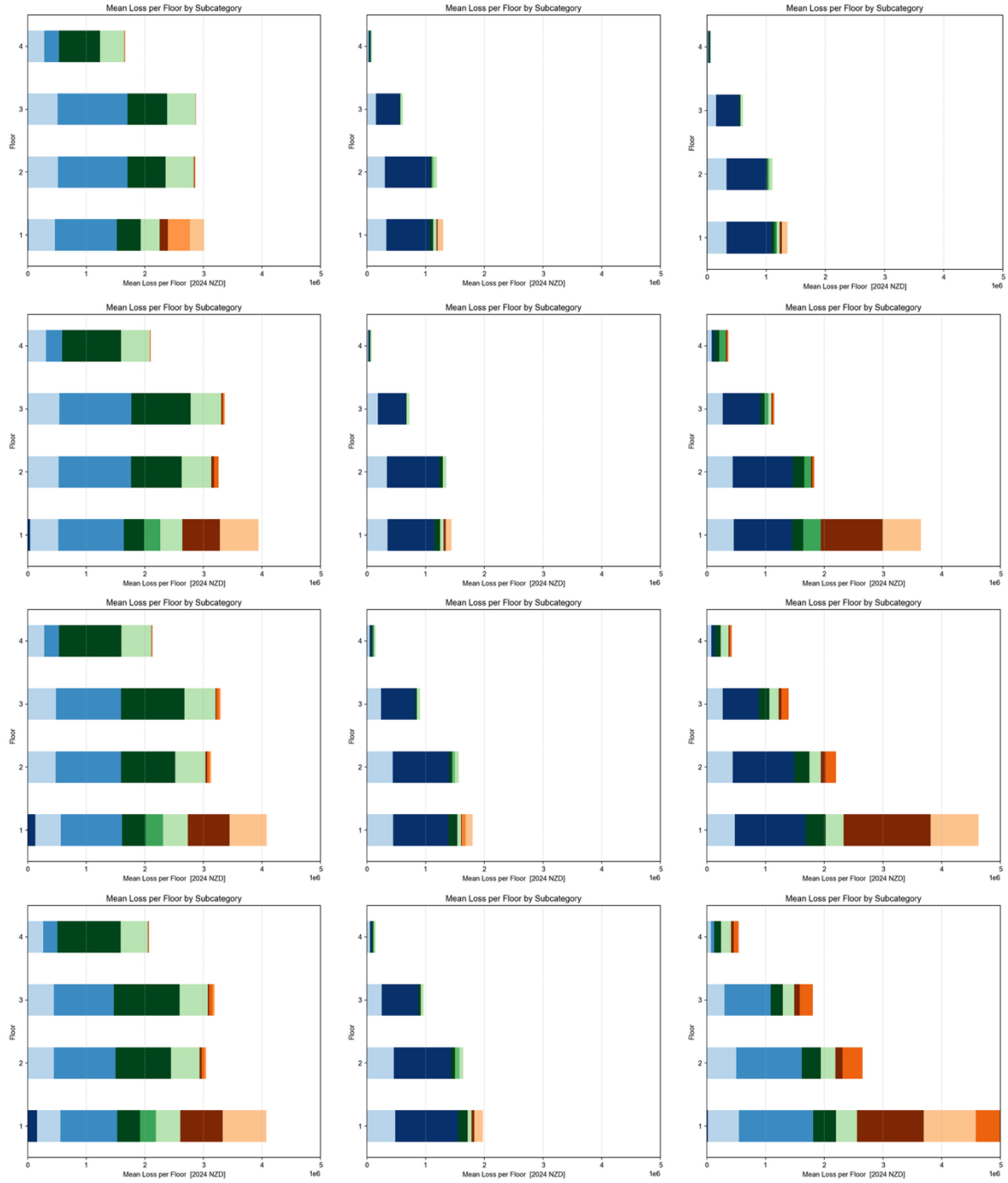


Figure 28: Expected loss for the L-shaped building categorised by component Group and by floor.

