Seismic loss assessment of reinforced concrete moment frames designed to different international codes

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Executive Summary

Observations of the performance of reinforced concrete (RC) buildings after the 2016 Kumamoto earthquake in Japan showed buildings designed to Japanese standards had less damage and downtime compared to buildings following the 2010/2011 Christchurch earthquakes in New Zealand. To evaluate the underlying reasons for the observed difference in performance, this work compares the seismic performance and resulting damage and losses of reinforced concrete moment frames designed according to various seismic design standards. Specifically, a case study building located in Auckland was designed using New Zealand material properties, but with scaled seismic demands and design requirements based on New Zealand, Japanese, United States, and Chilean standards. These countries were selected because they represent developed countries with modern seismic design standards and, in the case of Chile, Japan, and New Zealand, have recently experienced strong (above design level) earthquakes. First, a case study building was selected and re-designed following the requirements of each country. Auckland (a low seismic region) was selected as the location of the case study building for the purpose of validating numerical models using data from a large-scale shake table experiment on a building that was effectively designed for an Auckland level seismic hazard. Next, a numerical model was developed and validated to assess the seismic performance of each design for a range of seismic hazards. Then, a loss assessment was completed for each building to estimate damage and calculate the expected post-earthquake repair costs of structural and non-structural components. Results from the case-study indicate different design methodologies do not drastically influence the relative seismic performance and expected annual losses of moment frames in low seismic regions. Ongoing work is focused on additional case study buildings located in higher seismic regions to provide further insight into design requirements that result in more resilient building performance including non-structural elements that have the greatest influence on losses considering the design standards in different countries.

Technical Abstract

This study compared the relative seismic performance and expected damage and loss of concrete moment frames designed to New Zealand, United States, Chilean, and Japanese standards to help inform potential changes that can improve the resilience of concrete structures in New Zealand. First, a case-study building located in Auckland (a low seismic region) with site class C soil conditions was designed using New Zealand specific materials, but scaled seismic demands and design requirements following New Zealand, United States, Chilean, and Japanese standards. Next, a numerical model was developed of the case study buildings using the opensource structural analysis software OpenSeesPy. The modelling approach was validated using data from a full-scale shake table experiment in Japan. The models were used to subject the different building designs a suite of ground motions scaled to increasing intensities following an incremental dynamic analysis (IDA). An IDA was used (rather than a site-specific multiple stripe analysis) was used because this research was conducted just prior to the release of the updated National Seismic Hazard Model (NSHM). Then, the engineering demand parameters from the IDA were used to calculate the expected damage and repair costs to structural and non-structural components for a typical office building using PACT. Results from the case-study indicate different design methodologies do not drastically influence the relative seismic performance and expected annual losses of moment frames in Auckland (a low seismic region). However it is important to note that in this study the design of the case study was controlled by gravity loading an minimum detailing requirements based on the New Zealand specifications,, minimum strength requirements and minimum column-to-beam strength ratios following the United States and Chilean standards, and a 0.5% elastic drift limit following the Japanese standard. The Japanese and Chilean designs had slightly larger section sizes with a considerable reduction in peak inter-storey drifts, but similar peak floor accelerations compared to the New Zealand and United States designs. In terms of losses, the Japanese design had the lowest total repair cost at moderate and high earthquake intensity levels while the United States design had the highest total repair cost. Finally, losses from drift sensitive non-structural components were driven by damage to wall partitions and losses from acceleration sensitive non-structural components were driven by damage to HVAC systems. Ongoing work is focused on additional case study buildings located in higher seismic regions and assessing those buildings using a probabilistic structural response assessment based on the updated NSHM.

Key Words

International Code Comparison, Reinforced Concrete, Moment Frame Design, Incremental Dynamic Analysis, Seismic Performance, Post-Earthquake Losses, Non-Structural Components

1. Introduction

Over the past several decades, modern code-conforming buildings around the world have experienced a number of earthquakes with seismic demands ranging from Serviceability Limit State (SLS) to Ultimate Limit State (ULS) (e.g., Northridge 1994, Kobe 1995, Darfield 2010, Christchurch 2011, Cook Straight 2013, Kaikoura 2016) which have shown unexpected levels of damage to structural and non-structural elements, specifically at SLS demands (Hare et al. 2012), (Holden et al. 2013), (Bradley et al. 2017). This has led to some reflection within the structural engineering community as to the most effective approaches to design buildings for seismic resilience. Like a number of countries with modern seismic design standards (e.g. the United States and Chile), New Zealand employs a capacity-based design approach that relies on controlled ductile inelastic behaviour which decreases seismic design forces and results in cost savings in the form of smaller section sizes.. On the other hand, seismic design in Japan follows an allowable stress procedure with elastic loading and strict limits on inter-storey drift. Observations of the performance of reinforced concrete (RC) buildings after the 2016 Kumamoto earthquake in Japan showed buildings designed to Japanese specifications resulted in less damage and downtime compared to buildings following the 2010/2011 Christchurch earthquakes in New Zealand even though the earthquakes were of comparable magnitude for building periods above 0.7s (Sarrafzadeh et al. 2017).

Based on the difference in damage and recovery time observed in Kumamoto and Christchurch, there is an interest in quantifying and comparing the performance and post-earthquake losses of structures designed to seismic standards in different countries to help inform potential changes that can improve the resilience of concrete structures in New Zealand. However, previous comparisons of international design standards have demonstrated that differences in construction practices and methods for establishing seismic demands make it difficult to directly compare the performance of buildings designed according to different international standards.

To overcome this deficiency and appropriately identify specific design strategies that could improve the resilience of concrete structures in New Zealand, this study designed a series of case study buildings using New Zealand material properties, but with scaled seismic demands and design requirements based upon New Zealand, Japanese, United States, and Chilean standards. The selected case study building was a four storey concrete frame building located in Auckland – the reason for selecting this location are discussed below. The goals of this work were to (1) identify the factors from each country driving the design of concrete moment frames, (2) calculate the impact each design approach has on seismic performance, and (3) quantify and compare the post-earthquake damage and losses of modern code conforming buildings and identify structural and non-structural building components that drive the losses. This is organized as follows:

- Section 2 compares the seismic design requirements in New Zealand (NZ) to requirements in the United States (US), Chile (CH), and Japan (JPN) and highlights the key differences between the requirements of each country.
- Section 3 introduces the case study building and describes the seismic demands and resulting re-designs of the case study building. The factors driving each of the designs are identified and the designs are compared in terms of expected storey drift (as estimated using an equivalent static force method) and column-to-beam strength ratio.
- Section 4 provides an overview of the structural modelling, seismic performance assessment, as well as the numerical results including collapse mechanisms, inter-storey drift, and total floor acceleration.
- Section 5 evaluates the economic losses of the moment frame designs from each country in terms of total repair cost, and non-structural components most likely to impact total repair cost are identified.
- Section 6 provides key findings and conclusions, and Section 7 describes an overview of ongoing and future work.

2. *International Code Comparisons*

A thorough review of the seismic design requirements from NZ, US, CH, and JPN was first completed to understand the underlying differences between the requirements of each country. Despite continued interaction and collaboration between international codes over the years resulting in an increased similarity between design standards, this work has identified key differences between these standards by comparing them in terms of the following criteria:

1. *Design focus (Strength vs Ductility)*
2. *Load combinations*
3. *Seismic force demand*
4. *Force reduction factors*
5. *Performance requirements*

The following subsections describe how each country addresses the above criteria, followed by a summary of the key differences between countries and how this work addressed the challenges of comparing the performance of buildings designed in different countries.

2.1 New Zealand

New Zealand structural design relies on member ductility and inelastic behaviour to dissipate energy allowing members to be designed for seismic forces lower than the forces corresponding to an elastic response. Seismic design follows the New Zealand Standard for Structural Design Actions part 5 (NZS 1170.5) for earthquake actions (NZS 2004) and the New Zealand Concrete Structures Standard (NZS 3101) for concrete structures (NZS 2006). Loads are combined following load resistance factor design (LRFD) with temporary (live) loads reduced when combined with permanent (dead) and seismic loads.

A probabilistic seismic hazard model is used to generate an elastic response spectrum based on average accelerations and 5% equivalent viscous damping. Elastic accelerations are then scaled by a structural performance factor (S_p) and ductility factor (k_u) to give the design accelerations corresponding to a 1/500-year return period earthquake at the ultimate limit state (ULS). Seismic base shear demands may be calculated using the equivalent static method for most regular structures with a ratcheting index less than 1.5.

For moment frame buildings, beams and columns are sized to meet a minimum 2.5% inter-storey drift limit at ULS with elastic drifts amplified to account for inelastic and p-delta effects. Reinforcement is designed following a capacity design approach based on beam overstrength moments to ensure ductile behaviour and an appropriate hierarchy of strength (e.g. preventing joint failure or brittle shear failure). Additionally, at ULS the code limits the inelastic rotation of potential plastic hinge regions in beams, columns, and walls using an assumed effective plastic hinge length to calculate curvature. The resulting curvature is limited by a factor (K_d) based on the ductility of the structure. At the serviceability limit state, corresponding to a 1/25-year earthquake return period, inter-storey drifts are limited by the minimum drift capacity of individual building components to limit damage and maintain the ability for the building to be used as originally intended without needing any repairs.

2.2 United States

Seismic design in the United States follows the American Society of Civil Engineers ASCE-7 (ACI 2019) for earthquake actions and the American Concrete Institute (ACI 318-19) for concrete structures. Within ACI 381, special provisions are provided for the design of special moment resisting frames (SMRF) that are detailed specifically for seismic actions. Similar to NZS 1170.5, ASCE-7 uses a load resistance factor design with 1.2 times permanent loads and a reduction in live loads for large floor areas when combined with earthquake loading (ACI 2019). The elastic design response spectrum is based on a probabilistic seismic hazard model that uses accelerations from a maximum considered earthquake (MCE). Design accelerations are then scaled to two-thirds of MCE and elastic response values are scaled using a response modification factor (R) based on the structural system and importance factor (I) based on occupancy. Note that the R factor accounts for the design ductility of the lateral system, similar to S_p/k_u in the NZ specification. ASCE 7 allows the use of an equivalent lateral force method to calculate base and storey for regular structures less than 48.8 m tall that are not in a high-risk seismic area or are structures of high importance (ASCE 2016).

Design of beams and columns is similar to New Zealand in that capacity design procedures are utilized to ensure desirable ductile behaviour. This is explicitly required with a minimum column-to-beam strength ratio at joints equal to 1.2 at the design limit state (comparable to ULS in New Zealand). Additionally, the code imposes a maximum 2.0% inter-storey drift limit. There are no explicit performance requirements for serviceability.

2.3 Chile

Reinforced concrete structures in Chile are designed according to the requirements set by NCh430 which uses the requirements in ACI 318 with some modifications specified by amendment DS60 (INN 2008). Specific to seismic design of reinforced concrete structures, the modifications to ACI 318 relevant to special moment resisting frames include:

- Allowing the frame to be designed as an intermediate frame (with less strict detailing requirements than for SMRF) when part of a dual system where walls take more than 75% of the base shear
- A minimum development length equal to 1.4 times fy compared to 1.25 times fy in ACI 318

Seismic forces and drift limit requirements are set by NCh433 with amendments outlined in DS61 (INN 2009). The procedure allows for a standard equivalent lateral force procedure similar to ASCE 7-16 with some differences including:

- Seismic force reduction factor (R) equal to 7 for ductile moment frames compared to 8 in ASCE
- Maximum elastic drift limit equal to 0.2%
    - May use gross section properties
    - No p-delta or inelastic amplification factors applied
- A load combination factor of 1.4 applied to EQ loads when combined with dead and live loads

2.4 Japan

The Architectural Institute of Japan (AIJ) standards are used for seismic design of concrete structures in Japan (AIJ 2019) and are based on the government requirements of the Building Standard Law (BSL 2016). The procedure for seismic design in Japan for buildings under 60 m tall follows an allowable stress design focused on strength with two verification levels.

In the first level (Level 1), allowable stress limits are checked against elastic demands induced from a seismic base shear calculated using the building weight and seismic zone factor based on historical data. In the second level of design (Level 2) member demands are compared to capacity following a pushover analysis with checks on beam-column hinge development for a desired, though not explicitly required, strong-column-weak-beam mechanism and a maximum allowable drift of 0.5% under Level 1 demands for frames. No force reduction factors are applied in Japanese design and there are no explicit performance requirements for serviceability.

2.5 Summary of Differences Between Design Standards

The following tables provide a summary of the equations and parameters used in the design standards for each country in terms of the four criteria described above. Table 1compares the design procedures including design focus, load combination, seismic hazard model, force reduction factors, and performance requirements. Table 2compares the specific variables and equations used to create the elastic response spectrum and corresponding seismic base shear. Further comparisons between the detailing requirements of each country are provided in Appendix A.

Table 2 shows how the seismic demands used to design buildings vary drastically based on the specification for each country due to differences in site soil classification and parameters used to define seismic hazard, design response spectra, and structural ductility. This fundamental difference in establishing seismic demands presents a significant challenge to quantify the relative performance achieved when designing a building using the different standards. Fenwick et al. (2002) compared the resulting seismic demands, stiffness, drift, and ductility of reinforced concrete moment frames designed to NZ, US, and European standards. The study concluded comparisons between resulting designs were misleading due to the interaction and compounding differences between codes as the design progresses through the calculation of seismic base shear, design actions, deflections, and final building performance. A study by Hampshire et al. (2013) comparing ductile reinforced concrete buildings designed to US, European, Italian, and Brazilian standards came to a similar conclusion - that differences in the design spectra from each country made comparing the resulting designs a challenge. Specifically, the difference in shape between the elastic design spectra, without consideration of response modification factors, lead to differences in results of over 100% in some cases. Many additional studies have shown similar results regarding differences in response spectrum shape and resulting seismic demands between international codes (Anderson et al. 1992); (Aninthaneni and Dhakal 2016); (Yu and Chock 2016).

To overcome the deficiency observed in previous research that has compared international design standards, this study used the NZS 1170.5 elastic response spectrum (NZS 2004) as the starting point for seismic demands for all case-study buildings.

Table 1: Comparison of design parameters for New Zealand, United States, Chile, and Japan

Design Parameter	**New Zealand**	**United States**	**Chile**	**Japan**
Design focus	Ductility LRFD	Ductility LRFD	Strength LRFD	Strength ASD
Load Combination*	$G + \psi_E Q + E$	$1.2G + Q + E$	$1.2G + Q + 1.4E$	$G + Q + E$
Seismic hazard model	Probabilistic based on average accelerations	Probabilistic based on maximum accelerations	Deterministic based on maximum recorded earthquake	Deterministic based on maximum recorded earthquake
Force reduction factors	Yes	Yes	Yes	No
Explicit Performance requirements (Design Limit State)	Maximum 2.5% drift Maximum material strain limit (K_d) Columns required to resist probable beam strength with overstrength	Maximum 2.0% drift Minimum column-to-beam strength ratio = 1.2 with overstregth	Maximum 0.2% drift Minimum column-to-beam strength ratio = 1.2 with overstrength	Maximum 0.5% drift Maximum stress limits
Explicit Performance requirements (Serviceability Limit State)	Maximum drift limited by component capacity	No	No	No

*Load combinations represented with typical NZ variables

Table 2: Seismic base shear calculation for New Zealand, United States, Chile, and Japan

Design Parameter	**New Zealand** (NZS 1170.5)	**United States** (ASCE 7-16)	**Chile** (NCh 433)	**Japan** (BSL)

Elastic response spectrum coefficient	C(T) = Ch(T) x Z x R x N(T,D)	SDS = 2/3 x Fa x Sa SD1 = 2/3 x Fv x S1	C = 2.75 x S x Ao x (T'/To)^n	C = Z x Rt x Co
Building location	Z Hazard factor	Ss, S1 Mapped max accelerations	Ao Max acceleration based on location	Z Earthquake region coefficient
Site class	Ch(T) Spectral acceleration	Fa, Fv Site coefficient	S, To, T', n Soil type parameters	Rt (T) Spectrum shape factor
Site location near fault	N Near fault factor	Built into hazard maps	none	none
Design limit state (1/500 yr)	R = 1.0 Return period factor ULS	2/3 DBE	Cmin A0 x S/6g	Co = 0.2 Shear coefficient Level 1
Maximum limit state (1/2500 yr)	R = 1.8	1 MCE	Cmax 0.35Ao/g	Co = 1.0 Level 2
Base shear equation*	V = C(T) x (Sp/ku) x W	V = SDS x (I/R) x W	V = C x (I/R) x W	V = C x Ds x Fs x W
Factors to scale elastic response spectrum**	Ku = 3.59 Based on ductility factor (u = 4)	R = 8 Response modification factor	R = 7 Response modification factor	Ds = 0.3 Ductility factor
	Sp = 0.7 Structural performance	I = 1.0 Importance factor	I = 1.0 Importance factor	Fs = 1.0 Torsional effect factor

*Following an equivalent static or equivalent lateral force (ELF) procedure
**Values specific to ductile RC moment frame design

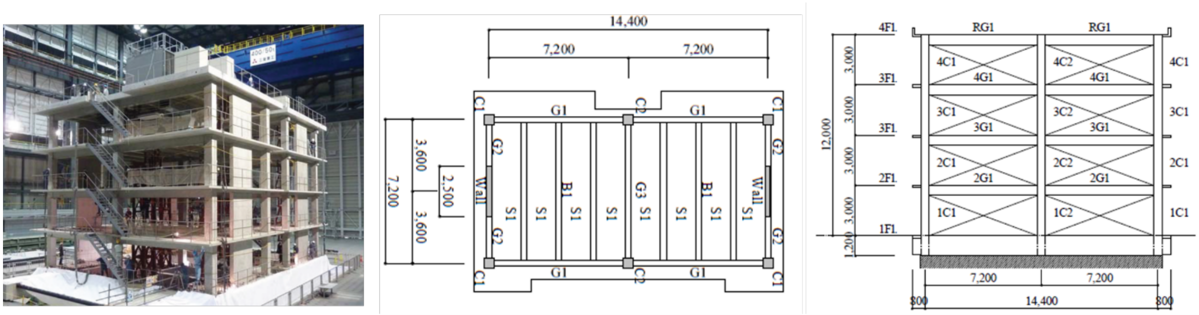
3. *Case Study Building and Moment Frame Design*

This section provides information on the base case study building, the demands used to re-design the case study building according to requirements in the different standards, and a comparison of the resulting designs in terms of expected drift based on equivalent static demands and beam-to-column strength ratios. Additionally, the requirements controlling the various design features are identified.

3.1 Case study building selection

An RC moment frame and shear wall building designed to current Japanese seismic design practice was selected as the case study building for this work. The structure was a full-scale 4-storey building tested at the E-Defense shake table facility in Japan in 2010 with two-bay perimeter seismic moment frames in the longitudinal direction and shear walls in the transverse direction, shown in Figure 1 (Nagae et al. 2015). Only the moment frame direction was considered for this assessment; details on the shear walls and gravity members are not included here. The building has plan dimensions 14.4 m by 7.2 m (Figure 1b) with 3 m storey heights (Figure 1c) and a 130 mm thick floor slab cast monolithically with beams, columns, and walls. Note that this structure was selected as the base case study building due to the availability of experimental data that was used to validate the numerical techniques described in the following sections.

