Seismic performance comparison of New Zealand and Japanese concrete moment frames

N. Buck, G. Clarke, L. Hogan & M. Stephens
University of Auckland, Auckland

ABSTRACT

Observations of the performance of reinforced concrete (RC) buildings after the 2016 Kumamoto earthquake in Japan showed buildings designed using Japanese standards resulted in 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 paper compares the seismic performance of RC moment frame buildings designed according to seismic design philosophies from New Zealand and Japan. Specifically, a case study building located in Dunedin was designed using New Zealand material properties, but with scaled seismic demands and design requirements based on the New Zealand and Japanese standards. The performance of the two buildings was compared using a suite of 78 ground motions selected for various magnitudes of expected seismic hazards in Dunedin.

The Japanese moment frame design was controlled by a 0.5% drift limit per Japanese Building Standard Law (BSL) whereas the New Zealand moment frame design was controlled by gravity loading and minimum detailing per NZS 3101. The Japanese requirements result in moment frames with larger section sizes, but lower reinforcement ratios and smaller beam-to-column strength ratios compared to frames designed to New Zealand specifications. For moderate earthquake intensities, the New Zealand frame had larger peak median inter-storey drifts with increasingly larger differences compared to the Japanese frame with not much difference in peak median floor accelerations.

1 INTRODUCTION

Over the past several decades, modern code-conforming buildings around the globe 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, New Zealand employs a capacity-based design approach that relies on inelastic ductile behaviour to allow cost savings in the form of smaller section sizes. In contrast, 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 0.7 s (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 of structures designed to earthquake standards in different countries to help inform 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. That is the objective of this work. Here, a four-storey case study building located in Dunedin was designed using New Zealand material properties, but with scaled seismic demands and design requirements based upon New Zealand and Japanese standards.

The main objective of this research is to quantify and compare the seismic performance of concrete buildings designed to the New Zealand and Japanese standards by answering the following questions:

1. How does Japanese seismic design differ from seismic design in New Zealand?
2. What are the main factors from each country driving the design of concrete moment frames?
3. What are the impacts on peak inter-storey drift and floor acceleration when designing to the two standards?

2 BACKGROUND

2.1 Previous code comparisons

Previous studies have shown the seismic demands used to design buildings can vary drastically from one country to another due to differences in site soil classification, parameters used to define seismic hazard, and resulting design response spectra (Khose et al., 2012). This fundamental difference in establishing seismic demands between various countries presents a significant challenge to identify specific design strategies that result in better or worse building performance. Fenwick et al. (2002) compared the resulting seismic demands, stiffness, drift, and ductility of reinforced concrete moment frames designed to New Zealand, United States (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.

Additionally, 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, the moment frames in this study were designed using the NZS 1170.5 elastic response spectrum (NZS 2004) as the starting point for seismic demands.

2.2 Key differences between New Zealand and Japanese design

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 main differences between the seismic codes from New Zealand and Japan are in predicting seismic hazard, using the hazard to calculate seismic demands, and building performance objectives. These differences stem from general design principles that focus on strength and stiffness verses energy dissipation and ductility. Specifically, the procedure for seismic design in Japan for buildings under 60 m tall follows an allowable stress design with two verification levels. In the first level (Level 1), allowable stress is checked against elastic demands induced from a moderate seismic event comparable to a design limit state with a 1/500-year return period in New Zealand (Narafu et al., 2017). Allowable stress limits depend on loading and material and are equal to yield stress (f_y) for steel and two-thirds compressive stress ($\frac{2}{3}f_c'$) for concrete under combined gravity and earthquake loading. 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 under Level 1 demands equal to 0.5% for frames.

3 METHODOLOGY

3.1 Case study building

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.

