# Base isolated building performance and the impact of the national seismic hazard model

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## ABSTRACT

Seismic isolation design guidelines recently released by NZSEE and MBIE aim to provide more consistency in the design and performance of base-isolated buildings in New Zealand (NZ). This paper investigates the seismic performance of base-isolated buildings and compares 'minimally' designed base-isolated buildings with conventional building designs in Christchurch and Wellington. The 4-storey case study buildings are first designed according to the NZ seismic isolation guidelines for generic sites in Wellington and Christchurch. The performance of the buildings is then assessed in accordance with the FEMA P-58 approach using non-linear time history analysis in OpenSees and 180 pairs of ground motions, to provide an indication of expected annual losses (EAL). The results indicate that the EAL for base-isolated buildings is likely to be around half that of traditional buildings. In addition, the impact of the 2022 national seismic hazard model (NSHM) is investigated by comparing the EAL estimated using the 2010 NSHM versus the 2022 NSHM. Use of the 2022 NSHM increases the EAL by a factor of 2.60 and 1.77 for the base-isolated case study buildings in Christchurch and Wellington respectively. In contrast, the 2022 NSHM increases the EAL of conventional RC wall buildings by a factor 1.37 and 2.09 in Christchurch and Wellington respectively. The different impacts of the NSHM on EAL for buildings in Christchurch and Wellington are attributed to a larger increase in the annual probabilities of exceedance of moderate earthquakes in Christchurch compared to Wellington.

## 1 INTRODUCTION

Base isolation, also known as seismic isolation, is an established strategy to improve building performance (Christopoulos and Filiatrault, 2006; Skinner et al. 1993, and Stanton and Roeder, 1991). A base-isolated building generally consists of a foundation, isolation plane, rattle space, and superstructure. A layer of flexible isolators is placed at the isolation plane and result in an increase in the fundamental period of vibration of the isolated structure. From the response spectra point of view, period elongation can reduce the acceleration demand felt by the superstructure, it may also increase the over displacement demand. However, the benefit of a well-designed base-isolation system is that the majority of the displacement is concentrated in the isolation plane. In addition, isolators are designed to dissipate seismic energy or can be installed alongside damping devices. This further reduces the seismic response of the structure.

The design procedures for base isolated buildings are generally provided in design standards, such as ASCE/SEI 7-22 (ASCE/SEI, 2021) from the United States and Eurocode 8 (CEN, 2004) from the Europe. In New Zealand, base isolation is currently considered an alternative solution due to the lack of acceptable solutions or verification methods cited in the NZ building code. In 2019, the first version of the *Guideline for the Design of Seismic Isolation Systems for Buildings* was proposed by the New Zealand Society for Earthquake Engineering (NZSEE) for trial use (NZSEE/MBIE, 2019). The initiation of the guidelines is to provide some consistency in base isolation design across the country since base isolation is not formally documented as part of the Building Code. The guideline is relatively new, and some sections directly refer to the US and European codes. This research conducts a numerical study to investigate the following questions:

- How does an isolated building designed using the NZSEE/MBIE guidelines perform?
- How does the 2022 National Seismic Hazard Model (NSHM) affect our expectations for the performance of existing isolated buildings?

## 2 METHODOLOGY

To answer the questions posed above, case study buildings are designed according to NZ codes and guidelines and the performance is then assessed following the Pacific Earthquake Engineering Research (PEER) Centre's Performance-Based Earthquake Engineering (PBEE) approach. Porter (2003) provides an overview of this approach which also forms the basis of the FEMA P-58 methodology (FEMA, 2018a).

### 2.1 Case study buildings

A four-storey conventional building configuration, previously investigated by John et al. (2022), is isolated and redesigned to be used as the superstructure of this case study building. The building has an Importance Level Two (IL2). The superstructure has a footprint of 24m by 40m and four storeys, see Figure 1a. The height for the first storey is 4.5m and 3.6m for all the storeys above, with a total height of 15.3m. The lateral resisting system consists of six 3m-long RC walls in the X direction and two 6m-long RC walls in the Y direction. The isolation plane consists of 14 Lead Rubber Bearings (LRBs) and 8 Flat Sliders (FSs), see Figure 1b. The isolators are connected through grillage beams.