Typical moment frame sections for beams and columns are shown in Figure 2 where hoop and joint transverse reinforcement for columns is specified with number of legs in the B, D directions at the designated spacing. The Japanese design material properties for the case study building were f'_c = 27 MPa for concrete compressive strength and f_y = 345 MPa and f_y = 295 MPa for yield strength of JD22 and JD10 steel reinforcement respectively. The total building weight was estimated at 364 tonnes (1785 kN) and includes the structural system, stairs, mechanical equipment, and testing instrumentation. The case study building, and subsequent re-designs were designed as a typical office building in New Zealand.



(a) (b) (c)

Figure 1: Case Study Building; (a) photo of building on shake table, (b) plan view, and (c) moment frame elevation. Note: all dimensions are in mm (modified from Nagae et al., 2015)

List of Column			
		C1	C2
4Fl. 3Fl.	Section		
	B x D	500 x 500	500 x 500
	Rebar	8-D22	10-D22
	Hoop	2,2-D10@100	2,2-D10@100
	Joint	2,2-D10@140	2,2-D10@140
2Fl.	Section		
	B x D	500 x 500	500 x 500
	Rebar	8-D22	10-D22
	Hoop	2,3-D10@100	2,4-D10@100
	Joint	2,2-D10@140	2,2-D10@140
1Fl.	Top Section		
	B x D	500 x 500	
	Rebar	8-D22	
	Hoop	2,3-D10@100	
	Joint	2,2-D10@140	
	Bottom Section		
	B x D	500 x 500	500 x 500
	Rebar	10-D22	10-D22
	Hoop	3,4-D10@100	3,4-D10@100
	Joint	2,2-D10@140	2,2-D10@140

List of Girder				
		G1		
	Location	End	Center	End
RFl. 4Fl.	Section			
	B x D	300 x 600		
	Top	4-D22	3-D22	4-D22
	Bottom	3-D22	3-D22	3-D22
	Web	4-D10		
	Stirrup	2-D10@200		
3Fl.	Section			
	B x D	300 x 600		
	Top	5-D22	3-D22	5-D22
	Bottom	3-D22	3-D22	3-D22
	Web	4-D10		
	Stirrup	2-D10@200		
2Fl.	Section			
	B x D	300 x 600		
	Top	6-D22	3-D22	6-D22
	Bottom	3-D22	3-D22	3-D22
	Web	4-D10		
	Stirrup	2-D10@200		

Figure 2: Case study building moment frame section details

3.2 Seismic Demands used for Design

A single moment frame from the case study building was re-designed using New Zealand material properties and seismic hazards for a building located in New Zealand but with scaled seismic demands and design requirements following NZ, US, CH, and JPN standards. The location and soil conditions within New Zealand were selected by matching the NZS 1170.5 elastic response spectrum with the base shear demand used to design the case study building moment frame. The Japanese Level 1 design base shear, V_{JPN}(ULS), for the case study building moment frame equals 357 kN and is calculated by multiplying half the weight of the building (for a single frame) by the Level 1 base shear coefficient (0.2); with a zone factor and spectrum shape factor of 1.0. The

Japanese moment frame design base shear is then set equal to the equation for horizontal seismic base shear in NZS 1170.5 and rearranged to solve for the elastic response spectrum values including site class and hazard factor, Z. Ultimately, the seismic base shear demand on the case study moment frame is comparable to the elastic demands calculated using NZS 1170.5 if the building were located in Auckland (Z = 0.13) with site class C soil conditions.

Figure 3 shows the resulting NZS elastic response spectrum compared to the Level 1 Japanese design base shear along with the scaled response spectrums for NZ, the US, and CH. The dashed vertical line represents the New Zealand code approximated period, T_1 = 0.6s, for the case study building in the moment frame direction.

The New Zealand elastic base shear is 1.15 times the Japanese level 1 base shear. This means the equivalent location within New Zealand results in elastic earthquake demands that are approximately 15% larger than the demands used to design the experimental case study building. However, in NZ, the US, and CH, elastic demands are reduced to account for the ductility and inelastic response of the structure. Ultimately, this results in design base shears at ULS that are much lower than those used in JPN design. The resulting scale factors used for the design base shears in each country are also shown in Figure 3 and are found using the parameters specified in the legend.

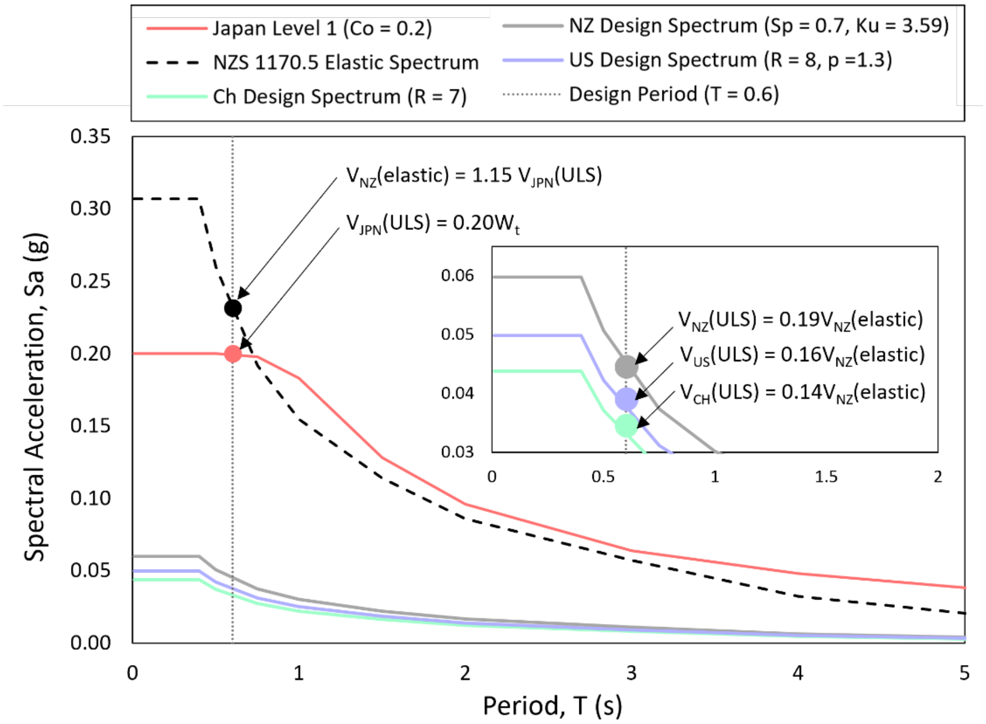


Figure 3: Equivalent demands used for design

3.3 Summary of Moment Frame Designs

As mentioned previously, New Zealand material strengths were used for all designs and were equal to f'_c = 30 MPa for concrete compressive strength and f_y = 300 MPa and f_y = 500 MPa for yield stress of beam and column reinforcement respectively. Each design followed the requirements of the standards set by each country described in Section 2 of this report. The only difference being the JPN "design" moment frame, which utilized the same design as the case study building moment frame but with materials updated to typical New Zealand materials. This involved changing the concrete compressive strength from f'_c = 27 MPa to f'_c = 30 MPa, the yield stress of steel reinforcement from f_y = 345 to f_y = 300 and f_y = 500 for beam and column longitudinal reinforcement respectively, and updating the size of longitudinal reinforcement in beams and columns to typical New Zealand reinforcement sizes. The original moment capacity of beams and columns with Japanese materials was maintained by selecting bar sizes that matched the original bar yield force in the case study beams and columns given the updated yield stress while simultaneously maintaining the relative arrangement of bars within sections. Additionally, the resulting change in concrete and steel stress given the new materials and bar sizes was checked against ultimate Japanese design stress limits of f_y for steel and $\frac{2}{3}f_c'$ for concrete.

Table 3 summarizes each moment frame design in terms of design base shear, controlling load combination, first mode period, section size, and longitudinal reinforcing, as well as the requirements controlling the various design features shown with italics. Complete beam and column section details for each design are provided in Appendix B.

The JPN moment frame is the stiffest having the deepest beams required to meet the 0.5% drift limit with the largest design base shear. The NZ, US, and CH designs differ slightly in section size and resulting stiffness despite similar design base shears. This is due primarily to the difference in load combinations, with the CH design requiring 1.4 times earthquake loading and the NZ design having a controlling load combination based on gravity demands. As such, the factors controlling the design of the moment frames were gravity loading and minimum detailing requirements for NZ, minimum strength requirements and column-to-beam strength ratios for US and CH, and a 0.5% elastic drift limit for JPN.

It is worth nothing that for the NZ design, the maximum ratio of longitudinal column bar diameter to beam depth for interior joints specified by NZS 3101 required either a decrease in already small HD16 column bars or an increase in beam depth. Ultimately, the beam depth was increased to 500 mm to avoid congestion of longitudinal reinforcement in the columns. The increase in beam depth then required larger columns to withstand beam overstrength moments following capacity design procedure for ductile moment frames. This increased the stiffness and strength of the NZ design beyond what was required for strength or displacement.

Figure 4 shows the resulting inter-storey drifts and drift limits (dashed vertical lines) at ULS for each moment frame design with the beam sizes shown in the legend for each country. For design purposes, inter-storey drifts were found using an elastic model of the moment frame with effective stiffness for beams and columns based on the requirements of each country. For the purposes of comparison, however, Figure 4 shows the resulting drifts using the same effective stiffness based on NZS 3101 parameters. The elastic drifts were multiplied by the corresponding factors shown in the legend based on the requirements of each code. For the NZ design, p-delta effects were not required, and elastic drifts were multiplied by 4.8 (equal to u x k_{dm}). For the US design elastic drifts were multiplied by p-delta effects and a deflection amplification factor (C_d) equal to 5.5. The maximum drift limit for moment frames in the US designed for seismic design categories D-F is the 2.0% drift limit divided by the redundancy factor (p) equal to 1.3, resulting in a maximum allowable drift of 1.5%.

Table 3: Summary of moment frame designs

Design Feature	**New Zealand**	**United States**	**Chile**	**Japan**
Design base shear (kN)	78	65	60	357
Load combination	1.2G + 1.5Q*	1.2G + Q + E	1.2G + Q + 1.4E	G + Q + E
Period, T1 (s)	0.85	1.09	0.77	0.68
Beam size (mm)	**300 x 500** *Column detailing requirements*	**300 x 450** *Strength requirements from load combination*	**300 x 525** *Strength requirements from load combination*	**300 x 600** *0.5% allowable drift*
Beam reinforcement (no. top/bot)	**(4/4) D20** *Strength requirements from load combination*	**(2/4-5) D20** *Strength requirements from load combination*	**(4-5/4-5) D20** *Strength requirements from load combination*	**(4-6/3) D25** *Strength requirements from pushover analysis*
Column size (mm)	**500 x 500** *Beam depth*	**450 x 450** *Strength requirements from load combination*	**525 x 525** *Strength requirements from load combination*	**500 x 500** *0.5% allowable drift*
Column reinforcement (total no.)	**(12-16) HD16** *Vertical joint shear*	**(8-10) HD20** *Column-to-beam strength ratio*	**(10-12) HD20** *Column-to-beam strength ratio*	**(8-10) HD20** *Strength requirements from pushover analysis*

*Gravity demand exceeded earthquake load combination

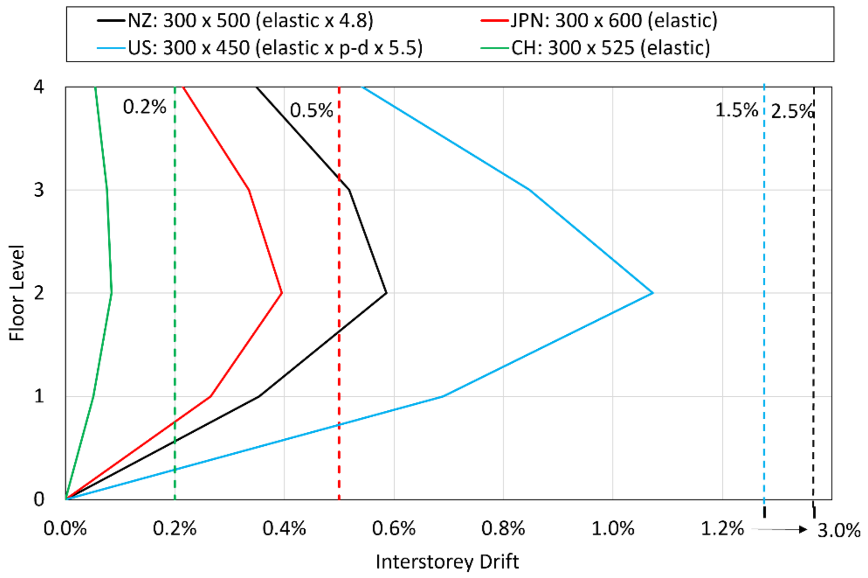


Figure 4: Design drifts and drift limits

Despite a significant difference in maximum drift requirements for each country, the seismic demands generated by the expected hazards in Auckland resulted in moment frames for NZ, US and CH that were generally governed by strength instead of stiffness (which is expected in a low-seismic region). It is worth noting that when using the effective stiffness specified by the Japanese code, the Japanese designed moment frame had a maximum inter-storey drift slightly larger at 0.49% than the 0.40% shown in Figure 4.

3.4 Column-to-Beam Strength Ratios

Figure 5 shows the sum of column-to-beam moment strength ratios at each joint with earthquake loading applied from left to right. A ratio greater than 1.0 indicates columns are stronger than beams and a ratio less than 1.0 indicates beams are stronger than columns. The ratios are different for the outer joints on the right and left sides of the frame because the T-beams have different positive and negative moment capacities, and these values will reverse with reversed earthquake loading.

The JPN design has the smallest column-to-beam moment strength ratios compared to the other designs due to the JPN design having the largest beam sections. Additionally, although not explicitly required by the JPN BSL, moment frame design at Level 2 aims to ensure a strong-column-weak-beam mechanism, however the column-to-beam ratios below 1.0 for the middle joints indicate the Japanese design moment frame may experience some undesirable collapse mechanisms.

NZ, the US, and CH have similar ratios with slight differences based on varying amounts of reinforcement used. This is expected as each of these countries use capacity design procedures with some form of weak beam-strong column requirement. However, instances where the NZ ratios are larger stem from vertical joint shear requirements resulting in stronger columns, and instances where the US ratios are larger stem from asymmetrical top and bottom beam reinforcement resulting in weaker beams.

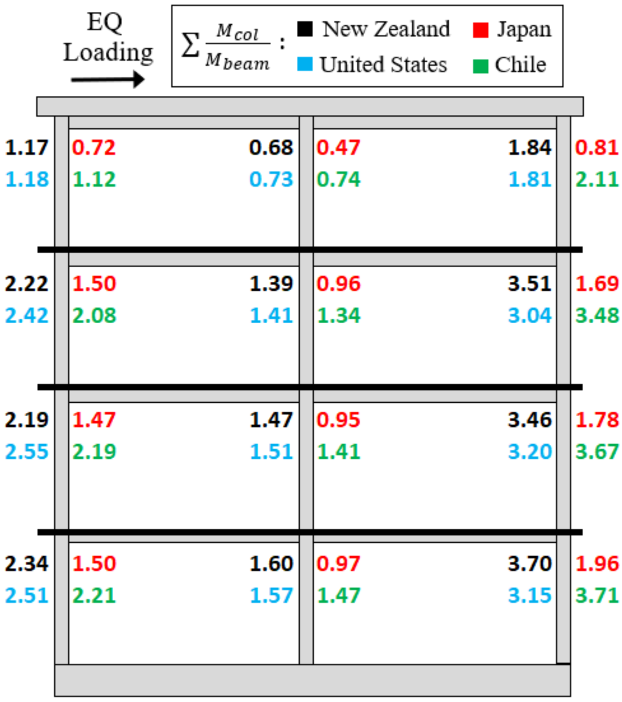


Figure 5: Sum of column-to-beam strength ratios at joints

4. *Seismic Performance Assessment in Moment Frame Direction*

This section provides and overview of the nonlinear structural model developed for each building, describes the seismic performance assessment completed for each design, and quantifies the building performance results in terms of collapse mechanisms, peak inter-storey drift, and total floor acceleration.

4.1 Structural Modelling

Nonlinear two-dimensional (2D) models were developed in the opensource structural analysis software OpenSeespy (Zhu et al. 2018). An overview of the numerical model is shown in Figure 6. The model geometry included a single frame, with special attention given to the joint. Nonlinearity was capture using a lumped moment rotation approach since previous research has shown it can effectively capture inelastic drift, storey acceleration, and collapse (Haselton et al. 2007). The base of the columns were fixed, and ground motion accelerations were applied in the horizonal direction.

The model utilizes lumped centres of mass and rigid diaphragms at each floor level with beam and column members modelled with elastic beam column elements having an effective stiffness equal to 0.4 times gross moment of inertia based on NZS 3101 Table C6.5. Nonlinear moment rotation hinges are assumed to develop at member ends and are modelled using zero length springs with material behaviour based on a Modified Ibarra-Medina-Krawinkler deterioration model with peak oriented hysteretic response (Mod IMK Model) defined using the parameters specified in ASCE 41-17 Tables 10-7 and 10-8 for beams and columns respectively (ASCE 2017). These parameters define the backbone curve shown in Figure 7a, and are converted to OpenSeespy inputs to define the Mod IMK Model shown in Figure 7b.

Beam-column joints were modelled as 50% rigid because the case study building was designed following contemporary ductile detailing standards and the joints were not expected to significantly influence the performance of the moment frame. Gravity loads were applied as distributed beam loads, and the structural response was assessed using a time history analysis with 2% Rayleigh damping applied to the first mode period.