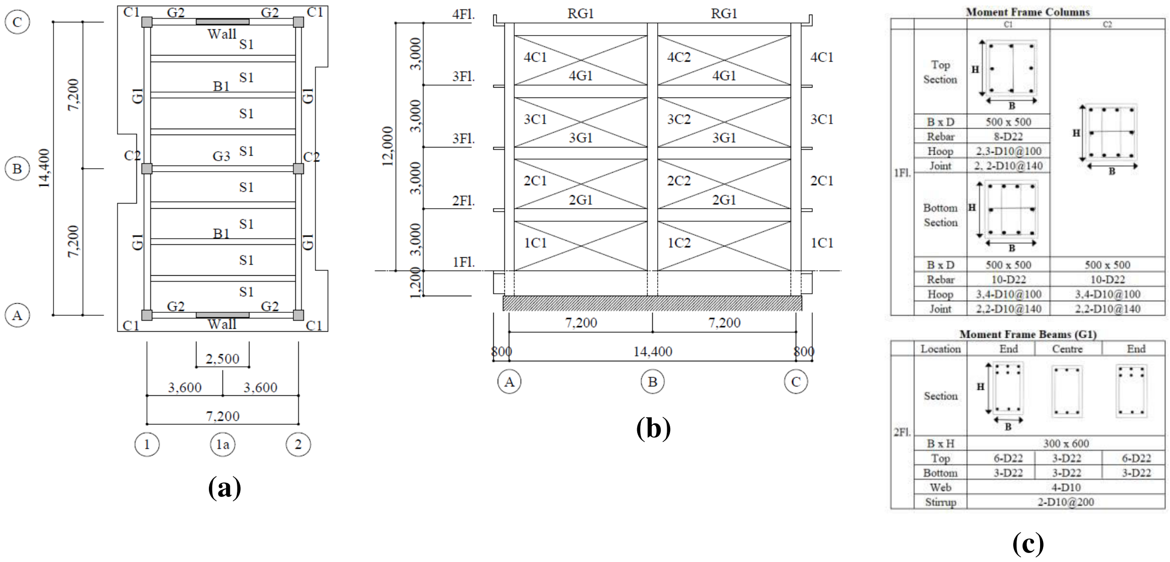

Figure 1: Case study building; (a) plan view, (b) moment frame elevation, and (c) moment frame sections Note: all dimensions are in mm (modified from Nagae et al., 2015)

The building has plan dimensions 14.4 m by 7.2 m (Figure 1a) with 3 m storey heights (Figure 1b) and a 130 mm thick floor slab cast monolithically with beams, columns, and walls. Typical moment frame sections for beams and columns are shown in Figure 1c where hoop and joint transverse reinforcement for columns is specified with number of legs in the H, B 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.

3.2 Moment frame designs

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 New Zealand and Japanese standards. The location and soil conditions in 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 as determined from Nagae et al. (2015). Ultimately, the base shear demand on the case study moment frame is comparable to the ULS (500-year, R = 1.0) elastic demands calculated using NZS 1170.5 if the building were located in Dunedin (Z = 0.13) with site class C soil conditions.

Table 1 summarizes the Japanese and New Zealand moment frame designs in terms of design base shear, controlling load combination, section size, and longitudinal reinforcing, as well as the code requirements controlling the various design features.

Table 1: Summary of moment frame designs

Design Feature	Japanese Design	New Zealand Design
Design base shear (kN)	357	78
Load combination	G + Q + E	1.2G + 1.5Q*
Beam size (mm)	300 x 600 0.5% allowable drift**	300 x 500 Column detailing requirements**
Beam reinforcement (no. top/bot)	(4-6/3) D25 – Asymmetric top/bottom Strength requirements from pushover analysis**	(4/4) D20 – Symmetric top/bottom Strength requirements from gravity analysis**
Column size (mm)	500 x 500 0.5% allowable drift**	500 x 500 Beam depth**
Column reinforcement (total no.)	(8-10) HD20 – each face Demands from pushover**	(12-16) HD16 – each face Vertical joint shear**

* Gravity demand exceeded earthquake load combination

** Code requirement driving aspect of design

The resulting New Zealand and Japanese moment frame designs are relatively similar with slight differences in beam depth and beam and column reinforcing despite a significant difference between the design base shears. The factors controlling the design of the moment frames were gravity loading and minimum detailing requirements for New Zealand and a 0.5% elastic drift limit for Japan. Specifically for the New Zealand design, the maximum ratio of longitudinal column bar diameter to beam depth for interior joints specified by NZS 3101 (NZS 2006) 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 NZS 3101 capacity design procedure for ductile moment frames.