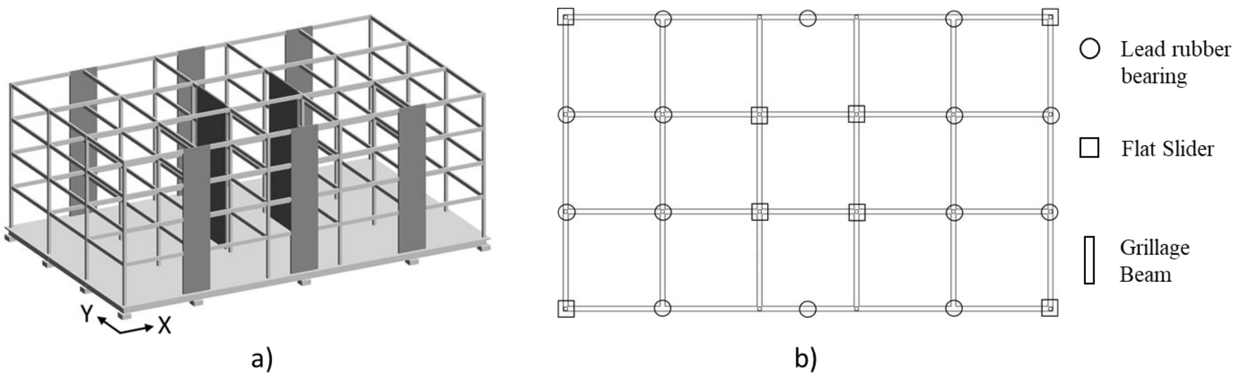


*Figure 1: Case study building: a) 3D view and b) isolation plane view*

## 2.2 Design criteria

Following the process recommended by the NZSEE/MBIE isolation design guidelines, the isolation plane is designed using the capacity-spectrum method, assuming the structure behaves as a single degree of freedom system. The base shear of the isolation plane is estimated using upper bound isolator properties at the Ultimate Limit State (ULS), whereas the displacement and rattle space are sized based on the lower bound isolator properties at the Collapse Avoidance Limit State (CALS). According to the isolation design guidelines, the superstructure can be classified as Type 1 Simple, which permits the use of the Equivalent Static Analysis (ESA) method. The superstructure is designed to respond elastically ($k_\mu = 1$) at the ULS. The base shear is then distributed to the superstructure with various distributions, and the superstructure design is governed by the highest demand. The details of these distributions can be found in the NZSEE/MBIE base isolation design guideline (NZSEE/MBIE, 2019). The drift limits for the superstructure are 0.5% at the Damage Control Limit State (DCLS) design intensity, and 2.5% at the ULS design intensity.

The case study buildings are designed for two sites: the Christchurch site has a subsoil class D and a $V_{S30}$ of 200m/s, and the Wellington site has a subsoil class C and a $V_{S30}$ of 450m/s. A previous study by John et al. (2022) investigated the performance of conventionally designed buildings at the same sites and with the same building layout. To directly compare the performance of the isolated buildings with the conventional buildings, the design seismic parameters are obtained from NZS 1170.5:2004 (Standards New Zealand, 2016), similarly to the conventional building design instead of site-specific hazard analysis. The resulting values are shown in Table 1.

*Table 1: Design seismic parameters for the base isolated case study buildings.*

| Sites | Z | R (IL2) | N (T, D) | α* |
|---|---|---|---|---|
| Christchurch (class D) | 0.3 | 0.25 (SLS1 1/25)<br>0.75 (DCLS 1/250)<br>1.0 (ULS 1/500)<br>1.5 (CALS 1/2500) | 1 | 1 |
| Wellington (class C) | 0.4 | | 1.24 ($T_{Dm}$) | 1 |

* α is the robustness factor to account for the structural resilience at CALS

## 2.3 Design results

This section reports the design results of the base-isolated case study buildings. Table 2 presents the design results of the isolation systems, including the structural performance factor ($S_p$), characteristic strength ($Q_d$), initial stiffness ($K_i$), post-yield stiffness ($K_d$), maximum displacement demand at the CALS ($D_M$) using lower bound isolator properties, base shear coefficient at the ULS ($S_a$) using upper bound isolator properties, and effective period of the isolation system at the corresponding limit state ($T_{eff}$). Note that the seismic weight used in the isolation system design is 25628kN. The superstructure design results are reported in Table 3.