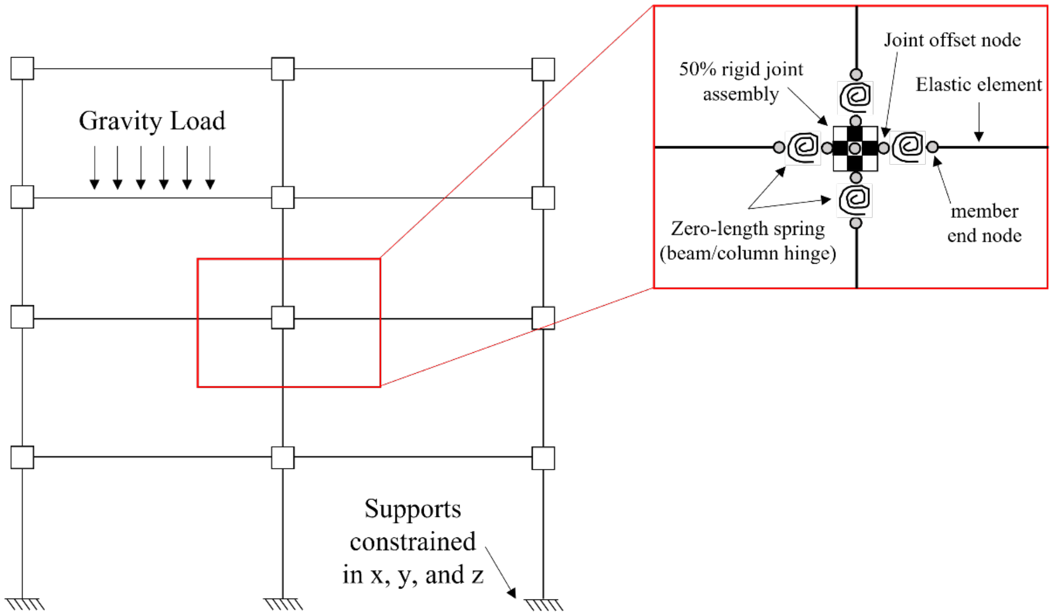
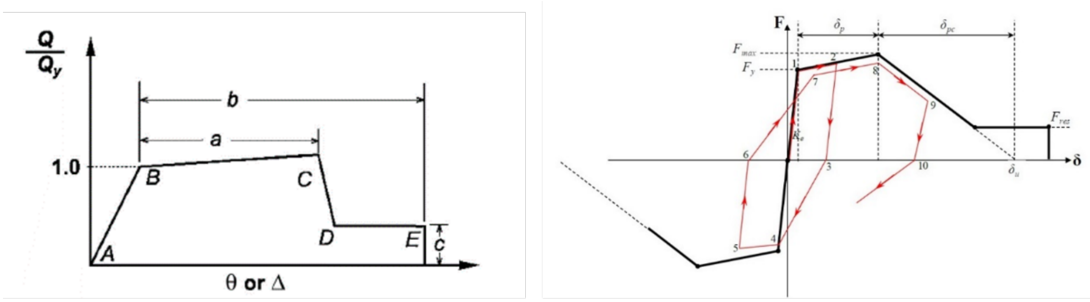


Figure 6: Nonlinear Modelling Parameters



(a) (b)

Figure 7: Nonlinear hinge behaviour; (a) ASCE 41-17 backbone curve (ASCE 2017) and (b) OpenSeespy IMKPeakOreinted backbone curve (Ibarra et al. 2005)

4.2 Incremental Dynamic Analysis (IDA)

To quantify and compare the seismic performance of each design, an incremental dynamic analysis (IDA) was performed using the nonlinear models described above. An IDA is a generic, site-independent approach that progressively scales a set of ground motions to higher intensities until a predetermined limit state is exceeded or structural collapse (Vamvatsikos and Cornell 2001). The IDA approach was used here (rather than a site specific multiple stripe analysis), because this component of the research was conducted in the same time period as the new NSHM was released. Ongoing research is incorporating the new expected hazards into this performance comparison framework.

A total of 39 orthogonal sets of ground motions from the expanded ATC-63 "Far-Field" ground motion set (Haselton and Deierlein 2007) were scaled at increasing $S_a(T_1)$ intensities (shown in Table 4) for a fundamental period equal to the average period of the moment frame designs T_1= 0.85s.

Table 4: IDA intensity levels

Intensity Level	IL1	IL2	IL3	IL4	IL5	IL6	IL7	IL8
$S_a(T_1)$ g	0.1	0.3	0.5	0.7	0.9	1.1	1.3	1.5

For demonstration purposes, Figure 8 shows the ground motion and corresponding response spectrum from the 1999 Kocaeli earthquake recorded at the Duzce station in Turkey. The unscaled values are shown in black, the scaled values for intensity levels 1 (IL1, S_a = 0.1g), 4 (IL4, S_a = 0.7g) and 8 (IL8, S_a = 1.5g) are shown in grey, and the average period of the moment frame designs is shown by the red dashed line in Figure 8b. The scale factor is calculated by targeting specific spectral acceleration intensities from the response spectrum, and the scale factor is applied to the ground motion input for nonlinear time history acceleration data.

Figure 9 shows the scaling processes applied to the entire set of ground motion records with response spectra in grey scaled for intensity level 1, 4, and 8. The mean acceleration values along with the values equal to one standard deviation away from the mean are shown in black. The range of periods from each moment frame design is shown in light red.

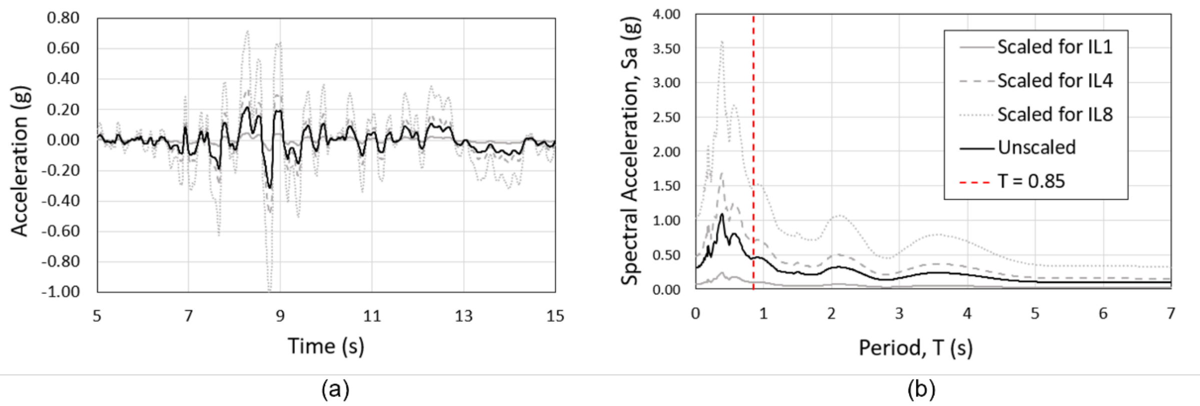


Figure 8: Example IDA scaling process; (a) unscaled and scaled ground motion input and (b) corresponding unscaled and scaled response spectrum

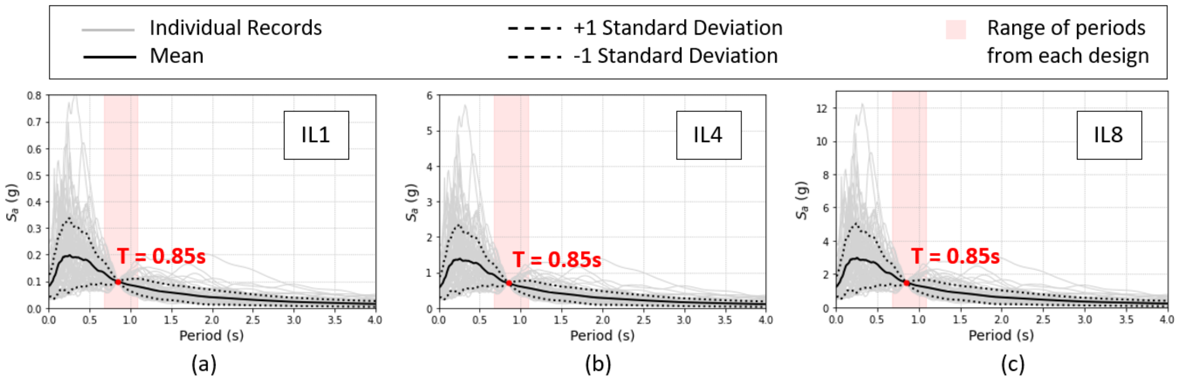


Figure 9: Response spectrums for all ground motions records scaled for (a) intensity level 1, (b) intensity level 4, and (c) intensity level 8

4.3 IDA Curves and Probability of Collapse

Figure 10 shows the IDA curves for each moment frame analysed for all ground motion records at increasing intensities. Each point on the IDA curves represents the maximum inter-storey drift from nonlinear time history analysis and the corresponding scaled spectral acceleration intensity at the average fundamental period of the structures $S_a(T_1)$. Each curve represents a single ground motion scaled to increasing intensity from $S_a(0.85)$ = 0.1 g to collapse (indicated by a horizontal line). The black dashed line represents the mean collapse intensity of all ground motions ($S_{Collapse}$) and the red dashed lines represent the NZS 1170.5 specified 1/500-year ULS design intensity (S_{ULS}) and the 1/2500-year maximum considered earthquake (MCE) intensity (S_{MCE}) for Auckland (Z = 0.13) with site class C soil conditions.

Overall, the moment frames performed well in terms of collapse prevention considering the average collapse intensity for all designs was well above the ULS (0.3g) and MCE (1/2500 year return period) (0.6g) intensities, however the Japanese design moment frame performed slightly better with a mean collapse intensity equal to 1.6g compared to 1.4g for Chile and New Zealand, and 1.3g for United States. The low magnitude of the ULS and MCE hazards relative to the mean collapse intensities clearly demonstrates the designs were not controlled by seismic forces.

The results from IDA were used to fit lognormal cumulative distribution functions to estimate the probability of collapse for each of the designs. The CDF curves were created using the method of moments to approximate the median (θ_m) and logarithmic standard deviation (or dispersion) (β) (Baker 2015). The median represents the spectral acceleration corresponding to a 50% probability of structural collapse. The dispersion is equal to the slope of the CDF and represents the level of uncertainty in the value of spectral acceleration likely to result in collapse. A shallow slope and low dispersion value means there is large uncertainty, whereas a steep slope and large dispersion value means there is a low amount of uncertainty. The resulting fitted collapse fragility functions are shown in Figure 11 and the corresponding median and dispersion values are shown in Table 5.

The probability of failure for the same intensity level decreases as the fragility curves shift to the right. It can be observed from Figure 11 the US design is most vulnerable to collapse, the JPN design is the least vulnerable, and the NZ and CH designs are in the middle with similar likelihoods of collapse. Results from Table 5 show as the period of the moment frame decreases the median collapse capacity and the dispersion increases. This indicates the more flexible United States and New Zealand designs have a higher probability of collapse as well as a higher uncertainty in their predicted collapse intensity. This uncertainty is likely caused by larger levels of nonlinearity in the models.

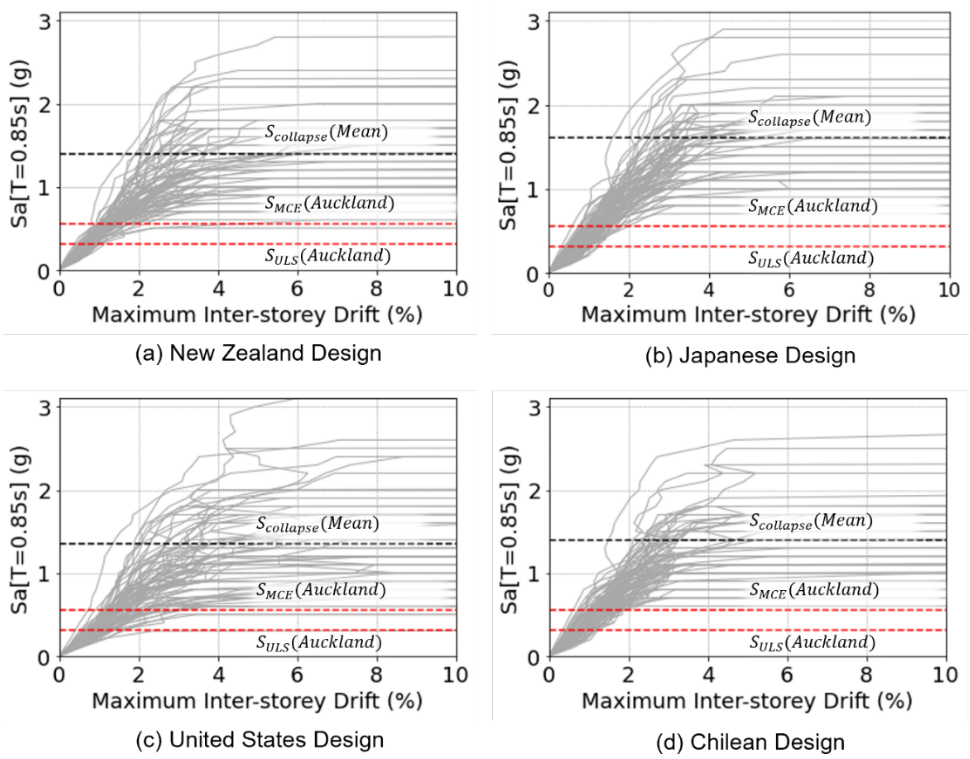


Figure 10: IDA curves; (a) NZ design, (b) JPN design, (c) US design, and (d) CH design

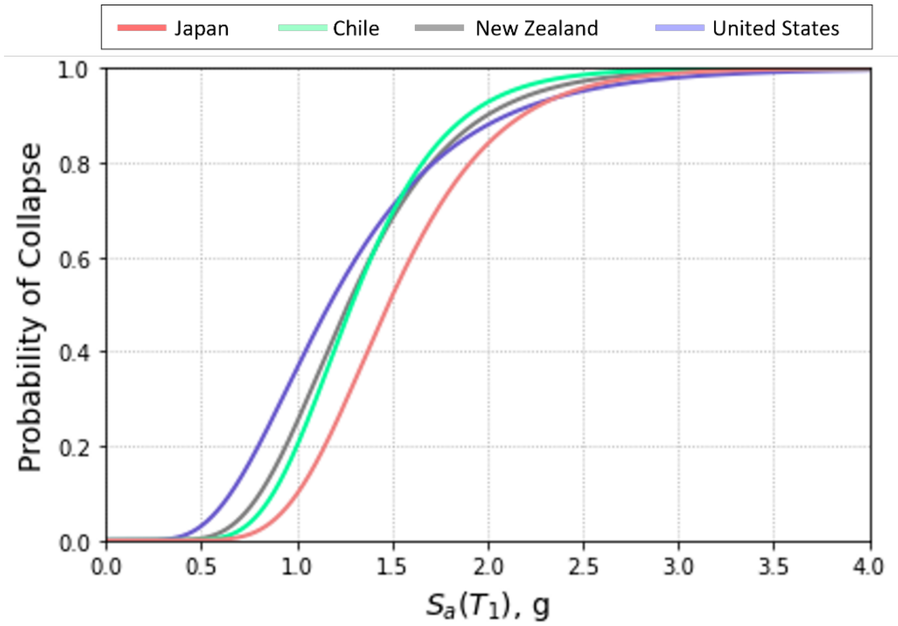


Figure 11: Fitted collapse fragility functions

Table 5: Median (θ_m) and dispersion (β) for the fitted fragility functions shown in ***Error! Reference source not found.***

Design	Period (s)	Median (θ_m)	Dispersion (β)
United Sates	1.09	1.17	0.159
New Zealand	0.85	1.27	0.239
Chile	0.77	1.29	0.255
Japan	0.68	1.48	0.393

4.4 Collapse Mechanisms

Figure 12 shows the percentages of different collapse mechanisms for each design where the small green circles represent beam or column rotations that exceed the yield rotation, and the large red circles indicate beam or column rotations that exceed the ultimate (post-capping) rotation. The US and NZ moment frame designs have more distributed beam-column hinging compared to the Japanese and Chilean moment frame designs which have more beam-column story mechanisms on the lower and middle floors. The difference in collapse mechanisms stem from asymmetrical top and bottom beam reinforcement and lower column-to-beam strength ratios.

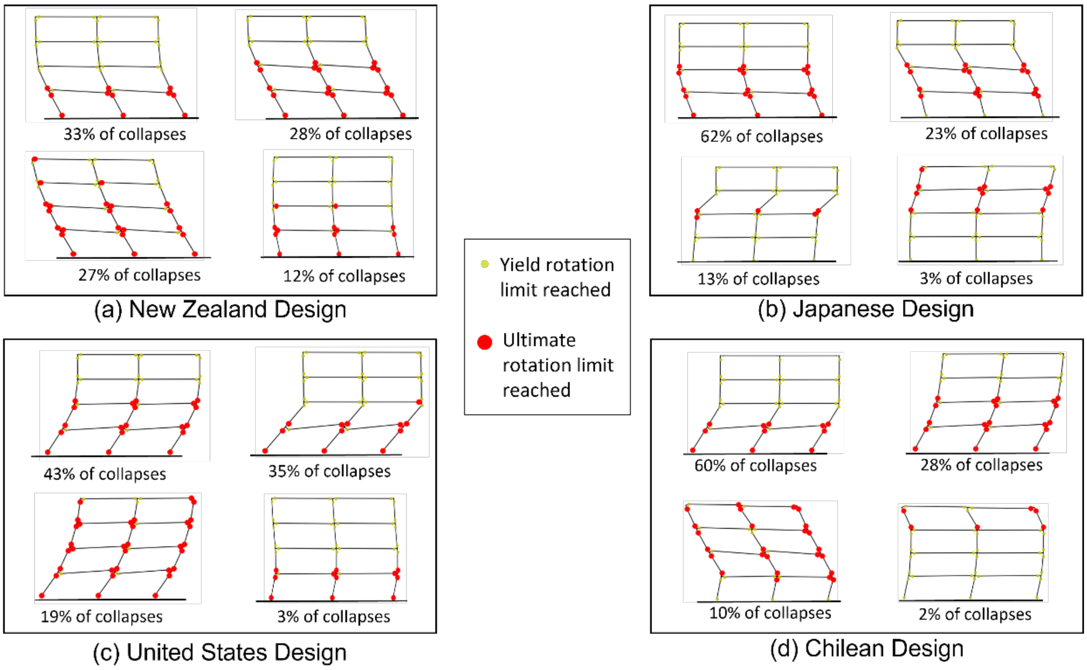


Figure 12: Collapse mechanism results; (a) NZ, (b) JPN, (c) US, and (d) CH

4.5 Engineering Demand Parameters

The inter-storey drift and acceleration results at each intensity level for all designs are shown in Appendix C where the results from each ground motion are shown in grey and the mean values are shown in black. The mean values are found by taking the average from all 39 orthogonal ground motion sets at a single intensity level. These mean values for interstorey drift and floor acceleration are shown in Figure 13 and Figure 14 respectively at intensity levels 2, 4, 6, and 8.

The inter-storey drift results in Figure 13 show the US design generally has the largest drift per floor and the JPN design generally has the smallest drift per floor. This is expected since the US moment frame is the most flexible (T = 1.09s) while the JPN moment frame is the stiffest (T = 0.68s). The largest difference in drift between the designs is seen at moderate intensity levels IL4 and IL6. At low intensity levels (IL1) the designs have similar inter-storey drifts at every floor, and at large intensity levels (IL8) the designs have similar inter-storey drifts at intermediate floors.

The mean total (not relative) floor accelerations in Figure 14 show the stiffer designs have higher floor accelerations, particularly at larger intensity levels. Although the Japanese design is stiffer and generally has lower inter-storey drifts compared to the CH design, the Japanese design also has less total floor accelerations than the CH design. This is due to the larger columns in the CH design (525mm x 525mm) which attract higher acceleration forces than the JPN columns (500mm x 500mm) despite an overall building that is not as stiff. Finally, although the section sizes are not that much larger for the JPN or CH designs, the resulting inter-storey drifts at low to moderate intensity levels (IL1-IL4) are about 0.5% less than the NZ and US design with only about a 0.2g increase in total floor accelerations.