Appendix 5: Base-isolated building database of New Zealand

Base Isolated Buildings New Zealand, Database

Number of Base Isolation Buildings 65

	Built Building			General information										Design Characteristics					Superstructure Characteristics														Isolator Informaion		
	Building Name	Alternative Name	Image	Consultants	Location	Address	Coordinate		Type	Year of Completion	Year of Base Isolation	Building Use	Number of storeys		Important Level	$K_{H,super}$	Soil Class	Superstructure Period	Base Shear	Structural System	Floor System	Foundation System	Seismic Weight (kN)	First Storey Height (m)	Typical Storey Height (m)	Total Height (m)	Max plan dimension (m)	Footprint area (m^2)	Span Width (m)		Steel		Concrete		Isolator Type	Most Size (mm)	Isolator Period (s)
							Latitude	Longitude					Above Grade	Below Grade															EW	NS	Primary Beam size	Primary Column Size	Primary Beam Size	Primary Colume Size			
	Wellington Regional Hospital	NA		Dunning Thornton	Wellington	49 Riddiford Street, Newtown, Wellington 6021	-41.3089	174.7801	Redevelopment	2002	2008	Hospital	7	1	4		C		0.11g (ULS) 0.21g (MCE)	two ways RC frame	Rib and Timber	Ground beam with pile		4.5	4.2	31.1	147.6	12880	7.2	7.2	-	-	600sq	600sq & 1000x800	LRB, PTFE sliders	600	
	Wellington hospital Emergency Department	NA		Dunning Thornton	Wellington	49 Riddiford Street, Newtown, Wellington 6021	-41.3099635	174.7797315		1999		Hospital	2	0	4					RC frames and steel flat tension only braced frame	Hollowcore													LRB, Slider	375-600	
?	Wellington hospital Acute Assessment Unit	Short Stay /ED Observation Unit			Wellington	49 Riddiford Street, Newtown, Wellington 6021	-41.3097207	174.7794777		2003		Hospital	1	0	4					RC frames (gravity) Steel flat tension only braced frame (lateral)	Rib and Timber													LRB, Slider	375-600	
	Ministry of Health Building	William Clayton		Beca	Wellington	133 Molesworth Street, Thorndon, Wellington 6011	-41.272853254	174.7776948	New	1981	2015	Commercial	5	1	3		C	0.3		RC frame and 2 way steel MRF at top two level	Flat Slab, Rib and Timber (top 2 level)	RC beam and RC pile		4	4	24.5	97	3900	7.2	7.2	460UB67	610x11.1CHS		800sq	LRB	150 (1981) 400 (2015)	2.5
	Parliament House	NA		Holmes	Wellington	1 Museum Street, Pipitea, Wellington 6160	-41.27767765	174.7764103	Retrofit	1912	1994	Public	4	1			D	0.45		Masonry														LRB, HDRB, Sliders	400	2.5
	Parliament library (West Wing)	NA		Holmes	Wellington	Pipitea, Wellington 6011	-41.27696578	174.7765166	Retrofit	1899	1994	Public					D			Masonry														HDRB, LRB, FS	550	
	Parliament library (East Wing)	NA		Holmes	Wellington	Pipitea, Wellington 6011	-41.27696578	174.7765166	Retrofit	1899	1994	Public					D			Masonry														HDRB, LRB, FS	550	
	Te Papa National Museum	NA		Holmes	Wellington	55 Cable Street, Te Aro, Wellington 6011	-41.290456	174.782089	New	1865	1998	Public	5	1			D			RC frame			64000			23		19760							LRB, Pot Sliders	400	2.5