## 2.4 Non-linear time history analysis

The 3D numerical models of the case study buildings are developed in OpenSees (McKenna, 2011) to perform Non-linear Time History Analysis (NLTHA). The models adopt a lumped plasticity approach with probable material properties. The RC walls are modelled as elastic cantilever sections with plastic hinges at the bottom and cracked section stiffness is set according to Priestley et al. (2007). The overall behaviour of the wall and the hinge follows a Takeda hysteretic model (Otani 1981). The gravity frames are modelled as truss elements and support gravity load only. The walls and frames are tied together with rigid diaphragms. More modelling details of the superstructure can be found in John et al. (2022). A Rayleigh damping model is applied with 3% damping at the first and fourth modes. The LRBs are modelled using a bilinear hysteretic

*Table 2: Design results of the isolation system.*

| Sites | $S_{p,iso}$ | $Q_d$ | $K_i$ | $K_d$ | $D_M$ | $T_{eff, Dm}$ | $S_{a, ULS}$ | $T_{eff, ULS}$ |
|---|---|---|---|---|---|---|---|---|
| Christchurch | 1 | 2072kN | 180kN/mm | 9kN/mm | 370mm | 2.99s | 0.164g | 1.31s |
| Wellington | 1 | 2356kN | 164kN/mm | 8kN/mm | 344mm | 2.87s | 0.169g | 1.31s |

*Table 3: Design results of the RC walls (per wall).*

| Sites | $S_p$ | $k_\mu$ | x-dir | $\varphi M_{n, x}$ | $\rho_{reinf, x.}$ | $T_{x, fixed}$* | y-dir | $\varphi M_{n, y}$ | $\rho_{reinf, y.}$ | $T_{y, fixed}$* |
|---|---|---|---|---|---|---|---|---|---|---|
| Christchurch | 1 | 1 | 3 × 0.4m | 8729kNm | 1.15% | 0.56s | 6 × 0.3m | 27066kNm | 1.31% | 0.41s |
| Wellington | 1 | 1 | 3 × 0.4m | 9553kNm | 1.31% | 0.56s | 6 × 0.3m | 30110kNm | 1.47% | 0.41s |

* $T_{fixed}$ of the superstructure is estimated using the Rayleigh method and the displacement profile is estimated using the effective section properties as per NZS 3101:2006 (Standards New Zealand, 2006).

model. The FSs are modelled using Coulomb friction model. Foundation flexibility is not included in the scope of this study, and large stiffness is assigned to the grillage beams which can be considered approximately rigid. The numerical model was loaded bi-directionally using selected ground motions. The analysis time step is set to be 0.005s and in the case of non-convergence, the time step is automatically reduced. Vertical excitation is not included as it was outside the scope of this study.

Yeow et al. (2018a) selected 180 pairs of hazard-consistent ground motions across nine intensity levels, with 20 pairs for each intensity, using the generalised conditioning intensity measure approach (Bradley, 2010). This selection is based on the 2010 National Seismic Hazard Model (NSHM) by Stirling et al. (2012). The ground motions are selected for Christchurch class D and Wellington class C, at SA(1.0s) and SA(2.0s). The SA(2.0s) ground motion sets are used to perform the NLTHA of the isolated buildings in this study, whereas the SA(1.0s) ground motions sets were used by John et al. (2022) to analyse the conventional buildings.

## 2.5 Loss assessment

The loss assessment is carried out following the PEER PBEE framework and using the Performance Assessment Calculation Tool (PACT) provided in FEMA P-58-3 (FEMA, 2018b). The framework has four steps: (i) Hazard analysis, to determine the hazard curve and select ground motions accordingly; (ii) structural analysis, to perform NLTHA and record the Engineering Design Parameters (EDPs), such as storey drift, floor acceleration, and isolator displacement; (iii) damage analysis, to identify damage to various building components based on EDPs. According to the building plans (Yeow et al. 2018a), the damageable components are listed in Table 4; and (iv) loss analysis, in which the losses such as repair cost of the damaged components are then identified.

The component fragility functions and consequence functions are provided by PACT and Yeow et al. (2018b). The building cost and repair cost are referenced to the timeframe of year 2020. In addition, two damage states are defined for the LRBs. Damage state one includes damage to individual isolators and requires individual repairing. Damage state two occurs when the isolators exceed their maximum displacement capacity and cause structural instability. The experimental tests on LRBs have shown that they can reach a shear strain of at least 400% before rubber hardening occurs (Eem and Hahm, 2019; Hamaguchi et al., 2019). Therefore, the median maximum isolator displacement capacity is set to be 500% shear strain times the total rubber thickness. Due to the limited number of specimens tested, the dispersion of 0.3 is assumed for the maximum isolator displacement capacity and future research should update this number.