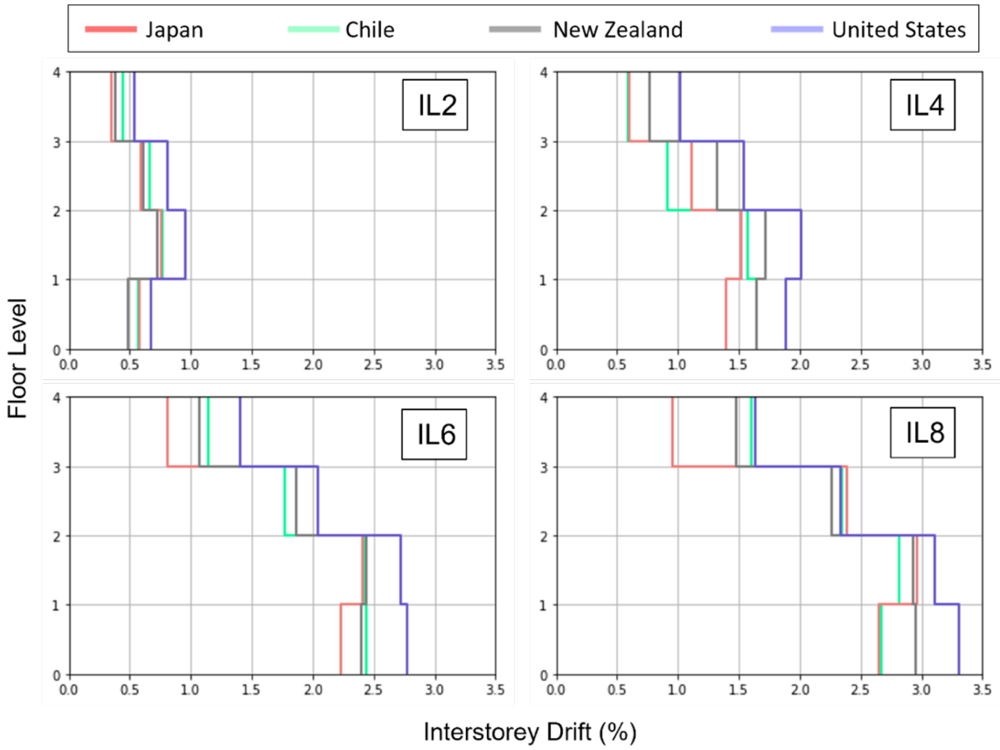


Figure 13: Mean interstorey drift results for intensity levels 2, 4, 6, and 8. Note all drift results can be found in Appendix C

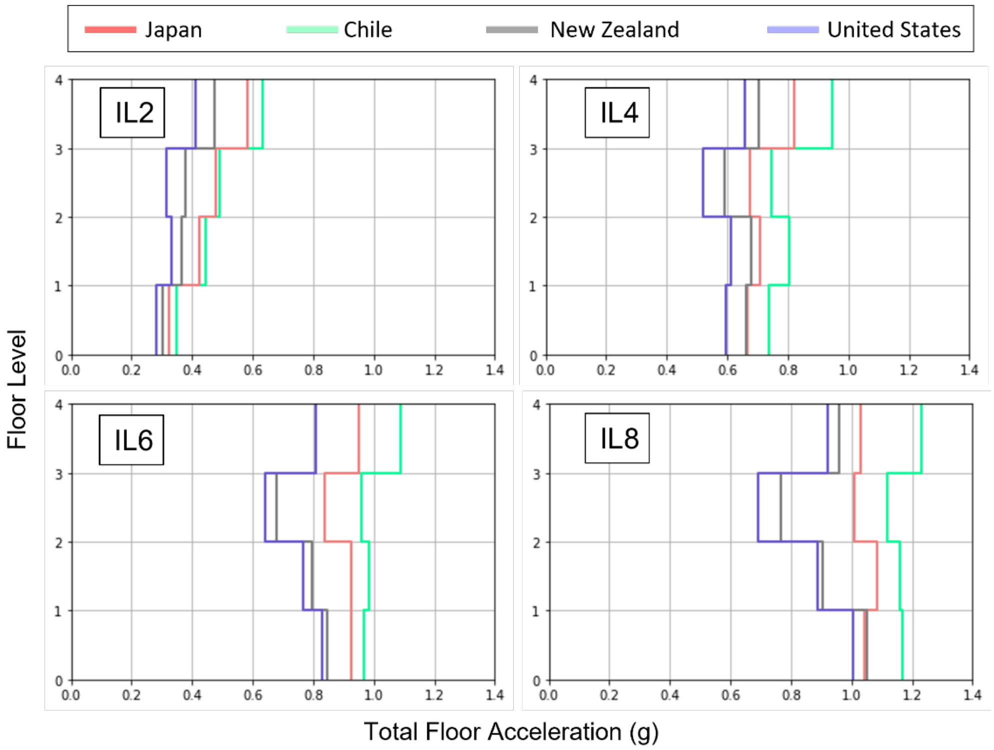


Figure 14: Total floor acceleration results for intensity levels 2, 4, 6,and 8. Note full acceleration results can be found in Appendix C

5. *Loss Assessment*

Finally, a seismic loss estimation to quantify economic losses in terms of damage-related repair costs was completed to assess the potential benefits of the design methodologies used in NZ, the US, CH, and JPN. Other metrics to assess post-earthquake losses include operational downtime and injuries, however only economic losses were considered for this work. The general procedure for estimating losses is based on the Pacific Earthquake Engineering Research (PEER) framework (Deierlein et al. 2003) and includes using engineering demand parameters (e.g. interstorey drifts and floor accelerations) to measure damage and correlate the amount of damage to associated performance metrics (e.g. repair costs, downtime, and/or injuries) using fragility functions for structural and non-structural components.

Here, post-earthquake repair costs were estimated using the Performance Assessment Calculation Tool (PACT) (ATC 2018a), (ATC 2018b) due to its extensive database of fragility functions. PACT provides expected repair costs in USD following a Monte Carlo simulation to create a statistical distribution of building damage states and associated consequences (repair costs) of building components. The same non-structural fitout was used for each design in this study and is based on a typical office building occupancy. Component quantities were obtained by reviewing plans of the case study building. Appendix D provides the building layout plans for non-structural components and includes the layout for partition walls, ceilings, sprinklers, and HVAC system (Clarke 2022). The full list of components including structural, drift sensitive non-structural, and acceleration sensitive non-structural components, fragility functions, and consequence functions are provided in Appendix E. The fragility functions and consequence data (in terms of repair/replacement cost) were obtained from literature or from PACT's fragility database. Note that ongoing work is focused on implementing New Zealand specific fragility functions into this framework.

The total expected repair cost at each intensity level and the loss curve associated with the range of hazards in Auckland for each moment frame design is shown in Figure 15. The total repair cost represents the repair cost for the entire building and is the sum of repair costs from each individual component, converted to New Zealand dollars, and shown in millions. Although PACT utilizes repair costs in USD, the aim of this investigation was to compare the relative performance of one design to the other, therefore it was decided that the difference between the total costs for each design was more important than the overall value. To calculate the expected annual loss, the hazard curve in Auckland was combined with the repair cost vs. spectral intensity plot shown in Figure 15a, and the repair cost vs. probability of annual exceedance curves were integrated (Figure 15b). From Figure 15, it is clear that the building with the JPN designed moment frame has the lowest total repair cost at moderate and high intensity levels (IL4-IL8) while the building with the US designed moment frame has the highest total repair cost. At low intensity levels (IL1-IL3) the repair costs are about the same for all designs, which has indicates that the design approach does not have a large influence on damage or economic loss in low seismic regions. The expected annual loss values in Figure 15b are low and fairly consistent across each design because the range of spectral accelerations for the expected hazards in Auckland (from 1/25-year return period event to a 1/2500-year return period event) only include the repair costs associated with intensity levels 1 and 2 (Figure 15a) from the IDA. This is a very important limitation of this study – the results only apply to regions with low seismicity.

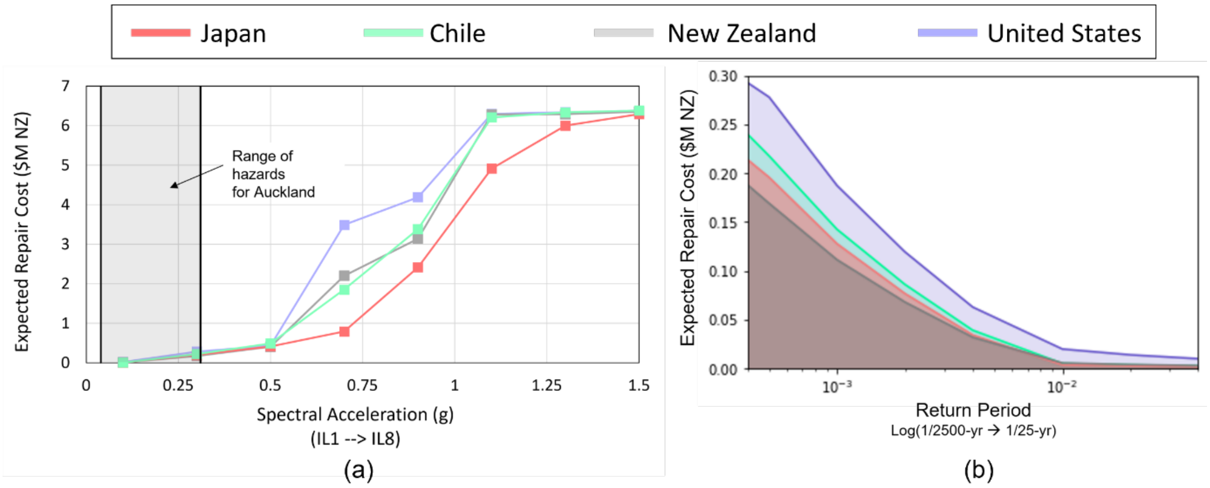


Figure 15: Loss Results; (a) Total expected repair cost and (b) Expected annual loss

To investigate the influence that different component types have on the total repair cost, Figure 16 shows the expected repair costs for different component categories. Costs are grouped by drift-sensitive structural components, drift-sensitive non-structural components, and acceleration non-structural components. Note that the x-axis for the first row of plots (IL1-4) is different than the x-axis for the second row of plots (IL5-8) and costs are shown in $1000 NZ.

Figure 16 shows the repair costs of drift sensitive non-structural components are larger at low to moderate intensity levels (IL1-IL5) while repair costs for acceleration sensitive non-structural components are larger for large intensity levels (IL6-IL8). Addition ally, the largest difference in repair costs between the designs is at intensity level 4, where the US design has considerably larger repair costs for drift sensitive structural and non-structural components, followed by NZ, CH, and JPN. At large intensity levels (IL6-8) the repair costs per component type are about the same for each design.

The repair costs of each individual building component are shown in Figure 17 for intensity levels 1, 4, 6, and 8. Results for all intensity levels are shown in Appendix F. Results show at low intensity levels (IL1 and IL4) losses from drift sensitive non-structural components are primarily from wall partitions. Repair costs for these partitions continue to increase for higher intensity levels with the JPN and CH design being the lowest, however as repair costs from acceleration sensitive non-structural components surpass those from drift sensitive non-structural components, it’s clear the costs are primarily driven by damage to HVAC systems. Only the JPN design building has slightly lower HVAC repair costs at larger intensity levels while the HVAC repair costs for the other designs are approximately the same.

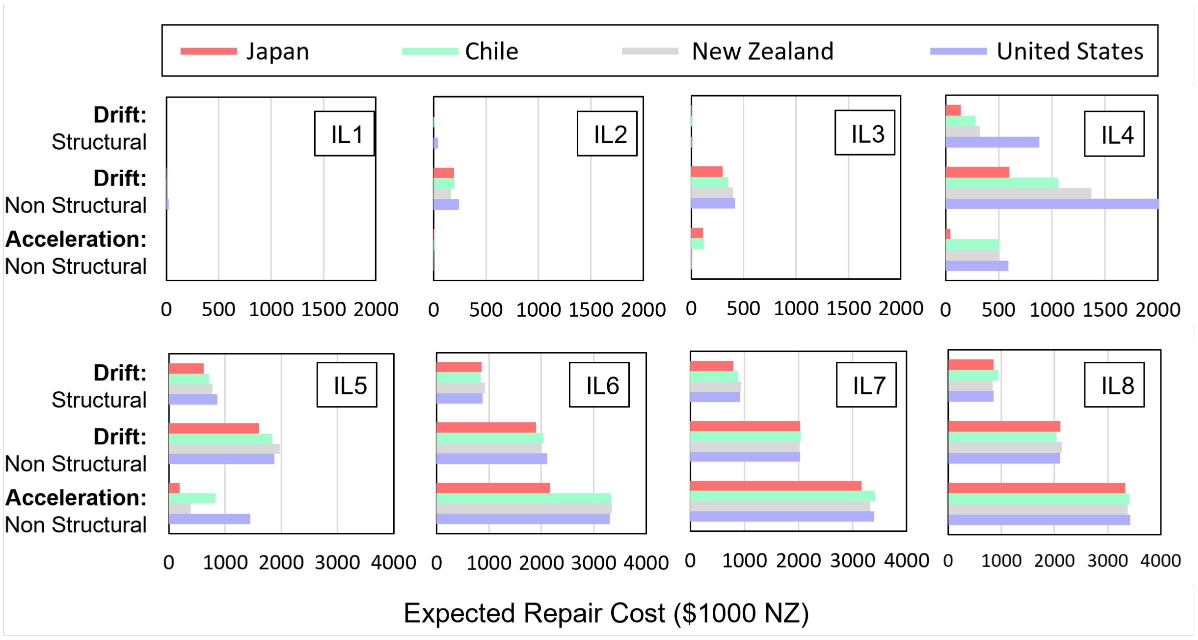


Figure 16: Expected repair cost by component type (Note: x-axis scales are different for IL1-4 and IL5-8)

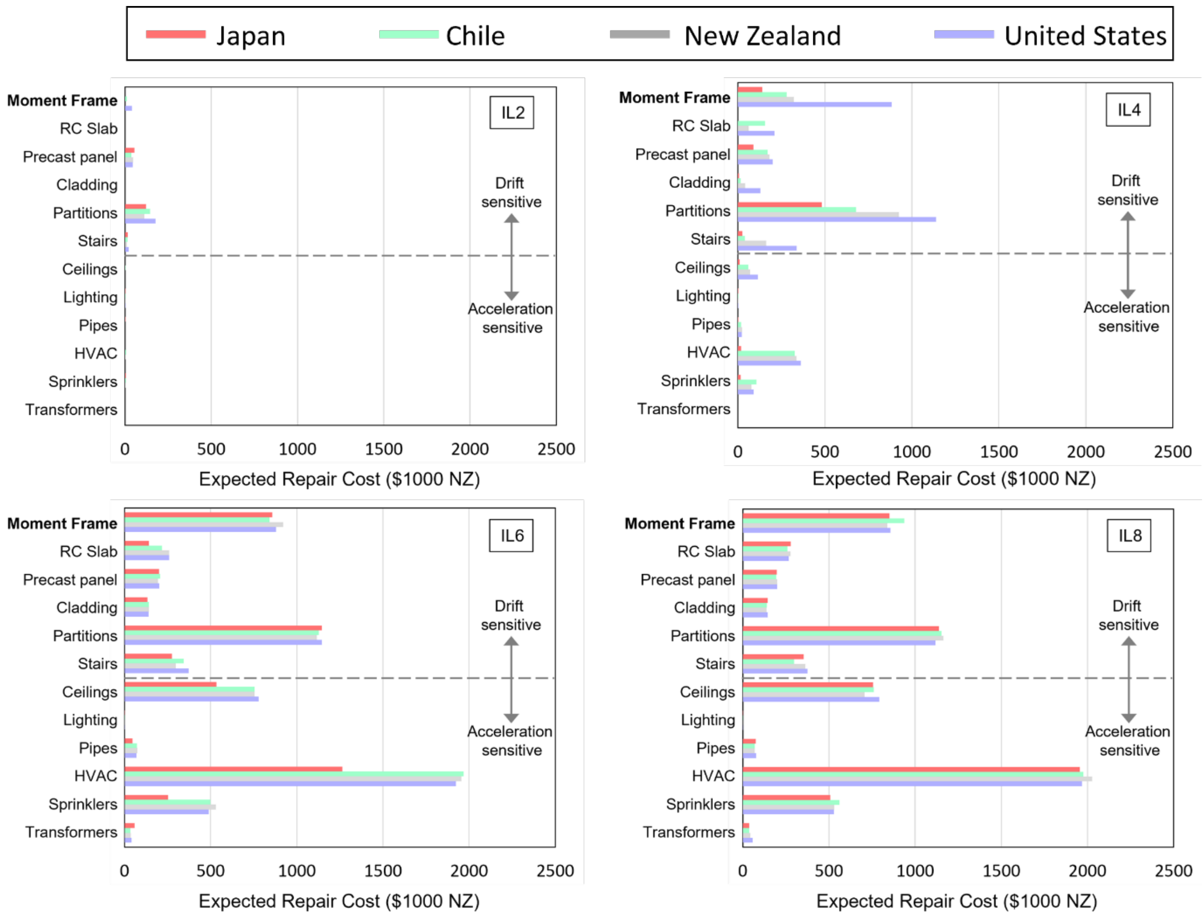


Figure 17: Expected repair costs by individual component (Note: bold indicates structural component)

6. *Conclusions and Key Findings*

This study compared the relative performance and loss of concrete moment frames designed to NZ, US, CH, and JPN standards. The frames were designed with material properties and seismic hazards for a building located in Auckland with site class C soil conditions but with scaled seismic demands and design requirements following NZ, US, CH, and JPN standards. The designs were subjected to a suite of ground motions scaled to increasing intensities following an IDA using nonlinear 2D models developed in OpenSeespy. Results from the IDA were used to calculate the expected loss based on damage to structural and non-structural components for a typical office building. The goals of this work were to (1) identify the factors from each country driving the design of concrete moment frames, (2) calculate the impact each design approach has on seismic performance, and (3) compare the post-earthquake losses of modern code conforming buildings and identify structural and non-structural building components that drive economic losses. The key findings from this work are summarized below for each of these goals.

Factors driving the design of concrete moment frames

- The moment frame design for the given case study building was controlled by gravity loading and minimum detailing requirements when following the NZ specifications noting that the building was located in a low seismic region.
- For low seismic regions, the moment frame design for the given case study building was controlled by minimum strength requirements and minimum column-to-beam strength ratio for the US and CH designs noting that the building was located in a low seismic region.
- The JPN design was controlled by a 0.5% elastic drift limit for the given case study building despite being located in a low seismic region.
- Despite similar drift requirements, the moment frames designed for the given case study building to Japanese specifications had deeper beams and smaller columns compared to the same building designed to Chilean specifications.

Impact on seismic performance

- Asymmetrical top and bottom beam reinforcement used in the JPN design resulted in disproportional hinge development in beam sections and inhibited distributed beam-column collapse mechanisms at moderate earthquake intensities.
- The capacity design requirements used in NZ, the US, and CH generally resulted in moment frames with larger column-to-beam strength ratios that showed more distributed beam-column collapse mechanisms compared to JPN designs.
- JPN and CH design requirements resulted in stiffer buildings with lower inter-storey drifts and larger floor accelerations compared to the NZ and US designs.
- Despite having similar section sizes, the buildings designed to JPN and CH specifications had considerable reductions in peak inter-storey drifts, but similar peak floor accelerations compared to moment frames designed to NZ and US specifications.

Impact on post-earthquake losses

All conclusions regarding relative loss have been made with the caveat that they are only applicable to buildings in low seismic regions as the NZ design was not controlled by the seismic load case. Ongoing work is completing the framework of work presented here for buildings located in Wellington, where all of the designs will be controlled by the seismic load case.

- For low seismic regions, the case study building designed to JPN requirements had the lowest total repair cost at moderate and high earthquake intensity levels while the building designed to United States requirements had the highest total repair cost.
- Repair costs of drift sensitive non-structural components in all cases were larger at low to moderate intensity levels while repair costs for acceleration sensitive non-structural components were larger for large intensity levels.
- Despite larger accelerations at high intensity earthquakes, the case study building designed to JPN specifications had the same or less repair costs to acceleration sensitive non-structural components compared the NZ, US, or CH designs.
- Losses from drift sensitive non-structural components in all cases were driven by damage to wall partitions and losses from acceleration sensitive non-structural components are driven by damage to HVAC systems

7. Future Work

The primary topics of ongoing work include:

1. Completing the procedure discussed in this report for a building located in a high seismic region where the seismic load case governs all designs (Wellington has been selected).
2. Using the updated NSHM and a multiple stripe analysis (rather than an IDA) to calculate site specific losses according to the most updated hazard (both in a low and high seismic region).