Figure 2 shows the sum of column-to-beam moment strength ratios at each joint with the New Zealand design ratios in black on the left side of the joint and the Japanese design ratios in grey on the right side of the joint. The New Zealand design has larger column-to-beam moment strength ratios compared to the Japanese design due to smaller beam sections and stronger columns. Although both designs utilize 500 x 500 mm columns, New Zealand capacity design procedures and vertical joint shear requirements result in columns needing additional longitudinal bars and greater moment capacity compared to the Japanese columns.

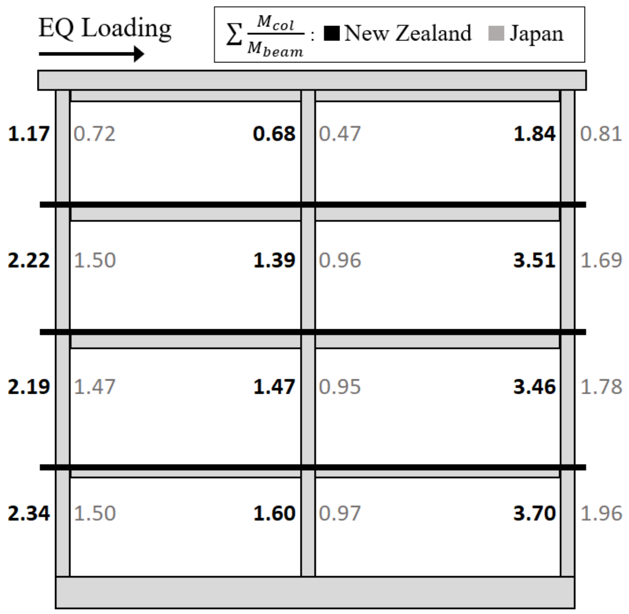

Figure 2: Column-to-beam moment strength ratios for New Zealand and Japanese moment frame designs

Finally, although not explicitly required by the Japanese 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 mechanisms.

3.3 Structural modelling and analysis

To quantify and compare the performance of the two designs, an incremental dynamic analysis (IDA) was performed using nonlinear two-dimensional (2D) models developed in the opensource structural analysis software OpenSeespy (Zhu et al., 2018). A lumped moment rotation approach was used because previous research has shown it can effectively capture inelastic drift, storey acceleration and collapse (Haselton et al., 2007). Beams and columns were modelled as elastic with an effective stiffness equal to 0.4 times gross moment of inertia, and nonlinear moment rotation hinges at member ends with hinge behaviour based on a peak oriented hysteric response defined using the parameters specified in ASCE 41-17 Tables 10-7 and 10-8 for beams and columns respectively (ASCE 2017).

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). For demonstration purposes, Figure 3 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 and the scaled values for intensity levels 1 (IL1, S_a = 0.1 g), 4 (IL4, S_a = 0.7 g) and 8 (IL8, S_a = 1.5 g) are shown in grey. This approach was selected based on the lack of site-specific ground motion data available at the time of this investigation during the update to the New Zealand National Seismic Hazard Model. A total of 39 pairs of ground motions (in two directions) from the expanded ATC-63 "Far-Field" ground motion set (Haselton and Deierlein, 2007) were scaled at increasing $S_a(T_1)$ intensities for a fundamental period equal to the average of the moment frame designs T_1= 0.85 s (shown by the red dashed line in Figure 3b). The corresponding scale factor is applied to the ground motion input for nonlinear time history analysis of the moment frames.