	Rankine Brown Library	Victoria University of Wellington Kelburn Library		Sinclair Knight Merz	Wellington	21 Kelburn Parade, Kelburn, Wellington 6012	-41.25012275	174.7696889	Retrofit	1960	2002	Public	8	2		3	C	$T_{1x,x} = 1.4$ $T_{1x,y} = 1.8$		RC frame-core wall structure	precast concrete waffle slab	Raft				69.54	2127	8.001	12.624	-	-		0.914m square	LRB, Teflonn slider	600	
	Hutt Hospital Emergency Department	NA		Aurecon	Lower Hutt	ED and Theatre Block, Boulcott, Lower Hutt 5011	-41.20440032	174.9234711	New	2012	2012	Hospital	3	0	4	2	D			RC moment frames, steel frames in the top floor	Rib and Timber	Grillage beam with piles		4.267		16.16		3417	7.5	7.8	-	-	650x800	550sq	LRB, slider	500	
	Hutt Hospital Orthopaedics	NA		Clendon Burns and Park	Lower Hutt	Hutt Hospital, Boulcott, Lower Hutt 5010	-41.20471709	174.923378	New	1998	1998	Hospital	2	0	4		D	0.42	0.65g (MCE)	RC frame	precast prestressed planks with an in-situ topping	Pile		4.267	3.575	14.9	37.5	1410	7.5	7.5	-	-	700x450	450sq	LRB	350	1.5
	Old Bank Arcade	Old BNZ		Holmes	Wellington	233-237 Lambton Quay, Wellington Central, Wellington 6011	-41.2858	174.7763	Retrofit	1901	1997	Commercial					C																	LRB, RB		
	PWC Centre Site 10	NA		Dunning Thornton	Wellington	PwC Centre 10 Waterloo Quay, Pipitea, Wellington 6011	-41.28149975	174.7796764	New	2018	2018	Commercial	5	1	2	1	D			CBF (transverse). Longitudinal: RC frame (L1-L3), perimeter steel truss (L4-L5)	TT	Pile	66000	4.4	3.85	23.66	51.5	710	7.5	7.6	410UB60	356x6.4 CHS Conc.Filled	-	-	LRB, PTFE	650	
	XXCQ Tower	NA		Dunning Thornton	Wellington	20 Customhouse Quay, Wellington Central, Wellington 6011	-41.28268068	174.7781002	New	2018	2018	Commercial	14	2	2	1	C			Steel perimeter diagrid	Comflor	Pile	135814kN	6.05	3.9	60.8	49.18	1570	8.4	8.25	610UB125	457x12.7 CHS conc. filled	-	-	LRB	650	
	MPI Nat. Bio. Laboratory	NA		Dunning Thornton	Upper Hutt	66 Ward Street, Waitacuvilla, Upper Hutt 5018	-41.13244658	175.0556408	New	2019	2019	Commercial	4	1	4	1.25	D	0.3		Concentric steel chevron bracing	in-situ concrete flat slab	shallow pad foundations with grid of RC beams	39184	4.9	4.9	16.4	45.3	1290	8.25	6.6	320C45 Fibrecased Beam	8x400x400 conc. filled steel	-	-	LRB	900	2.2 (MCE)
	Bob Scott Retirement Village (B04)	NA		Mitchell Vranjes	Lower Hutt	25 Graham Street, Petone, Lower Hutt 5012	-41.22154523	174.8945175	New	2017	2017	Residential	4	1	2	1.25	D			RC Shear Wall with steel beam and column	Rib and Timber	Raft		3.1	3.1	12	125	4900	-	-	460UB section	100x9.0SHS	-	-	LRB, PTFE	700	
	Bob Scott Retirement Village (B03)	NA		Mitchell Vranjes	Lower Hutt	25 Graham Street, Petone, Lower Hutt 5012	-41.22154523	174.8945175	New	2017	2017	Residential	3	1	2	1.25	D	0.7		RC Shear Wall with steel beam and column	Rib and Timber	Raft	59808	3.1	3.1	9	97	2088	-	-	460UB67	89x6.0SHS	-	-	LRB, PTFE	700	2.53