Topics of future work include (in no particular order):

- Evaluating differences in performance in the different code specifications for shear wall buildings.
- Evaluating differences in performance in the different code specification for taller buildings.
- Performance of a sensitivity study on the interaction of bi-axial loading using a 3D model with moment frames in one direction and shear walls in the other
- Develop a framework to assess the post-earthquake functionality of modern code conforming buildings given expected post-earthquake losses

8. Impact

This project developed a comprehensive work flow to evaluate the relative performance of buildings designed to different international standards, accounting for differences in both the detailed structural design and the expected seismic forces. This work flow demonstrated that for low seismic regions, there does not appear to be a large difference in expected damage and loss for a concrete moment frame building designed to NZ, US, CH, or JPN specifications. The results of this and ongoing work will provide insight into design requirements that result in more resilient building performance as well as non-structural elements that have the greatest influence on losses considering the different design philosophies in various seismic regions. As additional designs are investigated for larger seismic hazards and different structural systems, it will have significant impact in New Zealand in terms of designing resilient cities and infrastructure. International design strategies which are found to result in more resilient building response will be used to inform future design specifications in New Zealand. This project will provide engineers and policymakers with insight into the strengths and weaknesses of various international seismic design codes in terms of resilience and functionality.

9. Acknowledgements

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11. Outputs and Dissemination

Presentations:

- Hogan, L., Stephens, M., Elwood, K., Buck, N., "Earthquake Demands Assessment", presented at the Third Workshop on E-Defense, May 25, 2021.
- Buck, N., "Structural Resilience Implications of Designing to Various International Standards", presented at the 2022 QuakeCoRE Student Chapter Lightning Talk Competition, August 15, 2022.
- Buck, N., Clarke, G., Stephens, M., Hogan, L., "Structural Resilience and Functionality Implications of Designing to Various International Standards", poster presented at the 2022 QuakeCoRE Annual Meeting, Napier, New Zealand, September 1, 2022.
- Buck, N., "Comparing the Design and Seismic Performance of New Zealand and Japanese Buildings", presented at the March QuakeCoRE IP1 Monthly Meeting, March 8, 2023.
- Buck, N. "Seismic Performance Comparison of New Zealand and Japanese Concrete Buildings", planned presentation at the 2023 NZSEE Conference, Auckland, New Zealand, April 20, 2023.

Conference papers:

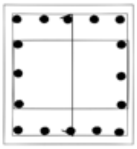
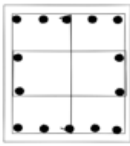
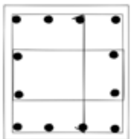
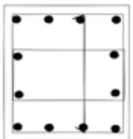
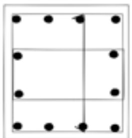
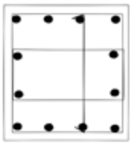
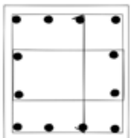
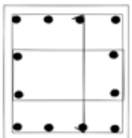
- N. Buck, G. Clarke, M. Stephens, L. Hogan, "Seismic Performance Comparison of New Zealand and Japanese Concrete Buildings", 2023 NZSEE Conference, Auckland, New Zealand, 2023.

Appendix A: Additional Code Comparisons

Table A- 1: Comparison of minimum detailing requirements

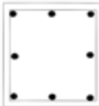
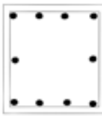
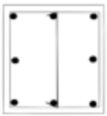
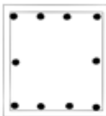
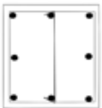
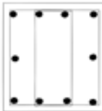
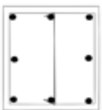
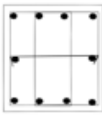
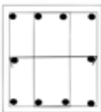
Design Parameter			New Zealand (NZS 3101:2006)	United States (ACI 318-19)	Chile (ACI 318-19)	Japan (AIJ 2019)
Columns	n (min)	*min number of long bars*	8	4	4	4
	p (min)	*min long bar reinforcement ratio*	0.008	0.008	0.008	0.008
	pt (min)	*min trans bar reinforcement ratio*	-	-	-	0.002
	s (max)	*max spacing of trans bars*	min(b/4, h/4, 6db)	min(b/4, h/4, 6db)	min(b/4, h/4, 6db)	min(100, 10db)
	hx (max)	*max spacing between unrestrained long bars*	max(b/4, h/4, 200)	150 mm	150 mm	-
	hx2 (max)	*max center-to-center spacing of tied long bars*	-	360 mm	360 mm	-
Beams	s (max)	*max spacing of trans bars*	min(d/4, 6db)	min(150, d/4, 6db)	min(150, d/4, 6db)	0.75d
	hx (max)	*max spacing between unrestrained long bars*	-	150 mm	150 mm	-
	hx2(max)	*max center-to-center spacing of tied long bars*	max(300 mm, h/6, 3t)	360 mm	360 mm	-
	p (min)	*min long bar reinforcement ratio*	0.0053	0.0053	0.0053	-
	p (max)	*max long bar reinforcement ratio*	0.025	0.025	0.025	-
Beam-Column Joints	db_bm (max)	*max beam long bar diameter passing through joints*	4h_col x sqrt(f'c)/fy	20h_col	20h_col	3.6h_col x (1.5 + 0.1f'c)/ft
	db_col (max)	*max col long bar diameter passing through joints*	4h_bm x sqrt(f'c)/fy	20h_bm	20h_bm	3.6*h_bm x (1.5 + 0.1f'c)/ft
	sh(min)	*min vertical spacing of trans bars*	max(bc/4, hc/4, 200mm)	per col req	per col req	-
	sv(min)	*min horizontal spacing of long bars*	min(10db_col, 200mm)	per col req	per col req	-
Walls	p (min)	*min long bar reinforcement ratio*	0.0025	0.0025	0.0025	0.0025
	n (min)	*min number of curtains*	2	2	2	2 (when t > 200)
	hx (max)	*max spacing of vertical bars laterally supported*	450 mm	350 mm	350 mm	-
	s (max)	*max spacing of transverse bars*	min(6db, 0.5tw)	min(0.25tw, 6db, 150)	min(0.25tw, 6db, 150)	300 mm
	tw (min)	*min thickness of wall*	100 mm	h_storey/16	h_storey/16	min(120, 1/30 x height)

Appendix B: Beam and column sections for design

List of Column			
		C1	C2
4FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	(16) 5, 5-HD16	(14) 5, 4-HD16
	Hoop	4, 3-D10 @ 80	4, 3-D10 @ 80
	Joint	5, 3-D10 @ 60	5, 3-D10 @ 60
3FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	(12) 4, 4-HD16	(12) 4, 4-HD16
	Hoop	4, 3-D10 @ 80	4, 3-D10 @ 80
	Joint	5, 3-D10 @ 60	4, 3-D10 @ 60
2FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	(12) 4, 4-HD16	(12) 4, 4-HD16
	Hoop	4, 3-D10 @ 80	4, 3-D10 @ 80
	Joint	4, 3-D10 @ 60	4, 3-D10 @ 60
1FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	(12) 4, 4-HD16	(12) 4, 4-HD16
	Hoop	4, 3-D10 @ 80	4, 3-D10 @ 80
	Joint	4, 3-D10 @ 60	4, 3-D10 @ 60

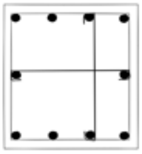
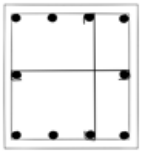
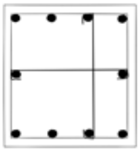
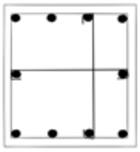
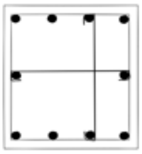
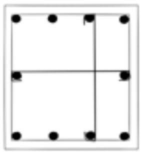
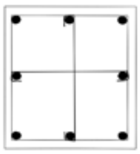
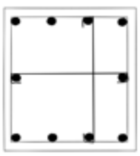
List of Girder			
		Ends	Center
RFL	Section		
	B x D	300 x 500	
	Top	4-D20	2-D20
	Bottom	4-D20	2-D20
	Stirrup	2-D10 @100	2-D10 @ 200
4FL	Section		
	B x D	300 x 500	
	Top	4-D20	2-D20
	Bottom	4-D20	2-D20
	Stirrup	2-D10 @100	2-D10 @ 200
3FL	Section		
	B x D	300 x 500	
	Top	4-D20	2-D20
	Bottom	4-D20	2-D20
	Stirrup	2-D10 @100	2-D10 @ 200
2FL	Section		
	B x D	300 x 500	
	Top	4-D20	2-D20
	Bottom	4-D20	2-D20
	Stirrup	2-D10 @100	2-D10 @ 200

Figure B- 1: New Zealand Design Sections

List of Column			
		C1	C2
4FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	8-HD20	10-HD20
	Hoop	2, 2-D10 @ 100	2, 2-D10 @ 100
	Joint	2, 2-D10 @ 140	2, 2-D10 @ 140
3FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	8-HD20	10-HD20
	Hoop	2, 3-D10 @ 100	2, 2-D10 @ 100
	Joint	2, 2-D10 @ 140	2, 2-D10 @ 140
2FL	Section		
	B x D	500 x 500	500 x 500
	Rebar	8-HD20	10-HD20
	Hoop	2, 3-D10 @ 100	2, 4-D10 @ 100
	Joint	2, 2-D10 @ 140	2, 2-D10 @ 140
1FL	Top Section		
	B x D	500 x 500	
	Rebar	8-HD20	
	Hoop	2, 3-D10 @ 100	
	Joint	2, 2-D10 @ 140	
	Bottom Section		
	B x D	500 x 500	500 x 500
	Rebar	10-HD20	10-HD20
	Hoop	4, 3-D10 @ 100	3, 4-D10 @ 100
	Joint	2, 2-D10 @ 140	2, 2-D10 @ 140

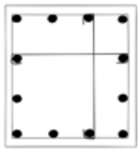
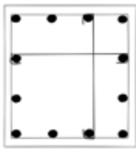
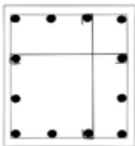
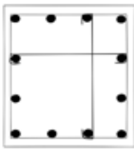
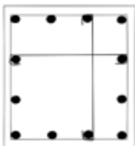
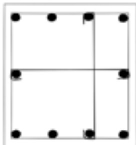
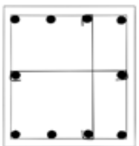
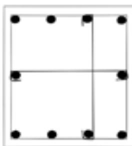
List of Girder			
		Ends	Center
RFL	Section		
	B x D	300 x 600	
	Top	4-D25	3-D25
	Bottom	3-D25	3-D25
	Stirrup	2-D10 @ 200	2-D10 @ 200
4FL	Section		
	B x D	300 x 600	
	Top	4-D25	3-D25
	Bottom	3-D25	3-D25
	Stirrup	2-D10 @ 200	2-D10 @ 200
3FL	Section		
	B x D	300 x 600	
	Top	5-D25	3-D25
	Bottom	3-D25	3-D25
	Stirrup	2-D10 @ 200	2-D10 @ 200
2FL	Section		
	B x D	300 x 600	
	Top	6-D25	3-D25
	Bottom	3-D25	3-D25
	Stirrup	2-D10 @ 200	2-D10 @ 200

Figure B- 2: Japanese Design Sections

List of Column			
		C1	C2
4FL	Section		
	B x D	450 x 450	450 x 450
	Rebar	(10) 4, 3-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
3FL	Section		
	B x D	450 x 450	450 x 450
	Rebar	(10) 4, 3-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
2FL	Section		
	B x D	450 x 450	450 x 450
	Rebar	(10) 4, 3-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
1FL	Section		
	B x D	450 x 450	450 x 450
	Rebar	(8) 3, 3-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70

List of Girder			
		Ends	Center
RFL	Section		
	B x D	300 x 450	
	Top	2-D20	2-D20
	Bottom	4-D20	3-D20
	Stirrup	2-D10 @80	2-D10 @ 150
4FL	Section		
	B x D	300 x 450	
	Top	2-D20	2-D20
	Bottom	5-D20	3-D20
	Stirrup	2-D10 @70	2-D10 @ 140
3FL	Section		
	B x D	300 x 450	
	Top	2-D20	2-D20
	Bottom	5-D20	3-D20
	Stirrup	2-D10 @70	2-D10 @ 140
2FL	Section		
	B x D	300 x 450	
	Top	2-D20	2-D20
	Bottom	5-D20	3-D20
	Stirrup	2-D10 @70	2-D10 @ 140

Figure B- 3: United States Design Sections

List of Column			
		C1	C2
4FL	Section		
	B x D	525 x 525	525 x 525
	Rebar	(12) 4, 4-HD20	(12) 4, 4-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
3FL	Section		
	B x D	525 x 525	525 x 525
	Rebar	(12) 4, 4-HD20	(12) 4, 4-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
2FL	Section		
	B x D	525 x 525	525 x 525
	Rebar	(12) 4, 4-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70
1FL	Section		
	B x D	525 x 525	525 x 525
	Rebar	(10) 4, 3-HD20	(10) 4, 3-HD20
	Hoop	3, 3-D10 @ 70	3, 3-D10 @ 70
	Joint	3, 3-D10 @ 70	3, 3-D10 @ 70

List of Girder			
		Ends	Center
RFL	Section		
	B x D	300 x 525	
	Top	4-D20	2-D20
	Bottom	4-D20	2-D20
	Stirrup	3-D10 @100	2-D10 @ 100
4FL	Section		
	B x D	300 x 525	
	Top	5-D20	2-D20
	Bottom	5-D20	2-D20
	Stirrup	3-D10 @100	2-D10 @ 100
3FL	Section		
	B x D	300 x 525	
	Top	5-D20	2-D20
	Bottom	5-D20	2-D20
	Stirrup	3-D10 @100	2-D10 @ 100
2FL	Section		
	B x D	300 x 525	
	Top	5-D20	2-D20
	Bottom	5-D20	2-D20
	Stirrup	3-D10 @100	2-D10 @ 100