*Table 4: Summary list of structural and non-structural components included in the loss assessment.*

| Structural | Non-structural (drift) | Non-structural (acceleration) | |
|---|---|---|---|
| Structural wall | Exterior glazing partitions | Suspended ceiling | Ducts |
| Lead rubber bearing | Precast cladding panels | Braced ceiling | Droppers and diffusers |
| | Interior glazing partitions | Air handling units | Coils |
| | Full height partitions | Traction elevator | VAV boxes |
| | Precast stairs | Water pipes | Independent pendant lighting |
| | | Sanitary waste piping | Fire sprinklers and pipes |
| | | Chiller capacity | Cooling tower capacity |

## 2.6 Comparison of the NSHMs

The NZS 1170.5:2004 uses the National Seismic Hazard Model (NSHM) developed in 2002 (Stirling et al., 2002) as the hazard basis, with some modification after the Canterbury earthquake sequence (Standard New Zealand, 2016). The ground motions used in this study are selected based on the 2010 NSHM (Yeow et al. 2018). The latest revision of the NSHM was released in 2022 (Gerstenberger et al., 2022). All hazard curves are plotted in Figure 2 for comparison. Comparing the design hazard curve from NZS 1170.5:2004 and the 2010 NSHM, the design hazard curves overestimate the rates for moderate to large intensity earthquakes for the Christchurch site. For the Wellington site, the design hazard curves overestimate the rates of small to moderate intensity earthquakes and underestimate the rates of moderate to large earthquakes. Comparing the 2010 NSHM with the 2022 NSHM, it appears that the 2022 NSHM has increased rates of moderate to large intensity earthquakes in Christchurch and increased rates of all intensity levels in Wellington.