	Bob Scott Retirement Village (B02)	NA		Mitchell Vranjes	Lower Hutt	25 Graham Street, Petone, Lower Hutt 5012	-41.22154523	174.8945175	New	2017	2017	Residential	3	1	2	1.25	D	0.7		RC Shear Wall with steel beam and column	Rib and Timber	Raft	61761	3.1	3.1	9	76.5	2150	-	-	310UC section	125x9, 89x6 SHS	-	-	LRB, PTFE	700	2.57
	Bob Scott Retirement Village (B01)	NA		Mitchell Vranjes	Lower Hutt	25 Graham Street, Petone, Lower Hutt 5012	-41.22154523	174.8945175	New	2017	2017	Residential	5	1	2	1.25	D			RC Shear Wall	Rib and Timber	Raft	154560	3.4	3.2	13	98	4450	-	-	-	-	-	600 x 600	LRB, PTFE	700	2.57
	Christchurch Women's Hospital	NA		Holmes	Christchurch	2 Riccarton Avenue, Christchurch Central City, Christchurch 8011	-43.53413212	172.6255761	New	2005	2005	Hospital	9		4			1.2		Perimeter RCMF with MF and partial height steel K-braced frames to the core of the building		Raft				33			9	11.5					LRB, slider	420	
	Christchurch Hospital Acute Services Building	NA		Holmes	Christchurch	3410435, Christchurch Central City, Christchurch 8011	-43.53277428	172.6240423	New	2020	2020	Hospital	10	0	4	1.25				Category 2 one-way steel MRF		Raft										-	-	LRB, PTFE Slider		Tx = 3 Ty = 3.5
	ANZ Center	NA		Beca	Christchurch	267 High Street, Christchurch Central City, Christchurch 8011	-43.53265504	172.6372275	New	2016	2016	Commercial	4		3	1.25				Two-way, steel MRF		Raft												Double cancave slider		
	Paragon Apartments	NA		Holmes	Christchurch	69 Armagh Street, Christchurch Central City, Christchurch 8013	-43.52844853	172.6327919	New	2019	2019	Residential	9	0	2	1.25	D	$T1x = 0.81$ $T1y = 0.7$	0.17g (MCE) 0.11g (ULS)	RC internal shear-wall structure with steel gravity frame	Comflor	Raft		3.23	3.23	30	22.6	500	4.6		250UB26	250UC90	-	-	LRB, PTFE sliders	450	2.1
	Deloitte House	NA		Aurecon	Christchurch	151 Cambridge Terrace, Christchurch Central City, Christchurch 8013	-43.5299466	172.6327916	New	2015	2015	Commercial	6	1	2	1 (ULS) 1.25 (MCE)	D			Steel MRF around perimeter and a pair in core	Comflor	Concrete pad foundations and strip footings connected by ground beams		5	4	28	51	1180	8.05	8.4	700CW8177	800CW8204	-	-	TFP	575	2.8s (1/500) 3.3s (1/2500)
	Grand Central	NA		Aurecon	Christchurch	161 Cashel Street, Christchurch Central City, Christchurch 8011	-43.53265343	172.6386669	Redevelopment	1986	2016	Commercial	7	1	2	1	D			Steel MRF (EW), CBFs (NS) and in perimeter, core and atrium	Comflor	Raft		4.55	3.85	31.85	60	3350	8.4	7.1	610UB125	400x370 section	-	-	TFP	700	3.4s (1/500) 4.0s (1/2500)
	King Edward Barracks	Pita Te Hori District Energy Scheme		Aurecon	Christchurch	93 Cambridge Terrace, Christchurch Central City, Christchurch 8013	-43.53270869	172.6326261	New	2017	2017	Commercial	5	0	2	1	D		0.15g (ULS) 0.22g (MCE)	Precast concrete wall and CBFs in both principal directions	Comflor	Raft		4.5	3.9	20.1	44	1460	10.8	10.8	700WB130	310UC97	-	-	TFP	600	2.5s (1/500) 3.2s (1/2500)