Figure B- 4: Chilean Design Sections

Appendix C: Additional Results from Seismic Analysis

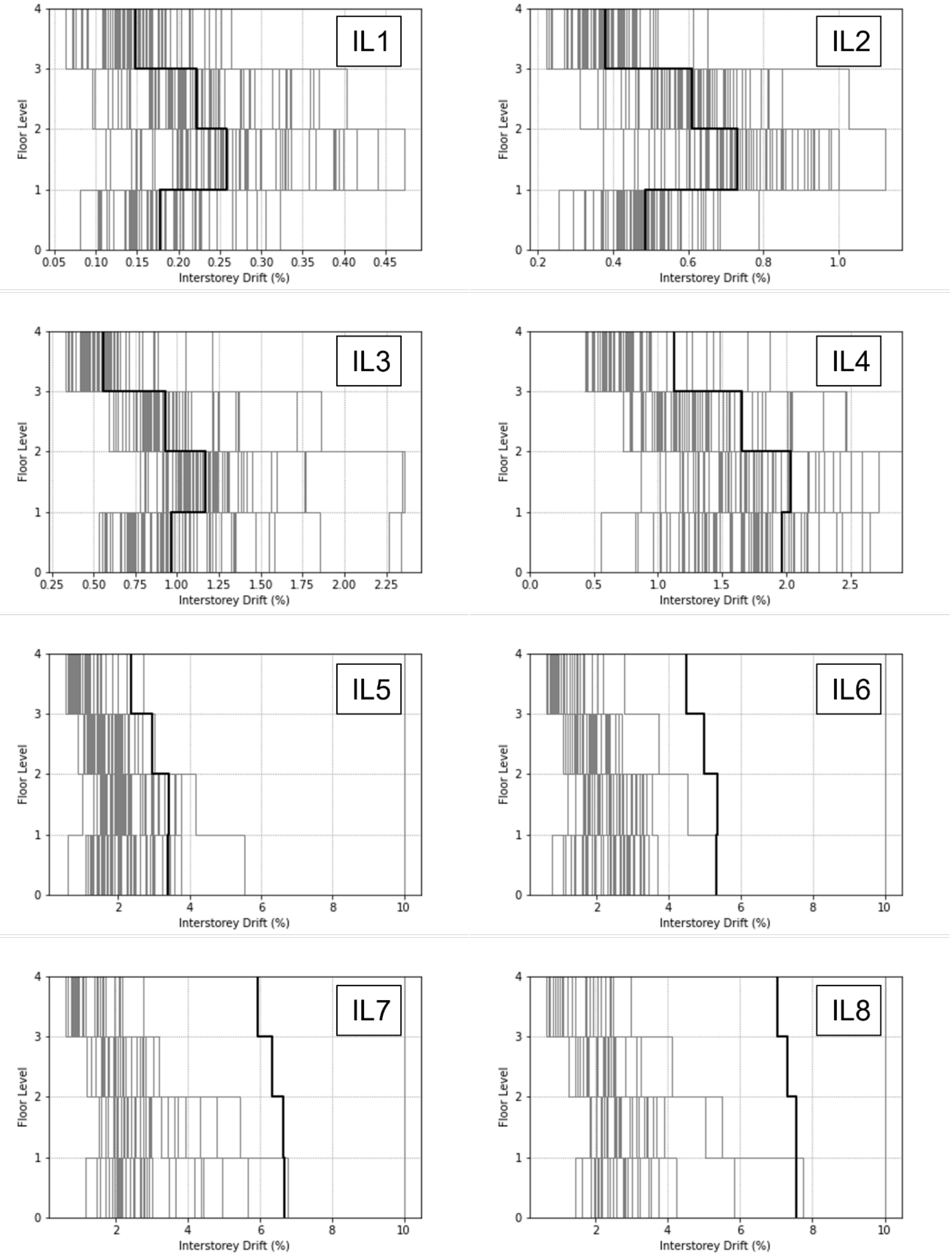


Figure C- 1: Interstorey Drift Results for New Zealand Design

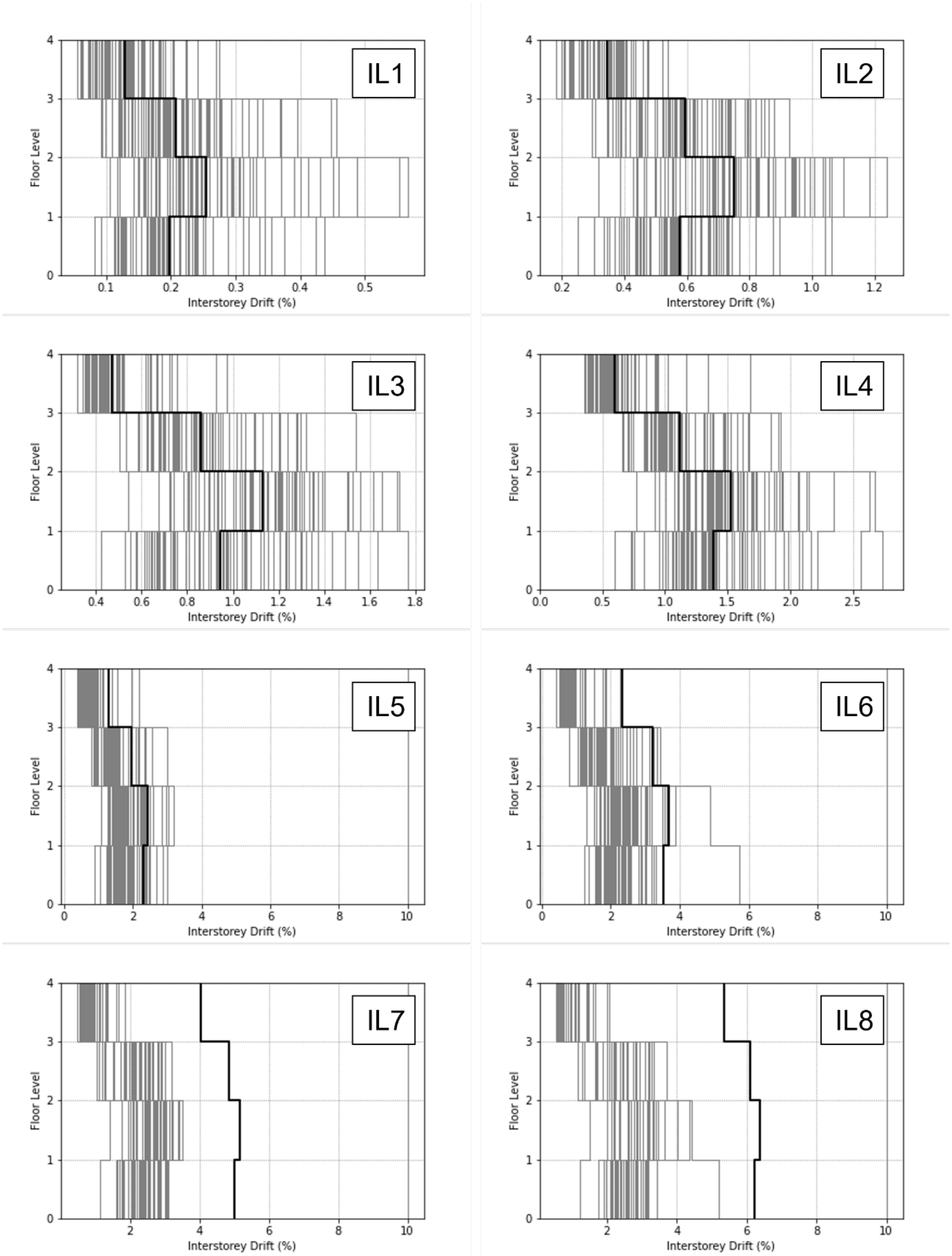


Figure C- 2: Interstorey Drift Results for Japanese Design

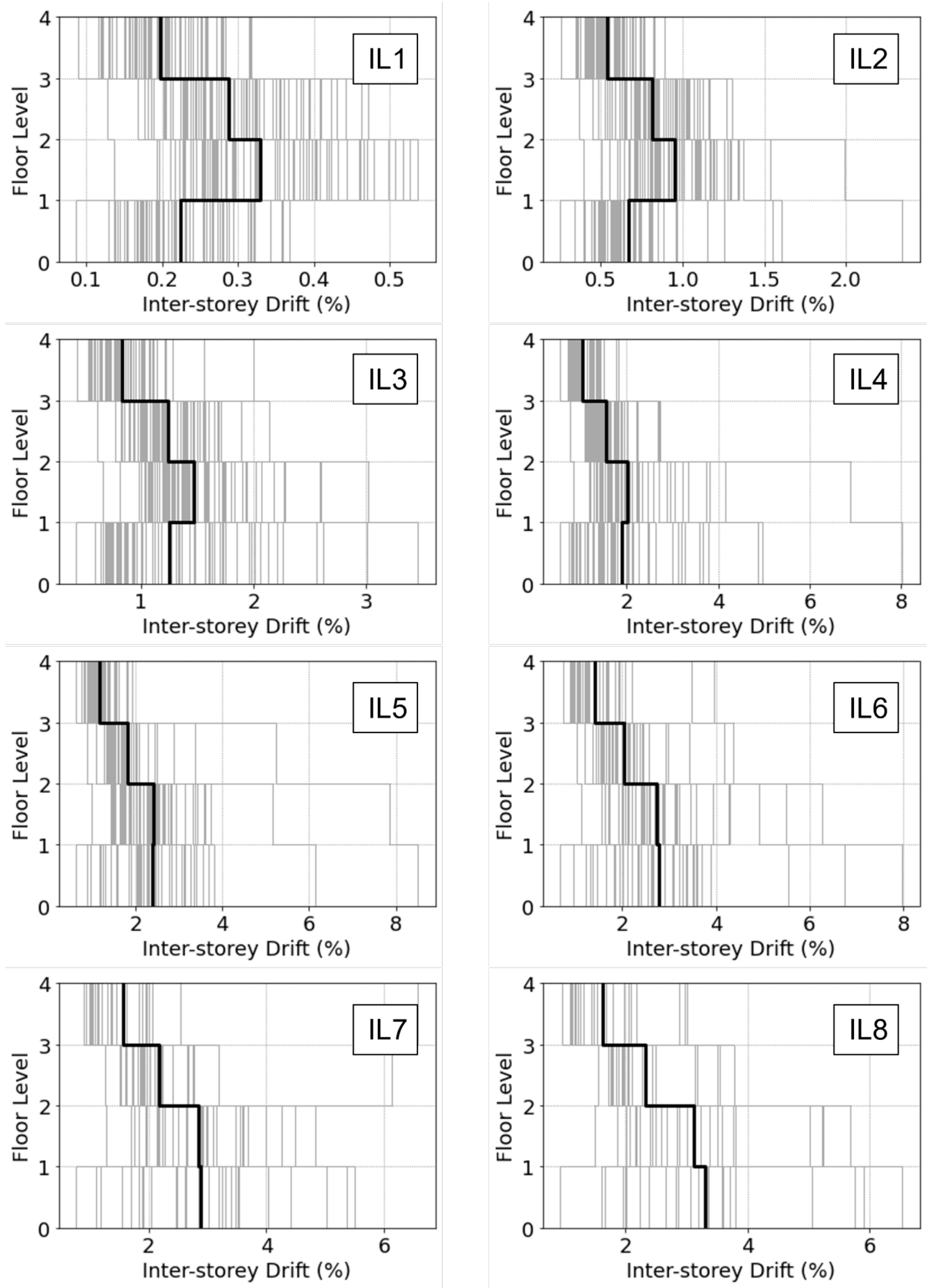


Figure C- 3: Interstorey Drift Results for United States Design

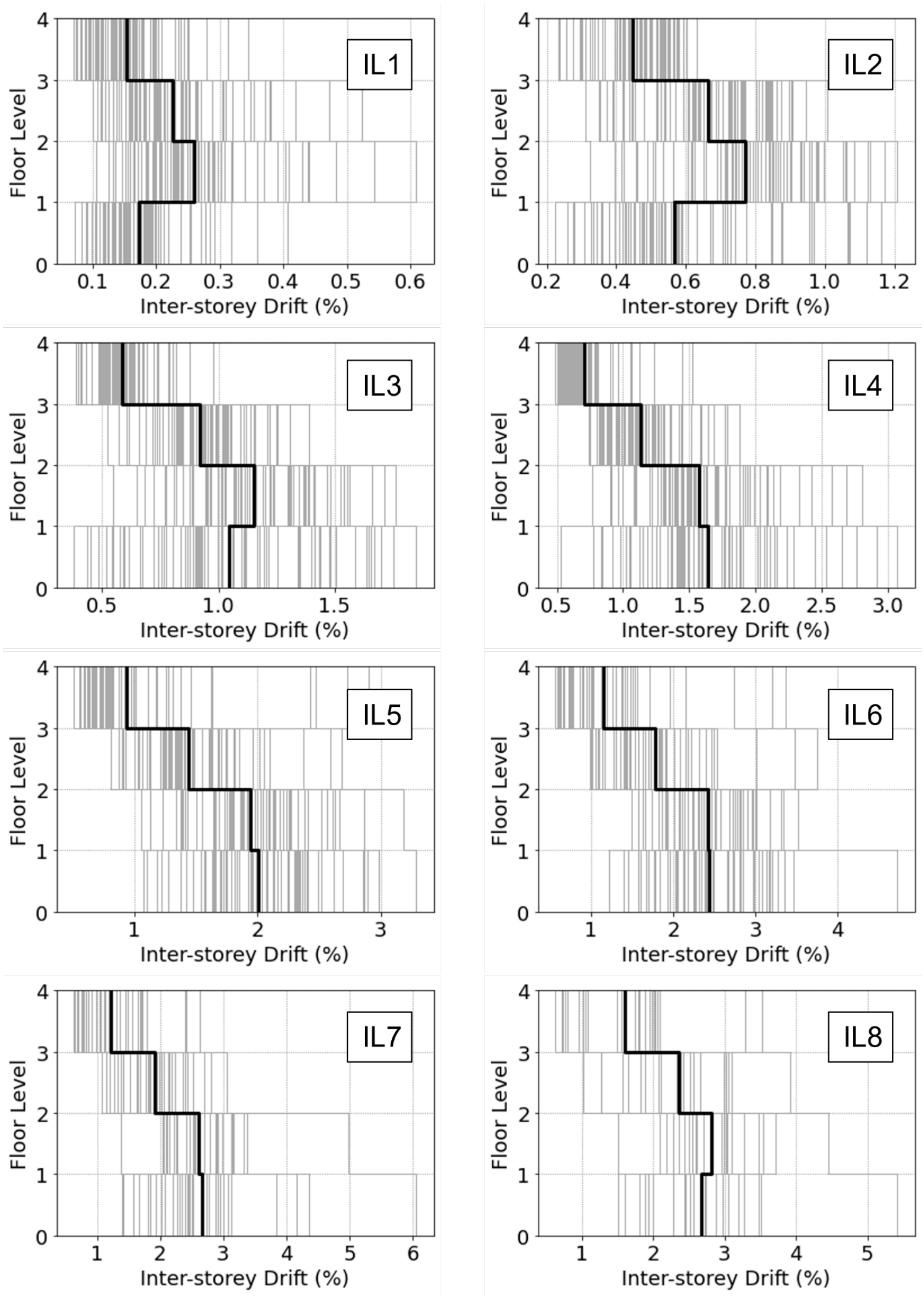


Figure C- 4: Interstorey Drift Results for Chilean Design

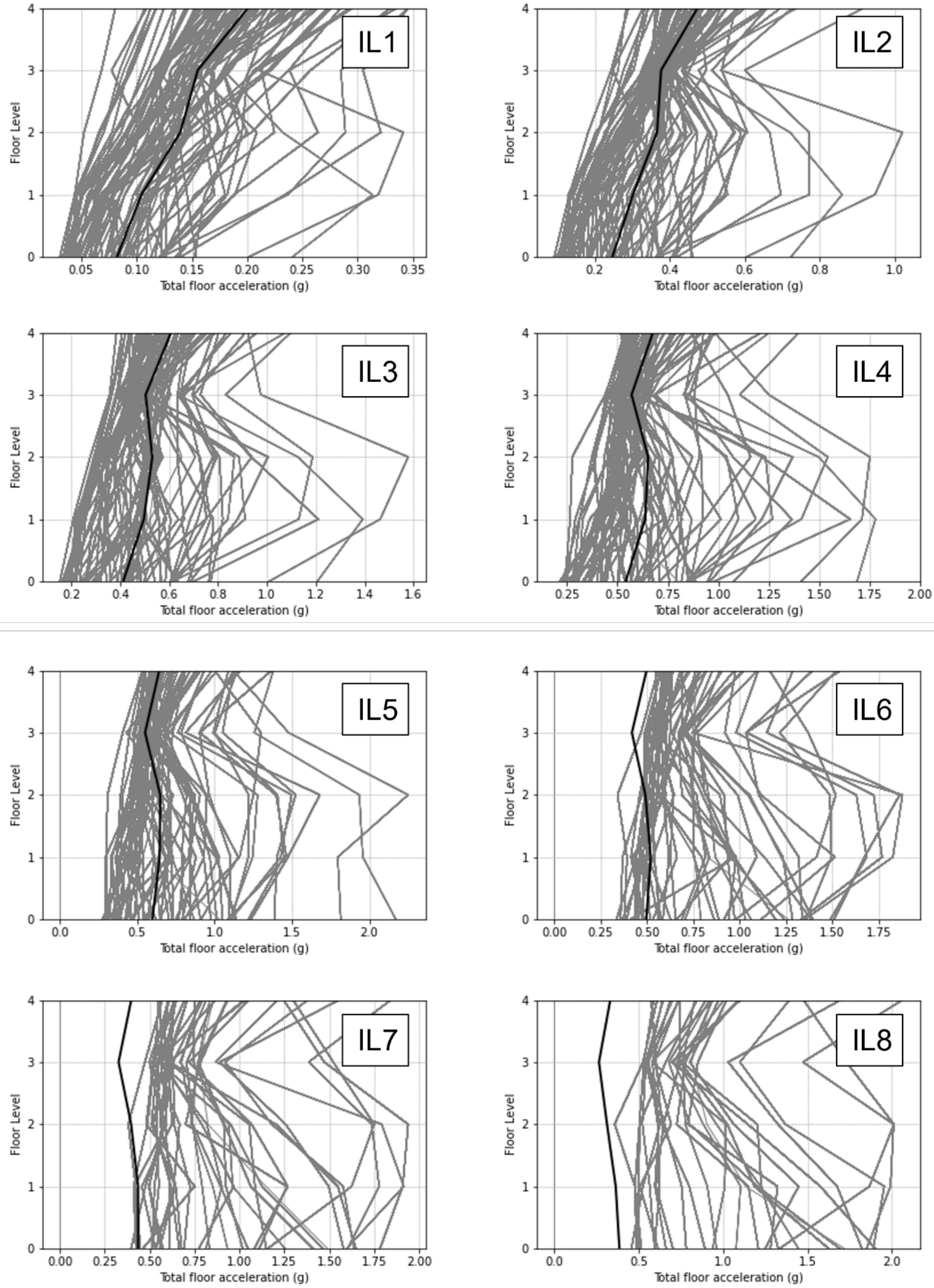


Figure C- 5: Total Floor Acceleration Results for New Zealand Design

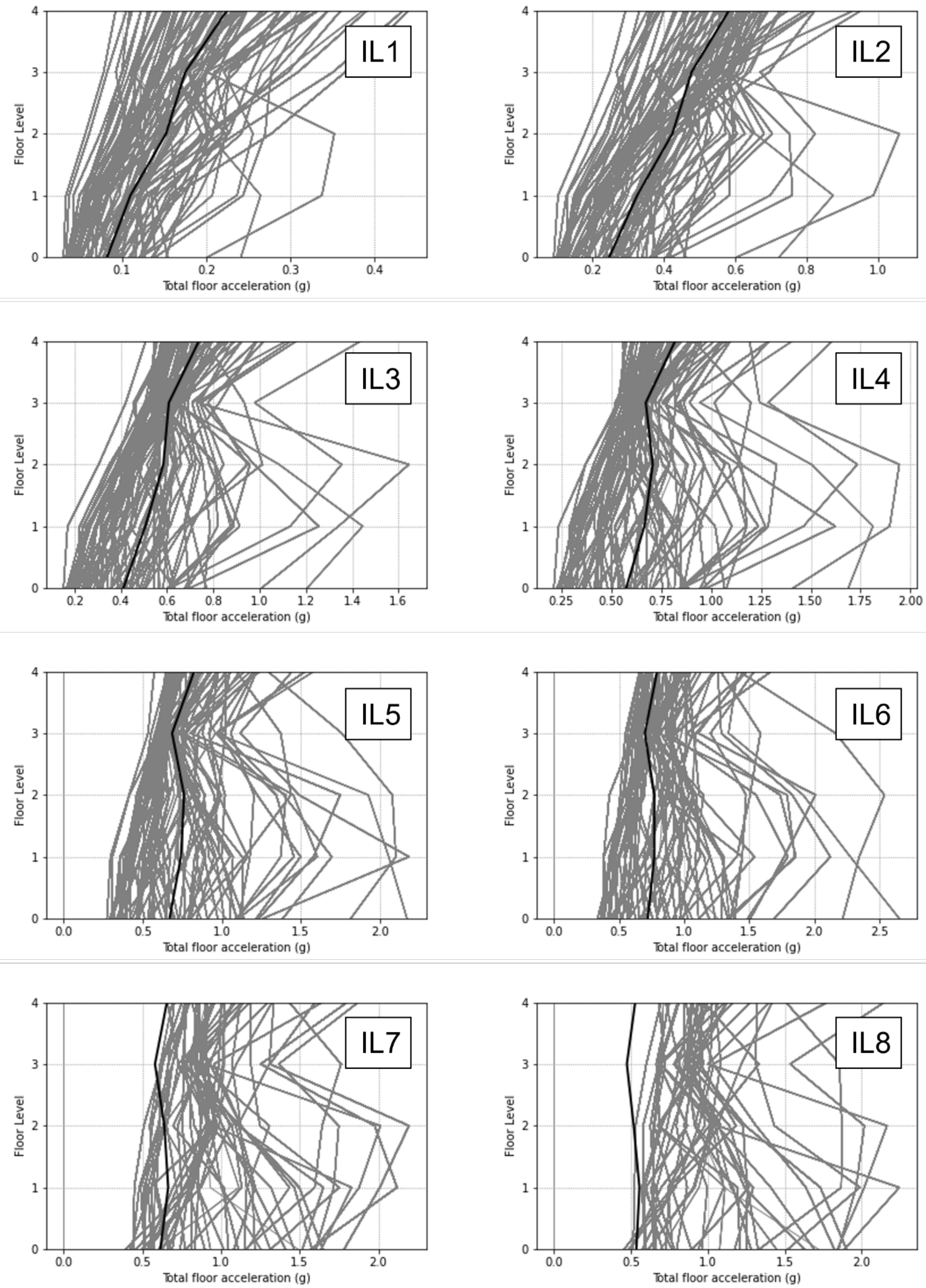