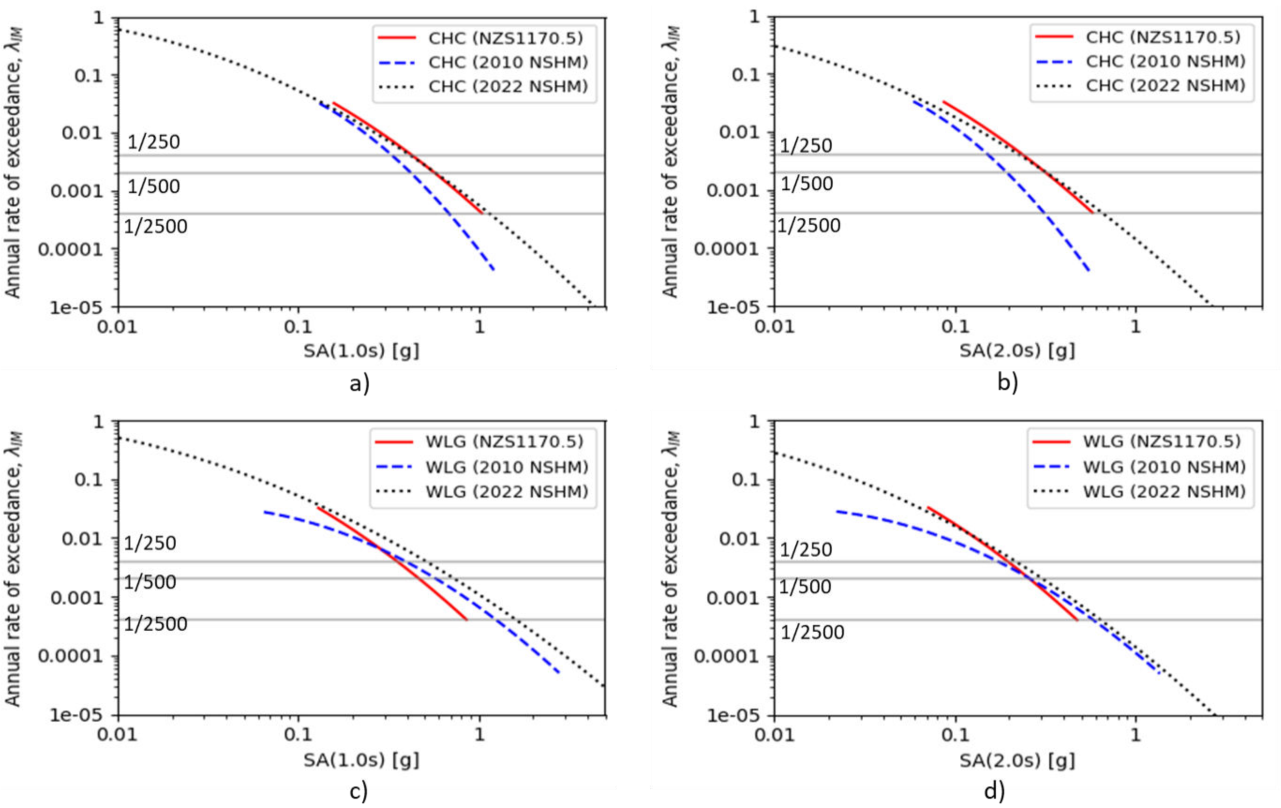


*Figure 2: Comparison of the seismic curves between NZS 1170.5:2004, 2010 NSHM, and 2022 NSHM: a) Christchruch SA(1.0s), b) Christchurch SA(2.0s), c) Wellington SA(1.0s), and d) Wellington SA(2.0s).*

## 3 ISOLATED BUILDING VS CONVENTIONAL

The structural response and direct repair costs of the conventional and isolated buildings are compared in this section. The conventional four-storey RC wall buildings have the same superstructure layout which were previously designed by John et al. (2022), their designs are replicated and analysed in this study for comparison. The conventional buildings are designed to meet the minimum building code requirement. A ductility of four is assumed and the design is governed by the 2.5% drift limit at the ULS. The conventional building in Christchurch has fundamental periods of 0.87s and 0.79s for the X and Y directions, respectively. The conventional building in Wellington has fundamental periods of 0.85s and 0.86s for the X and Y directions. The initial cost of the isolated building is approximately 5% more than the conventional design. The loss threshold is 0.5, meaning the building would require replacement once the repair cost reaches 50% of the replacement cost. The loss in this section is analysed based on the 2010 NSHM (Stirling et al., 2012) for which ground motions for NLTHA had already been selected by Yeow et al. (2018a).

### 3.1 Structural response

The dynamic response of the structure is recorded during the bi-directional excitation in OpenSees. The median peak acceleration of the roof level and the median peak storey drift among all storeys and both X and Y directions are plotted at each intensity, in Figure 3. The DCLS, ULS, and CALS design intensities are highlighted. For the same design requirement and targets, the buildings in Christchurch experience less floor acceleration and storey drift compared to the ones in Wellington. This could be explained by the difference in the design hazard curves and ground motion hazard curves (2010 NSHM), as mentioned in the previous section. As expected, the isolated building has effectively reduced the storey drift and floor acceleration. The peak storey drifts are less than 0.5% even at the ULS design intensity.

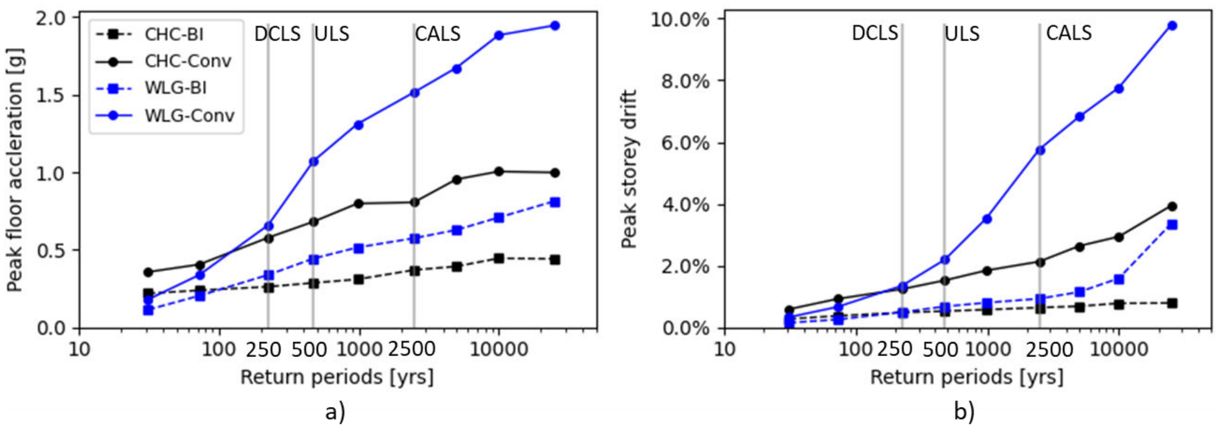


*Figure 3: Median peak structural response for the conventional and isolated building in Christchurch (X and Y directions): a) peak roof acceleration and b) peak storey drift of all storeys.*