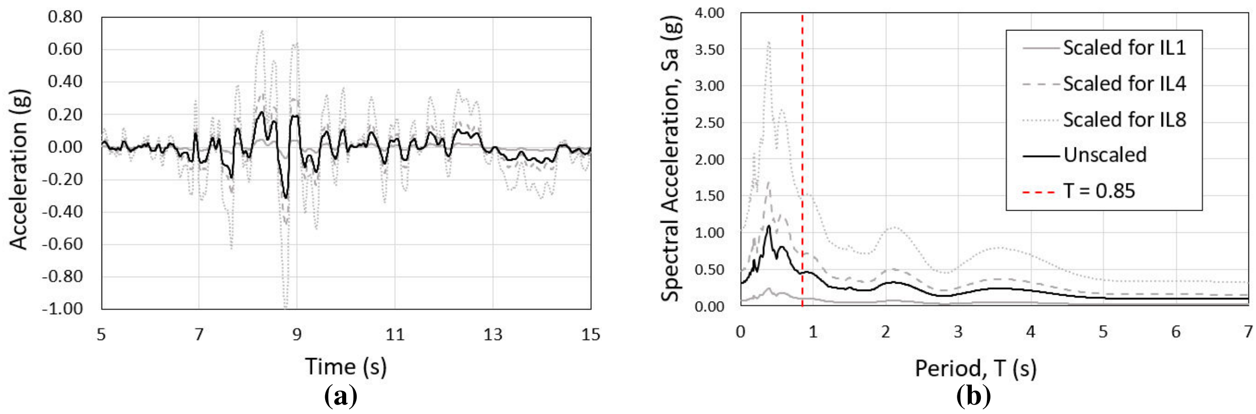

*Figure 3: Example of IDA scaling process; **(a)** unscaled and scaled ground motion input and **(b)** corresponding unscaled and scaled response spectrums*

4 RESULTS

Figure 4 shows the IDA curves for the Japanese and New Zealand design moment frames analysed for each ground motion record at increasing intensities. Each point on the IDA curves represents the maximum inter-storey drift from non-linear time history analysis and the corresponding scaled spectral acceleration intensity at the fundamental period of the structure $S_a(T_1)$. 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 Dunedin (Z = 0.13) with site class C soil conditions.

Overall, both moment frames performed well in terms of collapse prevention considering the average collapse intensity for both designs were well above the ULS (0.3 g) and MCE (0.6 g) intensities, however the Japanese design moment frame performed slightly better with an average collapse intensity equal to 1.6 g compared to 1.4 g for the New Zealand design. The low magnitude of the ULS and MCE hazards relative to the mean collapse intensity in both cases clearly demonstrates the designs were not controlled by seismic forces.