Figure C- 6: Total Floor Acceleration Results for Japanese Design

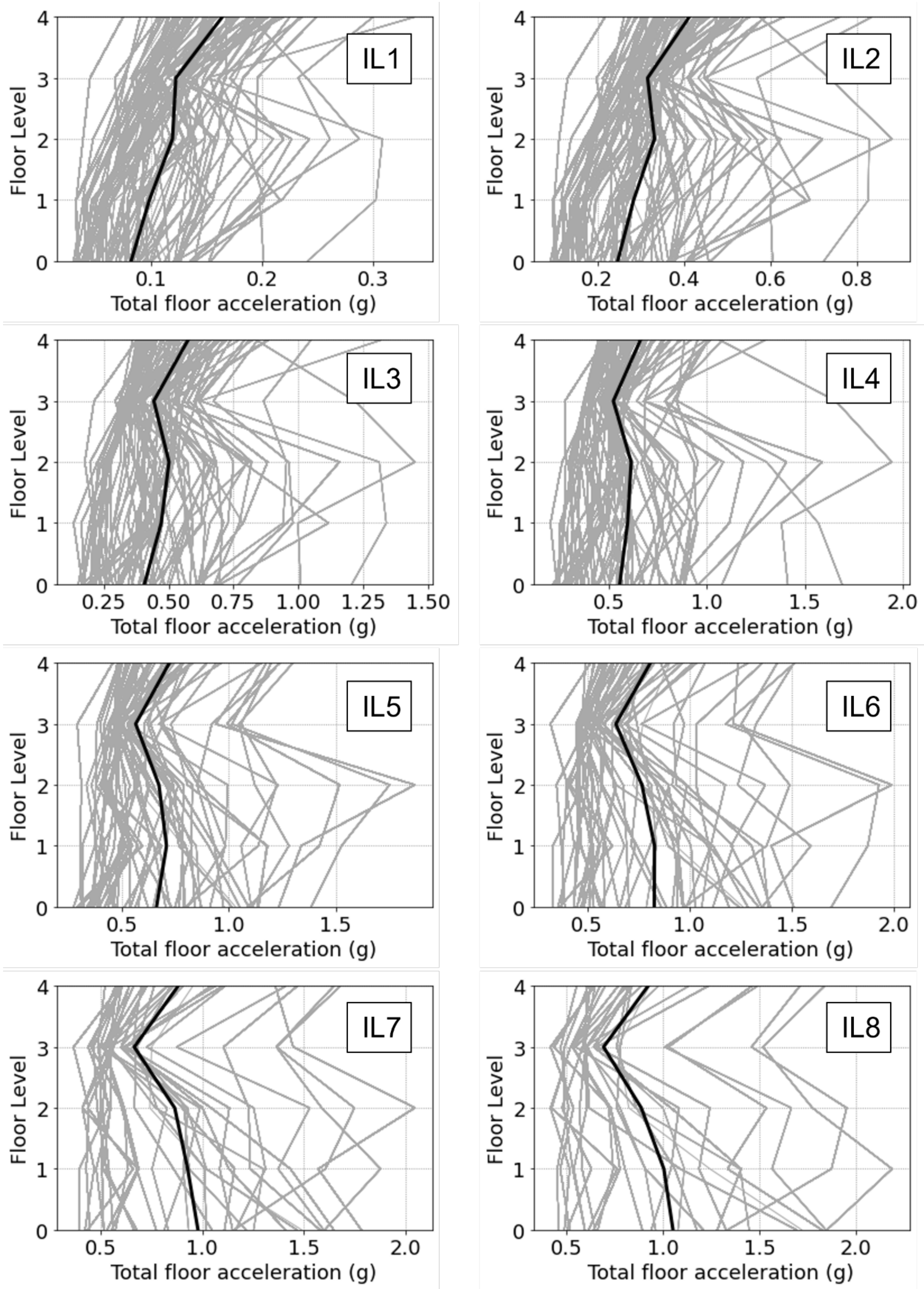


Figure C- 7: Total Floor Acceleration Results for United States Design

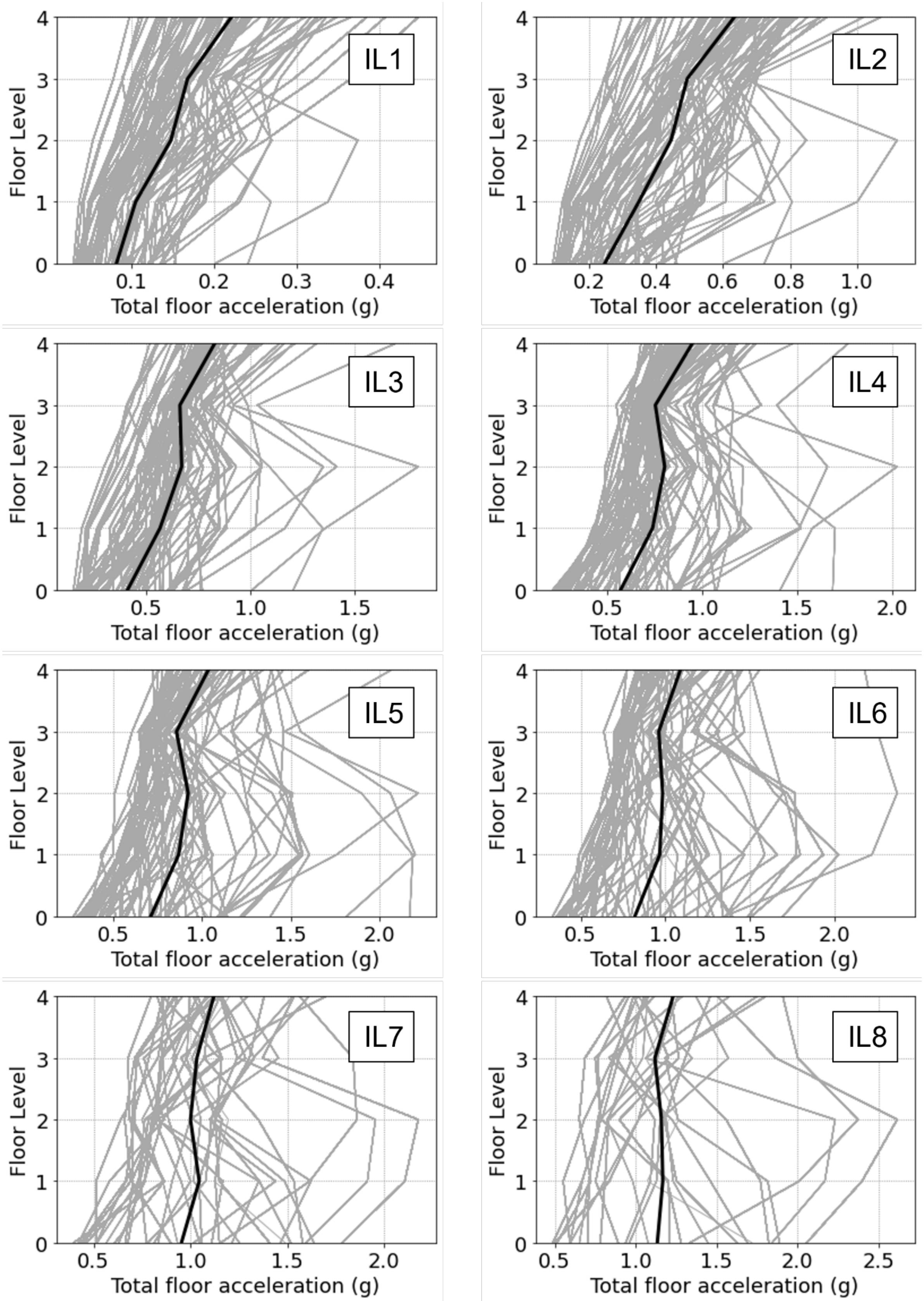


Figure C- 8: Total Floor Acceleration Results for Chilean Design

Appendix D: Building Fitout for Non-structural components

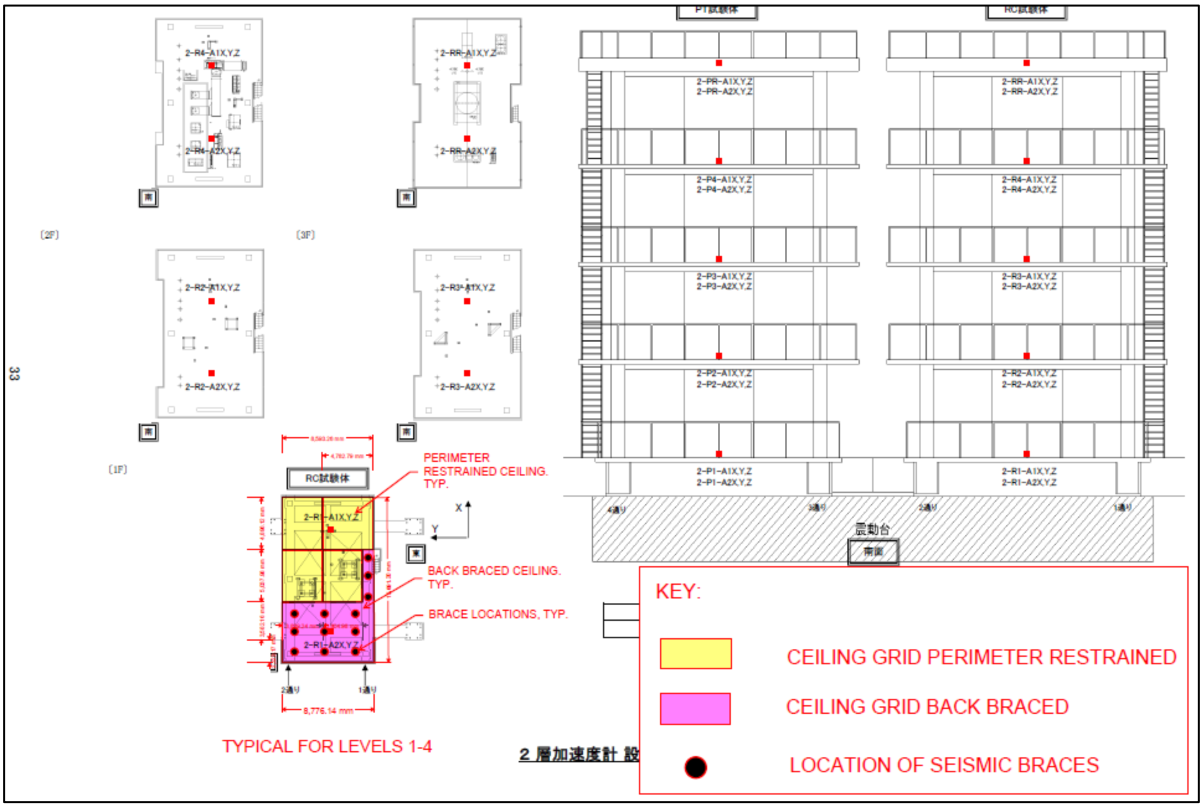


Figure D- 1: Ceiling Layout (Clarke 2022)

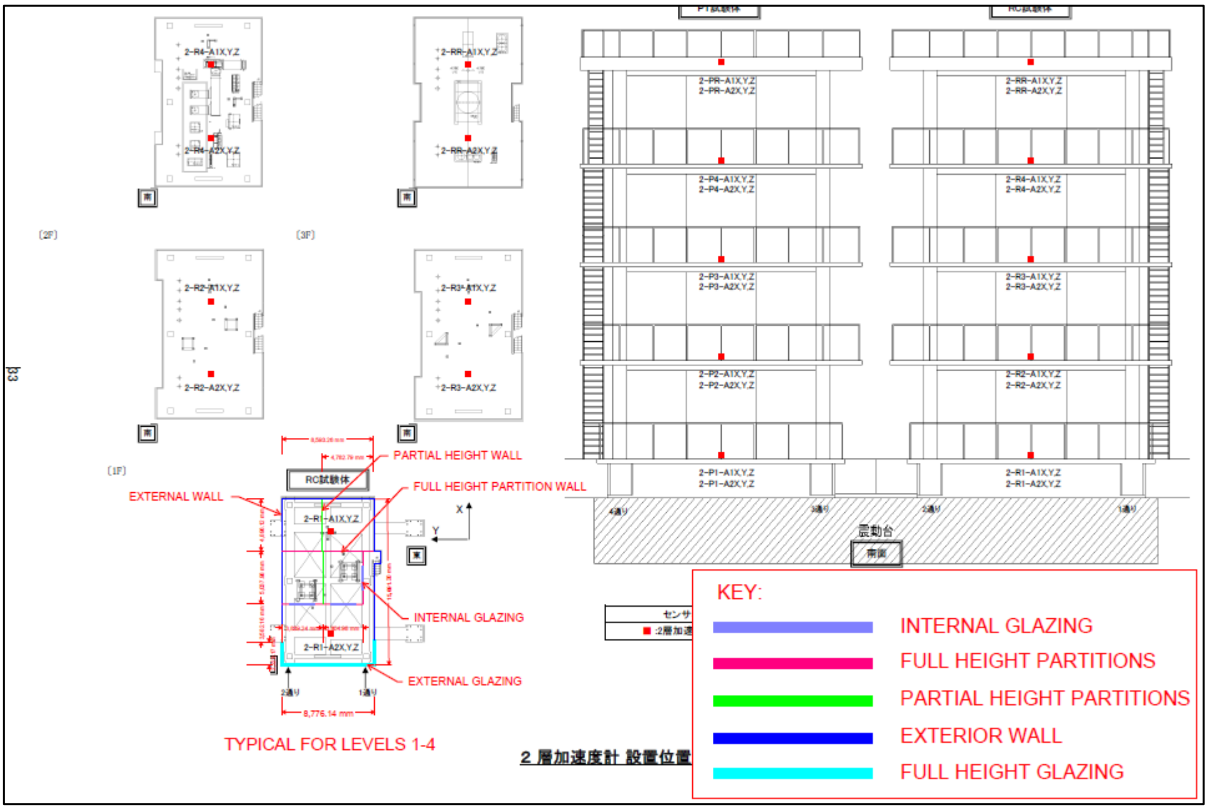


Figure D- 2: Partition Wall Layout (Clarke 2022)

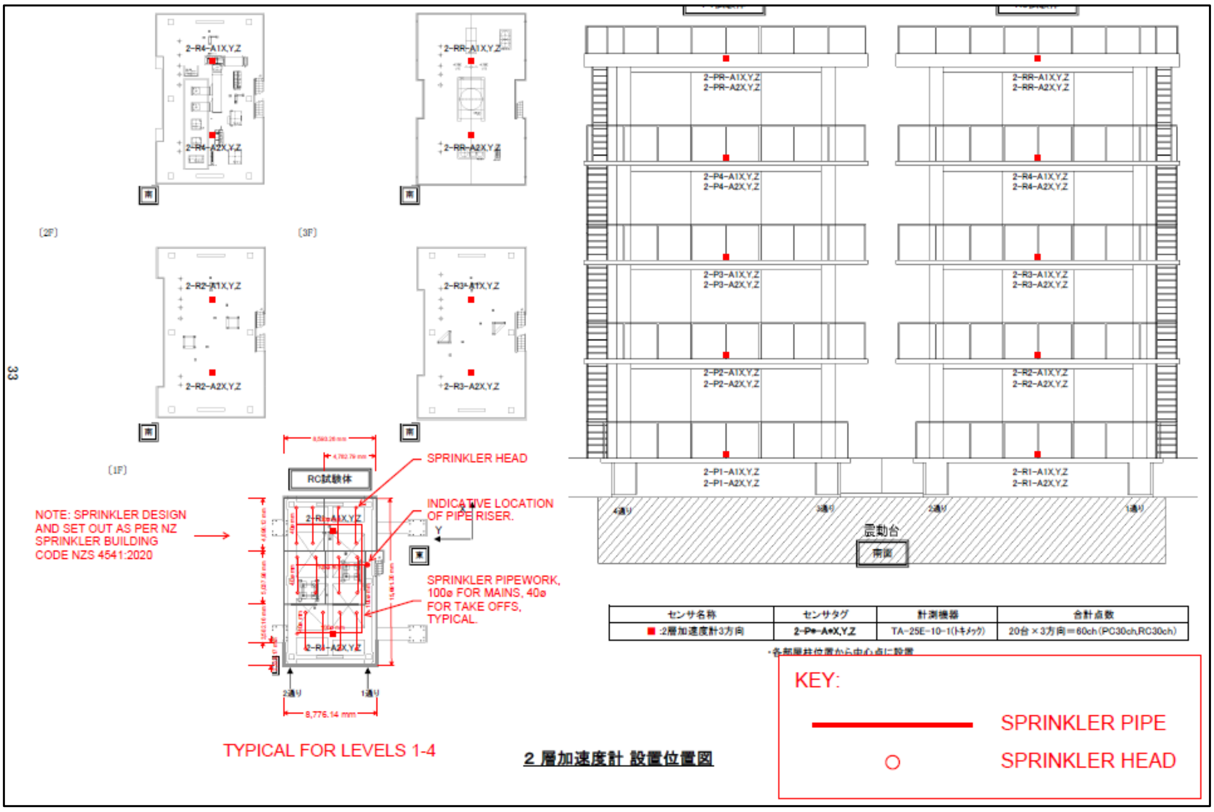


Figure D- 3: Sprinkler Layout (Clarke 2022)

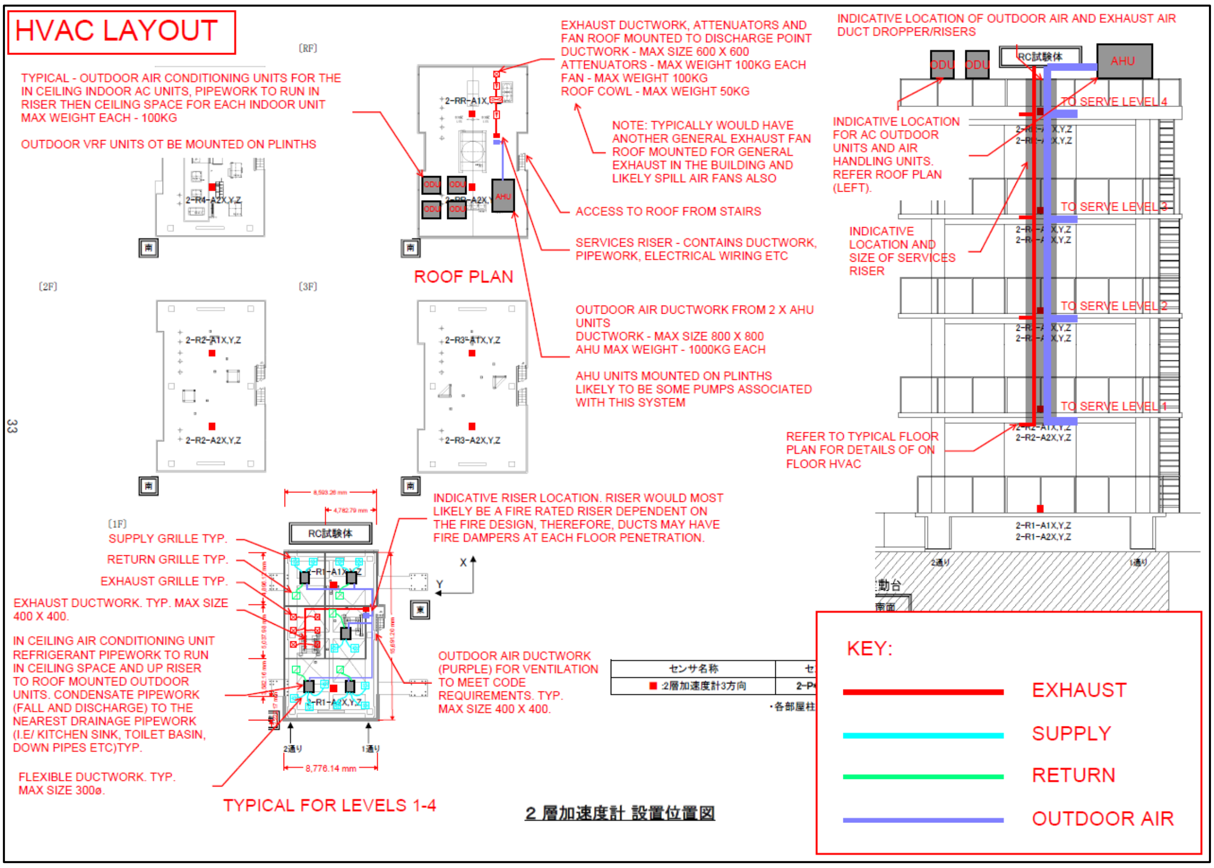


Figure D- 4: HVAC Layout (Clarke 2022)