### 3.2 Direct building repair cost

The direct building repair costs are estimated using PACT. The normalised Expected Annual Losses (EALs) are 0.13% and 0.06% for the conventional and isolated buildings in Christchurch, and 0.15% and 0.08% for the conventional and isolated buildings in Wellington, respectively. Figure 4a presents a breakdown of the EAL, including the loss contributions from building replacement, damages to structural, and damage to non-structural components, including acceleration-sensitive and drift-sensitive components. No building replacement is observed for the conventional buildings, whereas for isolated buildings the contribution from building replacement is observed. This is partially because it was conservatively assumed that if the isolators reach their displacement capacity, the subsequent damage to the building would prompt replacement. This behaviour is also reflected in the vulnerability functions in Figure 4b.

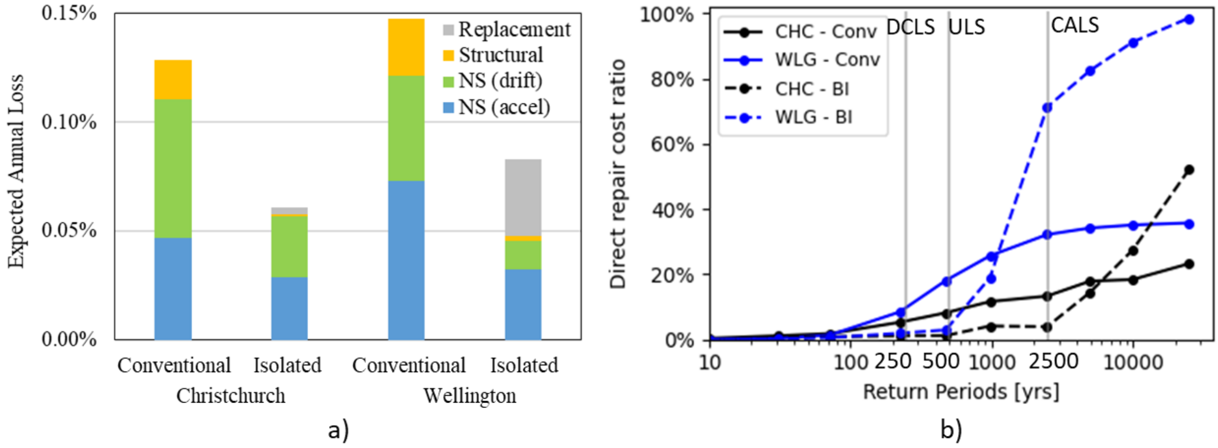


*Figure 4: Loss comparison of the conventional and isolated buildings in Christchurch and Wellington: a) breakdowns of expected annual losses and b) vulnerability curves of direct repair cost ratio.*

## 4 IMPACT OF THE 2022 NEW ZEALAND NATIONAL SESIMIC HAZARD MODEL

In the previous section, the loss is analysed based on the 2010 NSHM, which was the basis of the selected ground motions. This section examines how the loss estimates could change considering the 2022 NSHM. Comparisons are made to investigate the impact of the updated NSHM. Due to the lack of available ground motions for the updated hazard curves, the same ground motions are applied in the NLTHA. However, the rate of exceedance of each intensity is scaled to match with the updated hazard curves. The EALs of all four case study buildings NSHM are presented in Figure 5. It is clear that the increased seismic hazard in the 2022 NSHM results in various levels of increase in EALs for all four cases.