	Tui Brewery Tower	NA		Dunning Thornton	Pahiatua	Mangatainoka, Pahiatua 4982	-40.42184087	175.8651296	Retrofit	1931	2018	Commercial	9	1	2		D	0.3	0.16g (MCE)	RC frame and adding RC shear panel within URM walls	Combination of RC and timber T&G joists	Raft	15MN	3.36	-	21.7	28	640	7.45	12.9	150UB18 150PFC	-	-	300sq	TFP	1000	4.8 (MCE)
	Miriam Corban Retirement Village Center (BCC)	NA		Mitchell Vranjes	Auckland	211 Lincoln Road, Henderson, Auckland 0610	-36.85804504	174.6301289	New	2024	2024	Residential	5	1	3	1.25	D			RC Shear Wall with steel beam and column	Rib and Timber	Raft	180093	3.4	3.2	18.8	83.8	4160	-	8.6	-	See comment	-	600x600, 600x400,800x290	LRB, RB, FS	350	2.68
	Spark data centre	NA		Aecom	Auckland	23 Popes Road, Takanini, New Zealand	-37.03008405	174.9263897	New	2014	2014	Commerical	3	0	4	1	C	1.6	0.12g	Steel braced frame	Comflor	Pile	7500	4.5	4.5	15	53	1961	6	7	530UB	250UC	-	-	LRB	N/A	3
	Wellington Conference & Event Centre	Tākina		Dunning Thornton	Wellington	50 Cable Street, Te Aro, Wellington 6011	-41.29107118	174.7810064	New	2023	2023	Public	3	0	3	1	D		0.34g (ULS)	Diagrid (diagonal concrete filled CHS column)	Comflor	Steel tube pile	134MN	7.175	7.35	25.9	63.5	4600	9.6	9.6	610UB125	457x12.7 CHS	-	-	LRB, PTFE sliders	700	1.8s (ULS) 2.5 (CAL1)
	BNZ Office	NA		Dunning Thornton	Wellington	1 Whitmore Street, Wellington Central, Wellington 6011	-41.28161208	174.778704	New	2023	2023	Commercial	15	0	2	1	D	0.95		Diagrid (diagonal concrete filled CHS column)	Comflor	Pile	122000	6.96	3.9	57.16	63	1530	-	-	610UB125	610x16CHS	-	-	LRB	750	2.1s (ULS) 3s (CAL1)
	Whanganui Hospital (Perioperative block)	NA		Romulus Consulting Group	Whanganui	100 Heads Road, Gonville, Whanganui 4501	-39.94391874	175.0376926	New	2008	2008	Hospital	1	0						RCMF & Braced frames														Ro Gliders	400	3
	Whanganui Hospital (Acute services building)	NA		Romulus Consulting Group	Whanganui	100 Heads Road, Gonville, Whanganui 4501	-39.94391874	175.0376926	New	2008	2008	Hospital	1	0						RCMF & Braced frames														Ro Gliders	400	3
	Justice and Emergency Services Precinct	CJESP		Holmes	Christchurch	20 Lichfield Street, Christchurch Central City, Christchurch 8011	-43.53453694	172.6343013	New	2017	2017	Commercial	4	1	4		D	See comment		Category 2 two-way steel MRF	Comflor	Raft		4	4	20		40000					750 CHS concrete filled	-	-	LRB, PTFE	600	Tx = 3 Ty = 3.25

	Wynn Williams House	St Elmo Courts		Ruamoko Solutions	Christchurch	47 Hereford Street, Christchurch Central City, Christchurch 8013	-43.53167485	172.6310066	Redevelopment	1930	2014	Commercial	6	1	2	1	D	0.75		2 way seismic PT frames with precast concrete columns, PT timber LVL beams, secondary steel beams	Comflor	Raft	33218	3.5	3.6	24	40	1062	8.69	9.43	650x550 LVL	-	-	700sq	LRBs, Slider	400	2.18