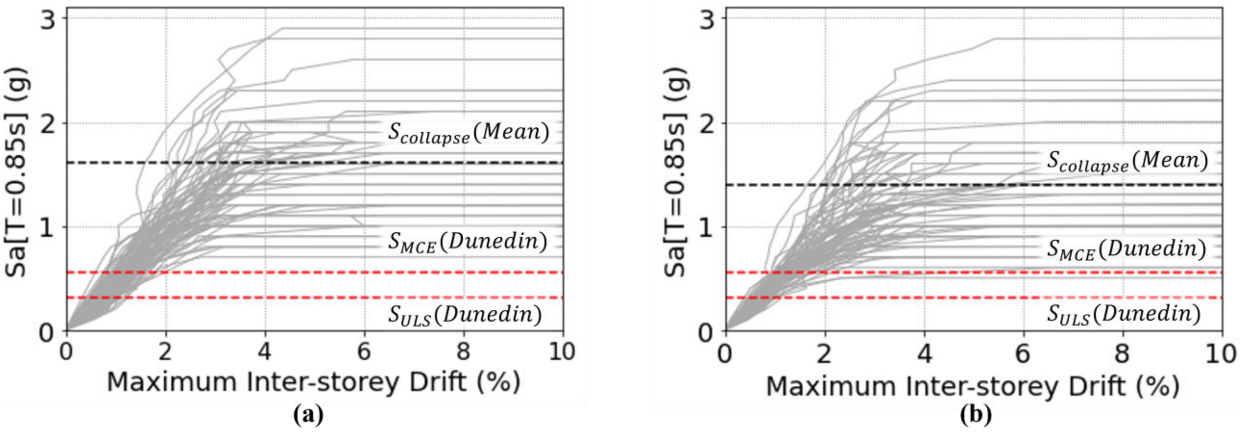

*Figure 4: IDA curves; **(a)** Japanese design and **(b)** New Zealand design moment frames*

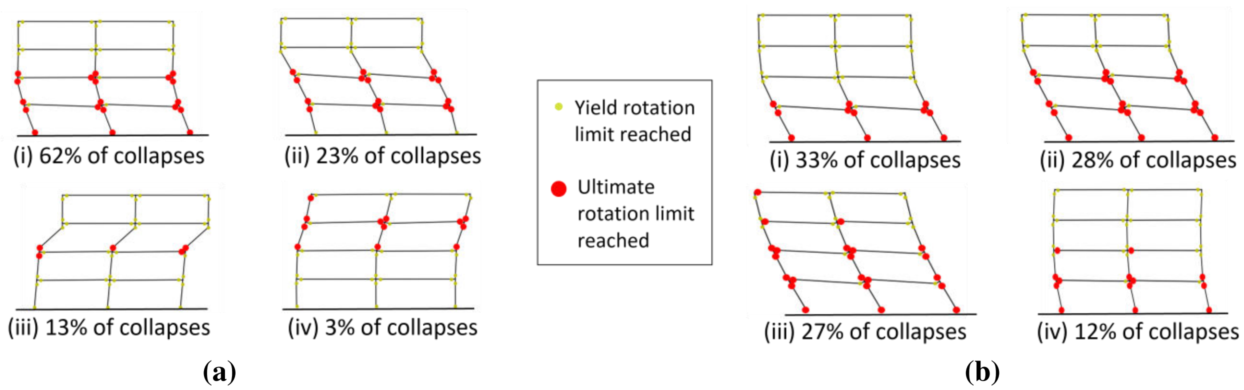

*Figure 5: Collapse mechanism results; **(a)** Japanese design and **(b)** New Zealand design moment frames*

Figure 5 shows the percentages of different collapse modes for each design where the small green circles represent beam or column yield rotation, and the large red circles indicate beam or column ultimate (post-capping) rotation. The most common collapse mechanism for the Japanese design was beam-column story mechanisms on the lower and middle floors compared to more distributed beam-column hinging for the New Zealand design. The difference in collapse mechanisms stem from asymmetrical top and bottom beam reinforcement (as shown in Figure 1c) and lower column-to-beam strength ratios (as shown in Figure 2) for the Japanese design compared to the New Zealand design.

The peak mean inter-storey drift and floor acceleration results from IDA are found by taking the average from all 39 orthogonal ground motion sets at a single intensity level. The maximum value from all floors represents the peak mean value of drift or acceleration. These values are shown in Figure 6 with the peak mean values at each intensity level for each design shown in Figure 6a and the relative difference between those values (e.g. New Zealand peak drift minus Japanese peak drift) at each intensity level shown in Figure 6b. The numbers at each point on the curves in Figure 6a correspond to the intensity level from IL1 ($S_a(0.85)$ = 0.1 g) to IL8 ($S_a(0.85)$ = 1.5 g). Although the section sizes are not much larger for the Japanese design, the resulting peak inter-story drifts are generally smaller compared to the New Zealand design at IL4, IL5, and IL7 with not much difference between total floor accelerations. This is further demonstrated in Figure 6b which shows at intensity levels IL2 to IL5, the difference between peak drifts continues to increase from 0.02% to 0.31% while the difference between peak accelerations stays approximately the same at 0.12 g.