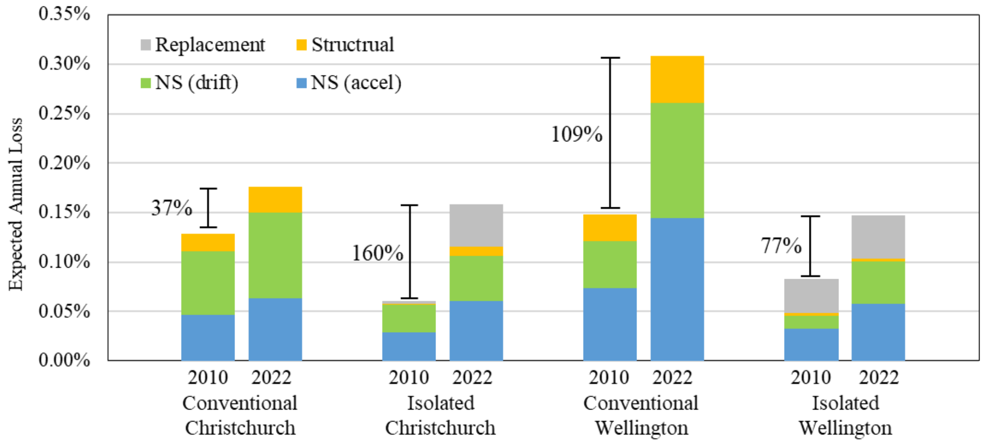


*Figure 5: Comparison of the EALs of conventional and isolated buildings in Christchurch and Wellington, based on the 2010 NSHM and 2022 NSHM.*

Figures 6a shows the contribution of each performance category to the total EAL for the conventional building in Christchurch. Most of the loss stems from damage to non-structural components at lower intensities (around 0 – 0.6g). Whereas in Figure 6b, the loss in the isolated building is also largely affected by the building replacement cost at higher intensity levels. As previously shown in Figures 2a and 2b, the 2022 NSHM has increased the effects of moderate-to-large earthquakes in the Christchurch site compared to the 2010 NSHM. This means that the losses associated with high intensity shaking are more likely to be impacted, hence the isolated building is likely to be impacted more than the conventional building. This is reflected in Figure 5, where the EAL is increased mildly by 37% for the conventional building and by 160% for the isolated building mostly due to changes in losses associated with replacement.

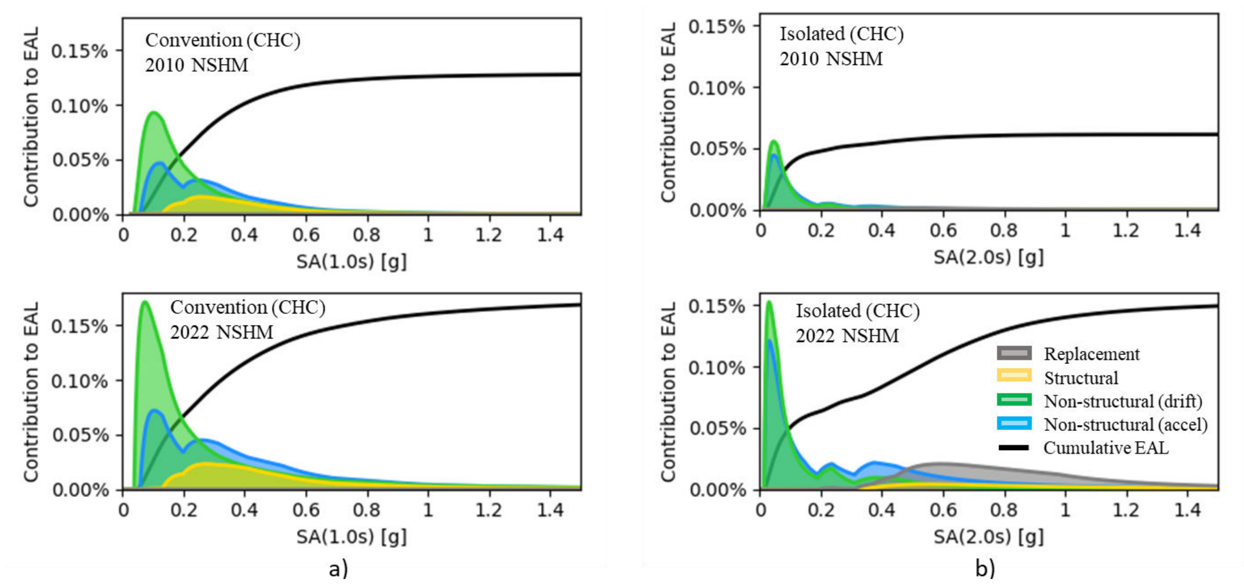


*Figure 6: Contributions to the expected annual loss for buildings in Christchurch based on the 2010 and 2022 NSHMs: a) conventional and b) isolated.*

The loss contributions of each performance category of the conventional and isolated buildings in Wellington are presented in Figure 7. Similar to Figure 6, the loss in the conventional building is mostly observed between 0 - 0.8g from the damage to structural and non-structural components. The loss in the isolated building is contributed to from both non-structural components at low intensities and building replacement cost at high intensities. From the discussion around Figures 2c and 2d, the 2022 NSHM increased the effects of small-to-moderate earthquakes compared to the 2010 NSHM in the Wellington site. This would have a larger impact on the losses at low intensities that are contributed from non-structural components. However, because the isolated building significantly reduces the floor acceleration and storey drift as shown in Figure 3, the updated NSHM will likely impact the conventional building more than the isolated one in Wellington. This again is reflected in Figure 5, where the EAL is increased by 109% for the conventional building and 77% for the isolated building.