	Christchurch Art Gallery	NA		Ruamoko Solutions	Christchurch	312 Montreal Street, Christchurch Central City, Christchurch 8013	-43.53067981	172.6309791	Retrofit	2003	2016	Public	3	1	3	1.25	D					Raft	236177	6.9	-	16.8	91.4	6060							TFP	550	
	Environment Canterbury Office	Ecan		Quoin Structural Consultants	Christchurch	200 Tuam Street, Christchurch Central City, Christchurch 8011	-43.38897	172.74215	New	2016	2016	Commercial	5	1	3		D			RC shear wall		Grillage beam with piles	96580	5.8	3.96	21.3	65.7	1860	8.6	5.94		-	800x450	750x750	LRB	500	2.27
	St George Hospital (Pacific Radiology - stage 2 & 3)	Cressy and Randolph Wing		Powell Fenwick	Christchurch	249 Papanui Road, Strowan, Christchurch 8014	-43.51075408	172.6187056	Redevelopment	1928	2020	Hospital	4	1	4	1	D		0.52g	RC shear wall with a combination of steel and concrete beams	Rib and Timber	Raft		3.5	3.5	17.6	76	3436	9.5	8.2	-	-	900x600	600sq	LRB, ERB	650	
	St George Hospital (Cancer Care Centre stage 1B)	NA		Powell Fenwick	Christchurch	249 Papanui Road, Strowan, Christchurch 8014	-43.51071156	172.6174015	New	2016	2016	Hospital		1	4	1	D	Tx = Ty = 0.02	0.49g	RC shear wall		Raft				26.4	478			-	-			LRB, ERB	650	
	St George Hospital (Cancer Care Centre Bunker 3)	NA		Powell Fenwick	Christchurch	249 Papanui Road, Strowan, Christchurch 8014	-43.51055672	172.6172301	New	2023	2023	Hospital		1	4	1	D		0.59g	RC shear wall		Raft				7	20.2	325			-	-			LRB, ERB	650	
	138 Victoria Street	NA		Quoin Structural Consultants	Christchurch	138 Victoria Street, Christchurch Central City, Christchurch 8013	-43.52159888	172.6288683	Redevelopment	2015	2015	Commercial	4	1	2	1	D		See comment	RC frame and precast panel with steel portal frame at top level	In-situ concrete flat slab on hollowcore	Piles		3.6	4.32	18.5	44.25	680	12.25	8.5	-	-	1050x650	120x650	LRB	630	2.17 (frame) 1.62 (panel)
	Old High Court	NA		Holmes	Wellington	85 Lambton Quay Lambton Quay, Whitmore Street, Wellington 6011	-41.27989619	174.7765866	Retrofit	1881	2010	Commercial					D			Masonry														LRB, Pot Sliders	250	
	Wellington Airways Control Tower	NA		Holmes	Wellington	1 George Bolt Street, Rongotai, Wellington 6022	-41.32608472	174.8027081	New	2018	2018	Commercial	9	0	4	1	C			Steel CBF	Comflor	Raft	19159	3.27	3.27	31	13.6	382	-	4.8	360UB51 460UB74	310UC118 200UC60 457X12.7CHS	-	-	LRB	725	
	Wakefield hospital specialist centre	NA		Aurecon	Wellington	99 Rintoul Street, Newtown, Wellington 6021	-41.31756418	174.7784673	New	2021	2021	Hospital	6		3																			LRB		
	Victoria Lane Apartments	NA		Dunning Thornton	Wellington	161 Victoria Street, Te Aro, Wellington 6011	-41.29156864	174.775102	New	2023	2023	Residential	17	0	2	1	D	1		Diagrid on the lower floors up to a CBF in the upper apartment floors	Comflor	Concrete filled steel tube pile	103000	4.025	3.85	58.2	37.5	1200	7.5	9.9	610UB113	508x12.7 CHS	-	-	LRB, pot bearing sliders	700	2.6s (ULS) 3.3S (CALS)