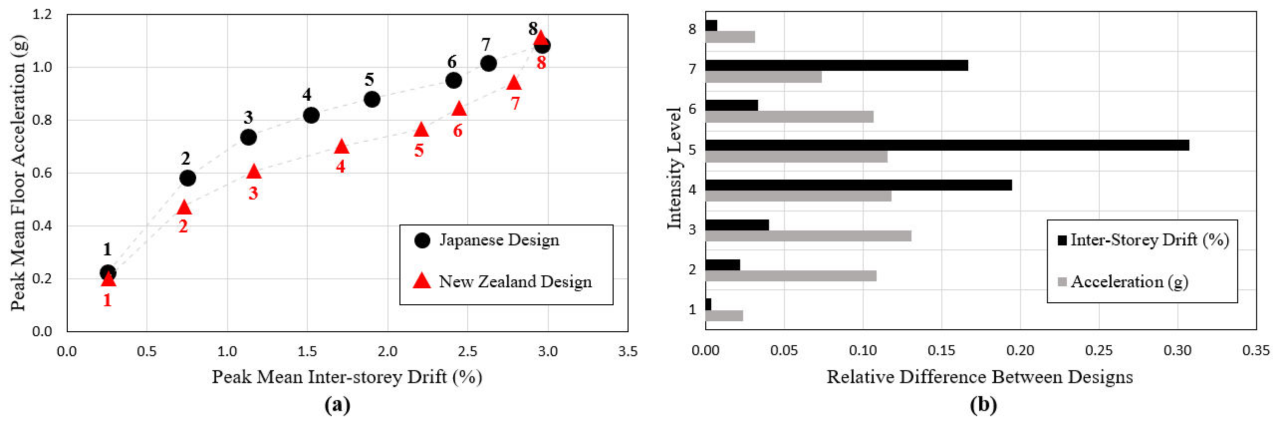

*Figure 6: IDA results; **(a)** Peak mean inter-storey drift vs acceleration at each intensity level and **(b)** Relative difference of peak mean inter-storey drift and acceleration between designs at each intensity level*

5 CONCLUSIONS

This study investigated the seismic performance of concrete moment frames designed to New Zealand and Japanese standards. The frames were designed with material properties and seismic hazards for a building located in Dunedin, NZ with site class C soil conditions but with scaled seismic demands and design requirements following New Zealand and Japanese 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.

Based on the case study discussed here, it was found that:

- For RC buildings located in low seismic zones in New Zealand, moment frame design following New Zealand requirements is controlled by gravity loading and minimum detailing while the same moment frame following Japanese requirements is controlled by a 0.5% elastic drift limit.
- Asymmetrical top and bottom beam reinforcement used in Japanese design results in disproportional hinge development in beam sections and can inhibit distributed beam-column collapse mechanisms at moderate earthquake intensities.
- New Zealand capacity design requirements and symmetrical top and bottom beam reinforcement result in moment frames with larger column-to-beam strength ratios that are likely to have a more distributed beam-column collapse mechanism compared to Japanese designed moment frames.
- Despite having similar section sizes, moment frames designed to Japanese specifications will have a considerable reduction in peak inter-storey drifts, but similar peak floor accelerations compared to moment frames designed to New Zealand specifications.

Given these conclusions, the next stage of this work will evaluate a case study building located in a location where seismic forces rather than gravity demands control the New Zealand moment frame design.

REFERENCES

AIJ (2019). *"AIJ Standard for Structural Calculation of Reinforced Concrete Structures"*. Architectural Institute of Japan, Tokyo, Japan. 77pp.

Anderson JC, Bertero VV, Hart GC, Krawinkler H and Moehle JP (1992). *"Design Guidelines for Ductility and Drift Limits"*. Report for the Kajima Research Project, California Universities for Research in Earthquake Engineering (CUREe), 215pp.