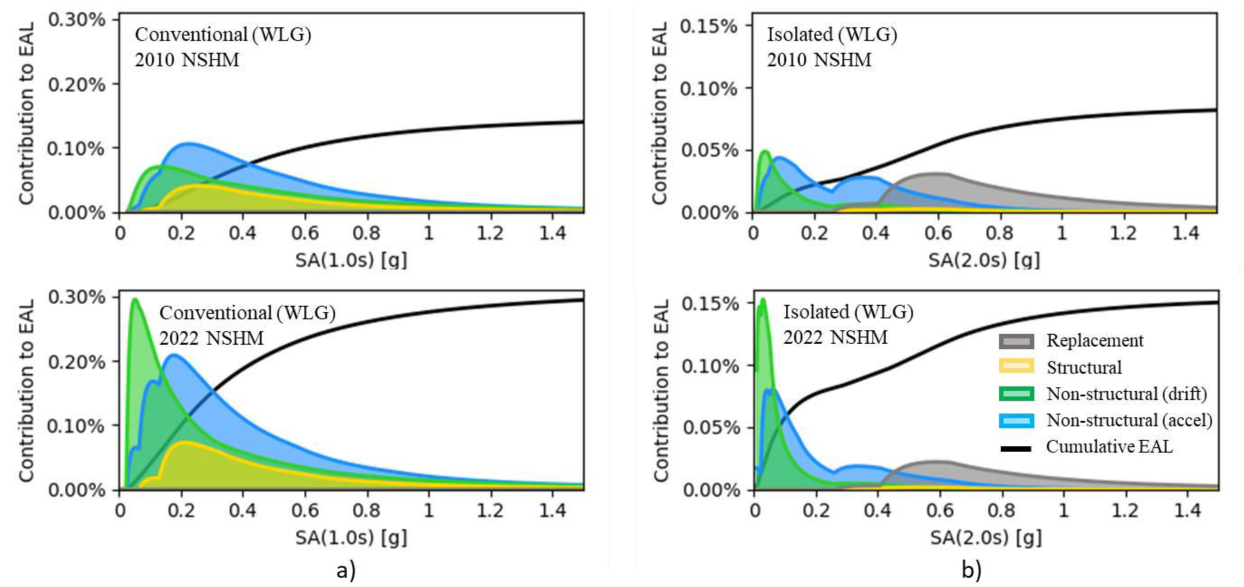


*Figure 7: Contributions to the expected annual loss for buildings in Wellington based on the 2010 and 2022 NSHMs: a) conventional and b) isolated.*

# 5 CONCLUSION

Four case study buildings located in Christchurch and Wellington, including two isolated and two conventional buildings, have been compared to investigate the performance of isolated buildings designed following the NZSEE/MBIE base-isolation design guidelines and the impact of the 2022 national seismic hard model. 180 pairs of ground motions across 9 intensity levels were used to perform the NLTHA. Loss assessment was the carried out following the PEER PBEE framework. Based on the work done, the following conclusions are made:

- An isolated building, designed following the NZSEE/MBIE isolated design guidelines, was found to perform better, with peak floor acceleration demands and peak storey drift demands significantly reduced compared to a conventional building. The reduced drift and acceleration demands also implied that the expected annual losses of base isolated buildings are likely to be around two times lower than conventional buildings.
- The implications of the using the updated NZ seismic hazard model on expected losses due to earthquake repairs were investigated. It was found that, with the same design criteria and targets, the impacts of the hazard model changes on estimated repair costs in Christchurch and Wellington are somewhat different. This highlights the importance of having more accurate seismic hazard models, and the need to update the design standards.
- The new hazard model suggests that the EALs for base-isolated buildings are 160% and 77% higher for the case study buildings in Christchurch and Wellington, respectively. In contrast, the EALs for traditional buildings are 37% and 109% higher in Christchurch and Wellington, respectively. The different impacts of the NSHM on EAL for buildings in Christchurch and Wellington is attributed to a larger increase in the annual probabilities of exceedance of moderate earthquakes in Christchurch compared to Wellington.


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