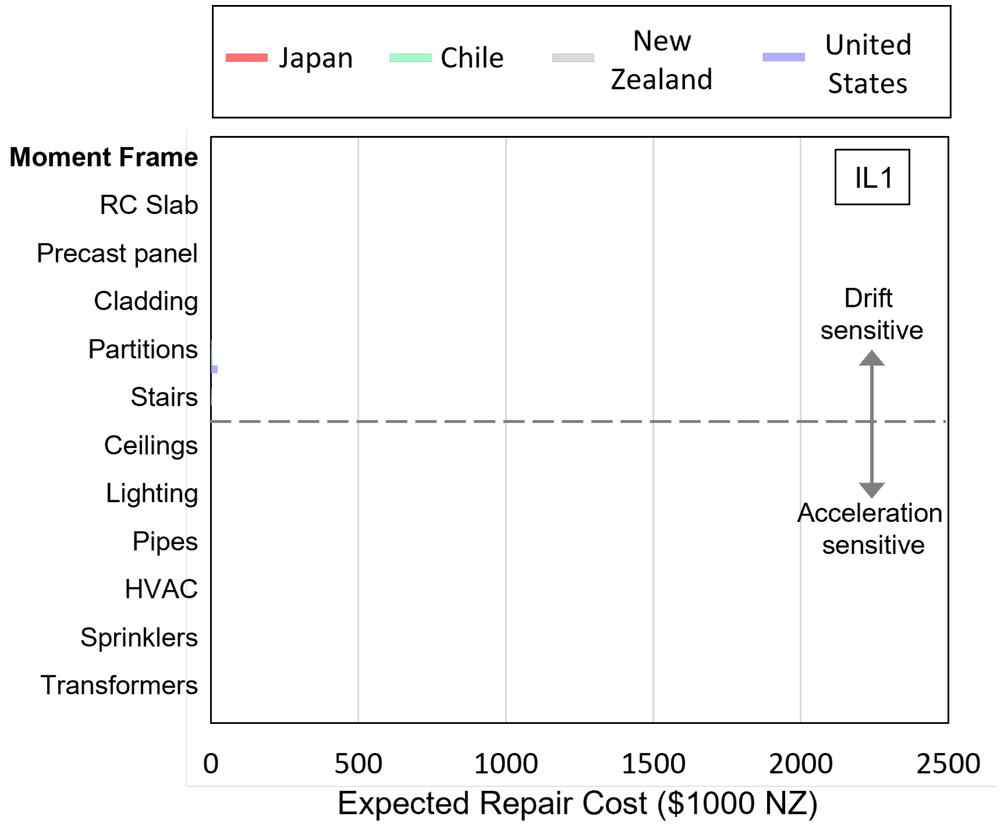
Appendix E: Building Fitout Information

Table E- 1: Fragility and consequence functions for structural and non-structural components

Fragility ID	Component	Fragility Functions			Repair/Replacement cost ($USD) [upper - lower]		
		DS1	DS2	DS3	DS1	DS2	DS3
B1041.001a	RC Moment Frame	0.02 rad	0.0275 rad	0.05 rad	25,704 - 17,136	38,978 - 25,985	MDS1: 47,978 - 31,985 MDS2: 38,978 - 25,985
B1049.011	RC floor slab	0.03 rad	0.048 rad	-	46,787 - 31,815	51,787 - 35,215	-
B2011.201a	Exterior walls (Pre-cast cladding panels)	0.008*	0.009*	0.012*	3,000 - 750*	9,800 - 2,450*	20,600 - 5,150*
B2022.001	Exterior glazing	0.0338 rad	0.0383 rad	-	2,055 - 1,096	2,055 - 1,096	-
B2023.001	Glazing partitions	0.029 rad	0.0473 rad	0.07 rad	924 -539	0	0
C1011.001a	Full-height partitions	0.005 rad	0.01 rad	0.021 rad	2,677 - 1,428	6,825 - 3,640	10,500 - 7,437
C1011.001b	Partial-height partitions	0.01 rad	0.013 rad	0.018 rad	3,570 - 1,071	9,100 - 2,730	17,500 - 5,250
C2011.011b	Stairs	0.005 rad	0.017 rad	0.028 rad	1,400 - 420	7,400 - 2,220	45,900 - 13,770
C3032.001a	PR Ceiling	1.17 g	1.58 g	1.82 g	435 - 290	3,405 - 2,270	7,005 - 4,670
C3032.003b	BB Ceiling	1.47 g	1.88 g	2.03 g	1,740 - 522	13,620 - 4,086	28,020 - 8,406
C3034.001	Light fittings	0.6 g	-	-	2.3 - 0.69	-	-
D2021.013b	Water pipe system	1.5 g	-	-	418 - 342	-	-
D2031.013b	Sanitary pipe system	2.25 g	-	-	800 -240	-	-
D3041.011c	HVAC - ducts < 6ft^2	1.5 g	2.25 g	-	715 - 585	6,985 - 5,715	-
D3041.032c	HVAC - grilles and difussers	1.5 g	-	-	3,300 - 2,700	-	-
D3041.041b	HVAC - In-celing units	1.9 g	-	-	16,500 - 13,500	-	-
D3052.013e	HVAC - air handling units	1.54 g	-	-	MDS1: 1,650 - 1,350 MDS2: 64,900 - 53,100	-	-
D4011.023a	Fire Sprinklers - pipes	1.5 g	-	-	MDS1: 385 - 315 MDS2: 2,915 - 2,385	-	-
D4011.033a	Fire Sprinklers - drops	0.95 g	-	-	550 - 450	-	-
D5011.013e	Transformers 100 to 350 kVA	3.05 g	-	-	26,675 - 21,825	-	-

*These values were obtained from (Baird 2014)

Appendix F: Additional Results from Loss Assessment



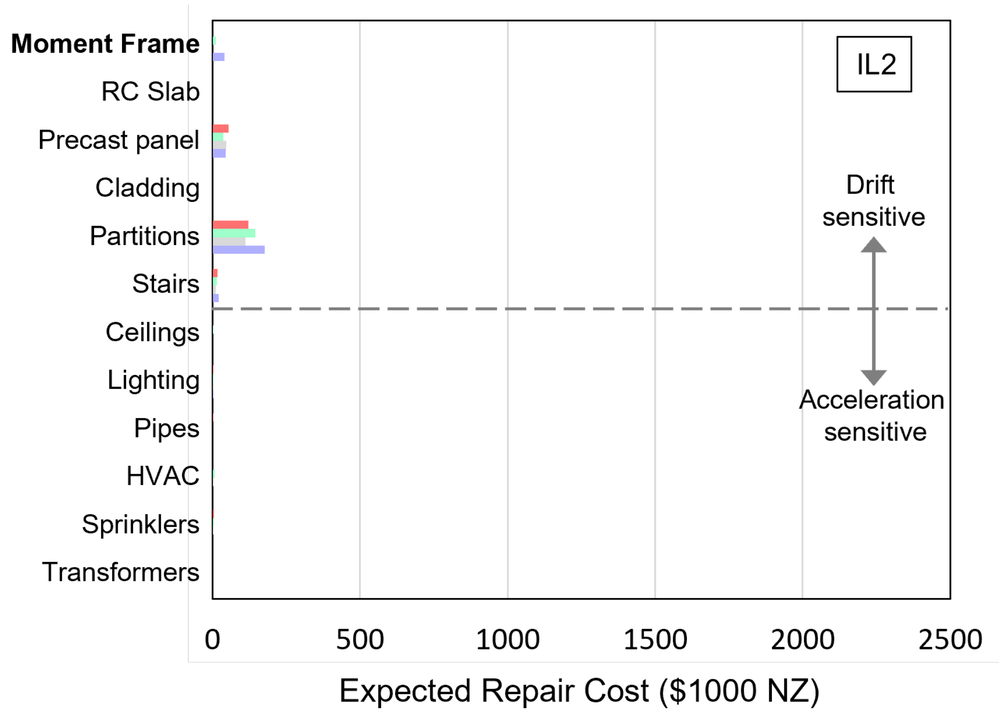


Figure F- 1: Component Repair Costs for IL1 and IL2

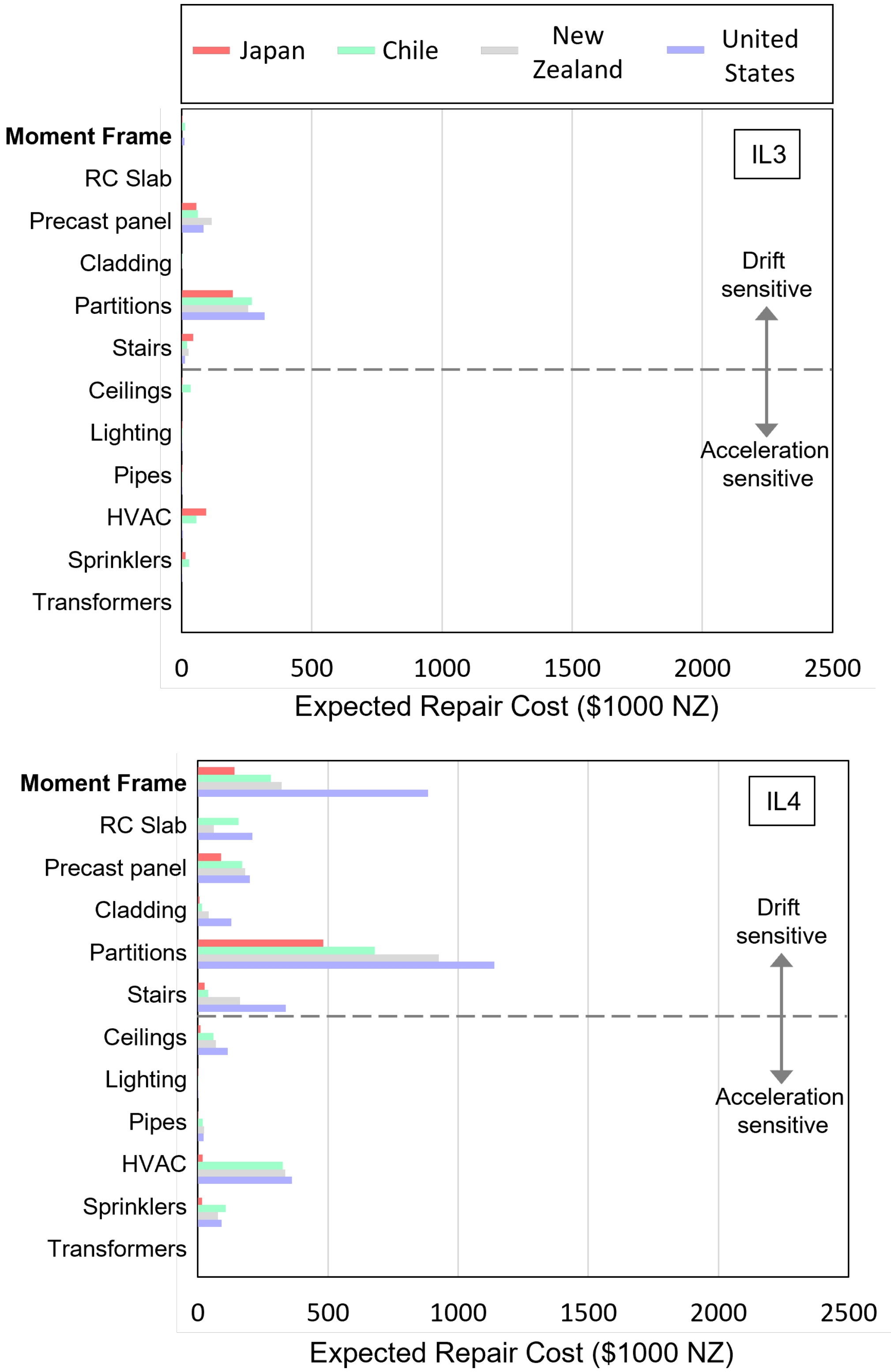


Figure F- 2: Component Repair Costs for IL3 and IL4

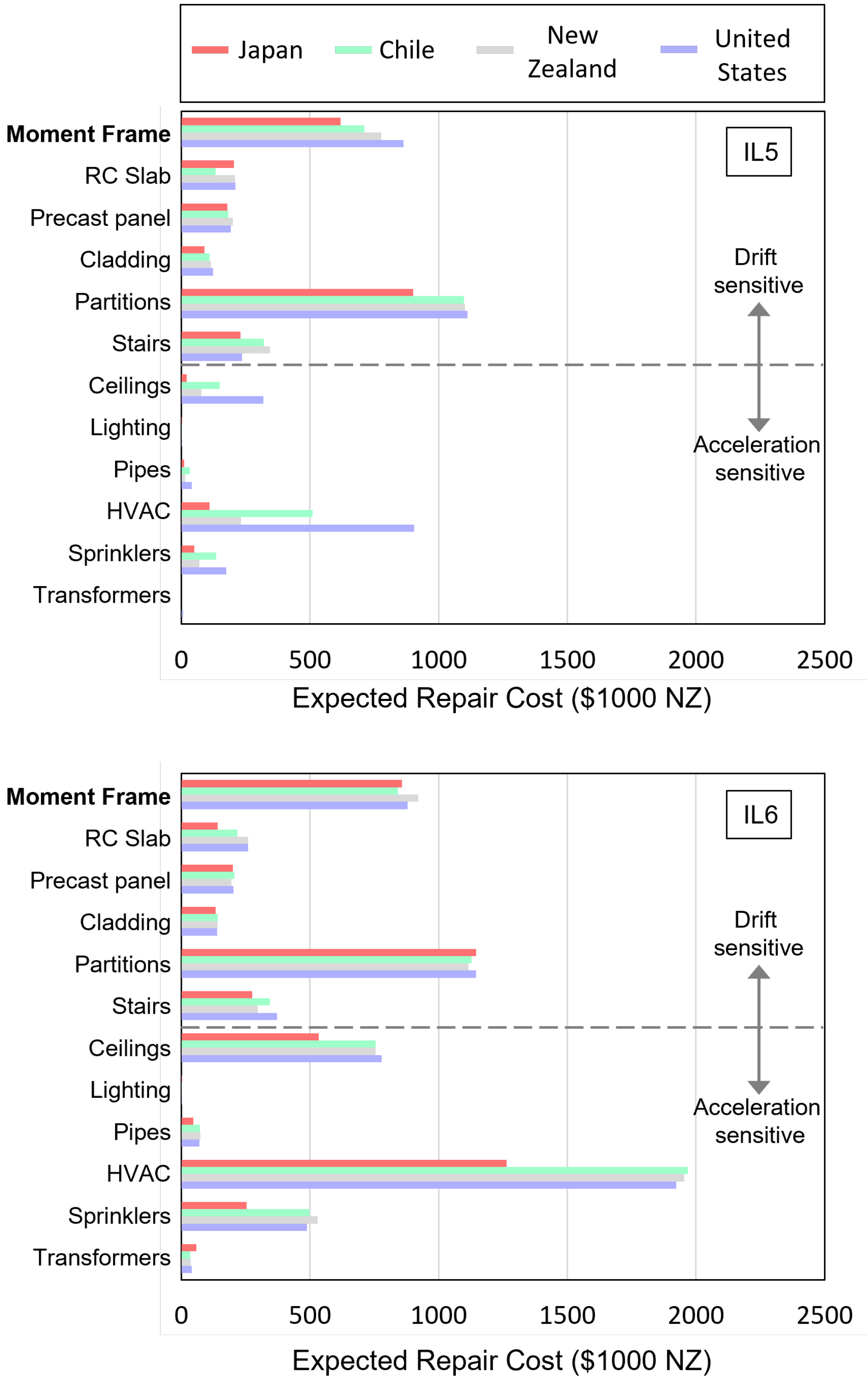


Figure F- 3: Component Repair Costs for IL5 and IL6

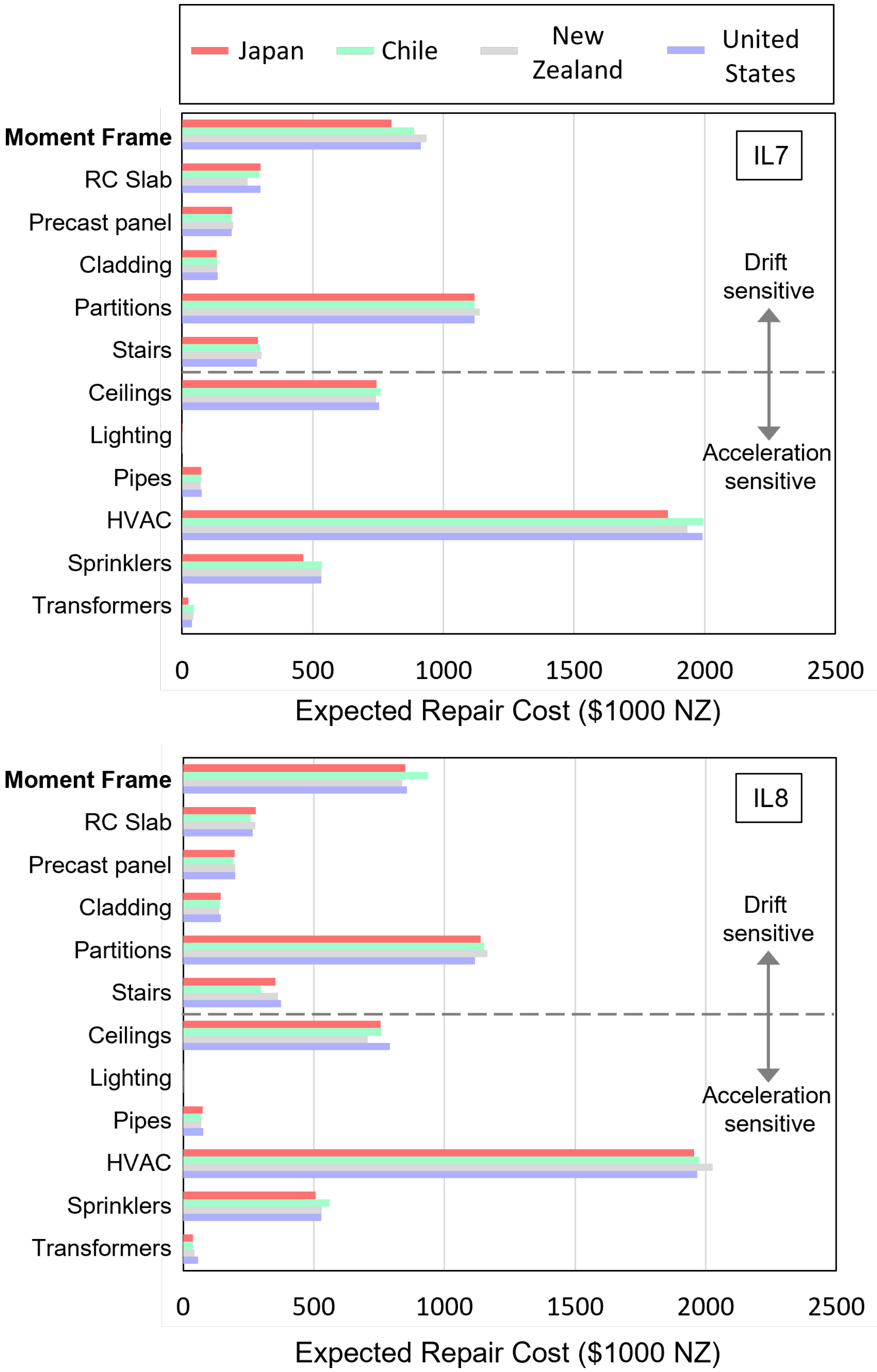


Figure F- 4: Component Repair Costs for IL7 and IL8