Aninthaneni PK and Dhakal RP (2016). "Prediction of fundamental period of regular frame buildings". *Bulletin of the New Zealand Society for Earthquake Engineering*, **49**(2): 175-189.

ASCE (2017). *"ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings"*. American Society of Civil Engineers, Restin, Virginia.

Bradley BA, Razafindrakoto HNT and Nazer MA (2017). "Strong ground motion obersvations of engineering interest form the 14 November 2016 Mw7.8 Kaikoura, New Zealand earthquake". *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(2): 85-93.

BSL (2016). *"The Building Standard Law of Japan"*. The Building Center of Japan (BCJ), Tokyo, Japan.

Fenwick R, Lau D and Davidson B (2002). "A comparison of the seismic design requirements in the New Zealand loadings standards with other major design codes". *Bulletin of the New Zealand Society for Earthquake Engineering*, **35**(3): 190-203.

Hampshire S, Zanaica L, Bucur C, Lima SDS and Arai A (2013). "Comparative study of codes for seismic design of structures". *Mathematical Modelling in Civil Engineering*, **9**(1): 1-12.

Hare J, Oliver S and Galloway B (2012). "Performance objectives for low damage seismic design of buildings". *New Zealand Society for Earthquake Engineering Conference*, Christchurch, New Zealand, 9pp.

Haselton CB and Deierlein GG (2007). *"Assessing Seismic Collapse Safety of Modern Reinforced Concrete Moment-Frame Buildings"*. PEER Report, Pacific Earthquake Engineering Research Center (PEER), 295pp.

Haselton CB, Liel AB, Lange ST and Deierlein GG (2007). *"Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings"*. PEER Report, Pacific Earthquake Engineering Research Center (PEER), 152pp.

Holden C, Kaiser A, Dissen RV and Jury R (2013). "Sources, ground motion and structural response characteristics in Wellington of the 2013 Cook Straight earthquakes". *Bulletin of the New Zealand Society for Earthquake Engineering*, **46**(4): 188-195.

Khose VN, Singh Y and Lang DH (2012). "A comparative study of design base sehar for rc buildings in selected seismic design codes". *Earthquake Spectra*, **28**(3): 1047-1070.

Nagae T, Ghannoum WM, Kwon J, Tahara K, Fukuyama K, Matsumori T, Shiohara H, Kabeyasawa T, Kono S, Nishiyama M, Sause R, Wallace JW and Moehle JP (2015). "Design implications of large-scale shake-table test on four-story reinforced concrete building". *ACI Structural Journal*, **112**(12): 135-146.

Narafu T, Ishiyama Y, Ison R, Sakuma J, Kato H and Kita S (2017). "Outline and features of Japanese seismic design code". *16th World Conference on Earthquake Engineering*, 1-10.

NZS (2004). *"NZS 1170.5: Structural Design Actions Part 5: Earthquake Actions - New Zealand".* Standards New Zealand, Wellington. 88pp.

NZS (2006). *"NZS 3101: Concrete Structures Standard".* Standards New Zealand, Wellington. 332pp.

Sarrafzadeh M, Elwood KJ, Dhakal RP, Ferner H, Pettinga D, Stannard M, Meada M, Nakano Y, Mukai T and Koike T (2017). "Performance of reinforced roncrete buildings in the 2016 Kumamoto earthquakes and seismic design in Japan". *Bulletin of the New Zealand Society for Earthquake Engineering*, **50**(3): 394-435.

Vamvatsikos D and Cornell CA (2001). "Incremental dynamic analysis". *Earthquake Engineering and Structural Dynamics*, **31**(491-514.

Yu G and Chock G (2016). "Compariosn of the USA, China and Japan seismic design procedures". *Civil Engineering Conference in the Asia Region (CECAR 7)*, Honolulu, Hawaii, 16pp.

Zhu M, McKenna F and Scott MH (2018). "OpenSeesPy: Python library for the OpenSees finite element framework". *SoftwareX*, **7**(6-11.