	Air Traffic Control Facility	NA		Aurecon	Christchurch	20-26 Sir William Pickering Drive, Burnside, Christchurch 8053	-43.49596475	172.558639	New	2018	2018	Commercial	2	0	4	1	D		0.163g (ULS) 0.187g (CALS)	Steel CBFs and MRFs	Comflor	Grillage of RC beam		4.5	3.8	11.5	49	1107	10.5	6	800WB168	700WB150	-	-	TFP	N/A	4.43s (ULS) 6.78s (CALS)
	Bell Gully Building	Site 9		Dunning Thornton	Wellington	Level 2/40 Lady Elizabeth Lane, Wellington Central, Wellington 6011	-41.28216429	174.7788167	New	2022	2022	Commercial	5	0	2	1	D			two way steel MRF	Comflor	RC piles	28930	3.8	3.4	17.5	53.1	650	7.2	5	530UB92	508x16 CHS Conc. Filled	-	-	LRB, pot bearing sliders	NA	1.8s (ULS) 2.5s (CALCS)
	Spark square	NA		Holmes	Christchurch	2 Cathedral Square, Christchurch Central City, Christchurch 8011	-43.53156699	172.6369683	Redevelopment	2020	2020	Commercial	4	1	2	1.25	D	T1x = 1.14 T1y = 1.06	0.23g	2-way steel MRF	TT	Raft		5.5	3.8	21.7	75	2340	7.5	8.4	700WB130 800WB146	610x10CHS concrete filled	-	-	LRB, PTFE sliders	600	2.5
	Wellington Children's Hospital	Te Wao Nui		New Zealand consulting engineer limited	Wellington	21 Riddiford Street, Newtown, Wellington 6021	-41.30763803	174.7790745	Redevelopment	2022	2022	Hospital	4	0	4		C	0.94 (long.) 0.9 (trans)		Steel MRF	Comflor	Raft and timber piles	58155kN	4.5	4.2	18.4	68	2745	8.5	8.5	3PL500x240x16x25 3PL500x240x12x20	500mm box column concrete filled	-	-	TFP	N/A	2.3
	Kaweka hospital (Stage II)	NA		Kotahi Engineering Studio	Hastings	209 Canning Road, Hastings 4120	-39.62778207	176.8224977	Redevelopment	2025	2025	Hospital	3	0	3	1	D	0.34		Braced core with hybrid CBFs and conventional CBF	Comflor	Grillage of ground beams supporting the plinths	46825 kN	4.32	4.32	16.2	59.5	2200	8.5	8.5	710CW8162 610UB101	250UC (interior) 310UC (core) 200UC (exterior)	-	-	LRB, RB, FS	700	2.56
	Wellington Museum	Bond Store		Holmes	Wellington	3 Jervois Quay, Wellington Central, Wellington 6011	-41.2851609	174.7781027	Retrofit	1892	1999	Public	3							load-bearing brick masonry		Pile													HDRB, LRB, FS		
	Awly building	NA		Beca	Christchurch	287, Awly Building level 1/293 Durham Street North, Christchurch 8013	-43.5291936	172.63334	New	2017	2017	Commercial	5																									
	Upcoming Building																																						
	Keith Park Retirement Village Center (BIO)	NA		Mitchell Vranjes	Auckland	3 Scott Road, Hobsonville, Auckland 0616	-36.80035153	174.6562737	New	2025	Under Construction	Residential	4	1	3	1.25	D			RC Shear Wall with steel beam and column	Rib and Timber	Raft	181000	3.4	3.2	13	62.1	1454	6.8	-	-	310UC96.8	-	360x600	LRB, RB	300	2.45
	Turnbull house	NA		Dunning Thornton	Wellington	25 Bowen Street, Wellington Central, Wellington 6011	-41.27904699	174.7764207	Retrofit	1916	Proposed	Public	3	0			D			LRW (existing) + RC frame longitudinally and a shear wall parallel (new)		Raft		3.5											TFP		
	Heke Rua Archives	NA		Aurecon	Wellington	12 Aitken Street, Thorndon, Wellington 6011	-41.27709	174.780226	Redevelopment	2006	Under Construction	Public	8	1	3	<1.25 (ULS) <2.5 (CALS)	D	Tx=1.21s Ty=1.19s		two way steel MRF	Comflor	Bored piles with a grillage of ground beams		4.5	3.8	34.9	66	2530	8	9	700-800 deep HC8/HC8C sections	1000HC8C section (interior) 950CHS concrete filled sections (exterior)	-	-	TFP	1300	3.34s (CALS) 3.12s (ULS)
	Dunedin Hospital (Inpatient Building)	NA		Holmes	Dunedin	201 Great King Street, Central Dunedin, Dunedin 9016	-45.86918502	170.5081687	New	Proposed	Proposed	Hospital	11		4		C					Pile														
	Parliament precinct (Museum St Building)	NA		Holmes	Wellington	1 Museum StreetPipitea, Wellington 6160	-41.27724838	174.7764823	New	Under Construction	Under Construction	Residential																									
	Canterbury Museum (McDougall Building)	NA		Holmes	Christchurch	66 Gloucester Street, Christchurch Central City, Christchurch 8013	-43.52987702	172.6320716	Redevelopment	1870	Proposed	Public																									
	Wellington Central Library	Te Matapihi		Aurecon	Wellington	65 Victoria Street, Wellington Central, Wellington 6011	-41.28792897	174.7769747	Retrofit	1991	Under Construction	Public	5	1	3	1 (ULS) ≤1.25 (MCE)	C	Tx=0.82 Ty=0.73	See comment	RCMF (existing) & Steel CBF (new)	Hollowcore (existing) Comflor80 (new)	concrete ground beams and concrete bored piles	123440	6.825	3.55 & 4.85	27		1155	8	8	-	-		800 dia	LRB, Flat slider	850	See comment
	Auckland City Hospital ANO Central Plant	NA		Aurecon	Auckland	2 Park Road, Grafton, Auckland 1023			New	Under Construction	Under Construction	Hospital	5	1	4	1	C	0.8	0.12g	RC frame (gravity) 200SHS X frame (lateral)	TT + In-situ	Pile	12MN							-	-	700D*600W	750sq	TFP	1350	2.6
	110 Jervois Quay	NA			Wellington	110 Jervois Quay, Wellington Central, Wellington 6011	-41.29005594	174.7785225	New	Proposed	Proposed	Commercial																									UCS: 2.23 (U.B.), 2.12 (L.B.)
	1-23 Tasman Street (Northern Apartments)	NA			Wellington	1-23 Tasman Street, Mount Cook, Wellington 6021	-41.299078	174.7782322	New	Proposed	Proposed	Residential	10														36										
	1-23 Tasman Street (Southern Apartments)	NA			Wellington	1-23 Tasman Street, Mount Cook, Wellington 6021	-41.299078	174.7782322	New	Proposed	Proposed	Residential	9														33										
	Wellington Town Hall	NA		Holmes	Wellington	109 Wakefield Street, Wellington Central, Wellington 6011	-41.28919283	174.7773055	Retrofit	1904	Under Construction	Public	3	1	3		D			URM structures strengthen with concrete overlay walls, vertical banding and 4 RCMF	Comflor	Raft and screw piles		4.67	4.29	14.3	64.2	3300								LRB, PTFE slider	476 (min) - 945	
	Taranaki Base Hospital's acute clinical services	NA		Holmes	New Plymouth		-39.07205604	174.0561524					5																									