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INVESTIGATING THE IMPACT OF DESIGN CRITERIA ON THE EXPECTED SEISMIC LOSSES OF MULTI-STOREY OFFICE BUILDINGS

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ABSTRACT

The Ministry of Building, Innovation and Employment is developing advice on how to deliver Low Damage Seismic Design (LDSD) protection for buildings through their Tū Kahika: Building Resilience platform. The draft LDSD advice is considering a design drift limit for multi-storey buildings of 0.5% as part of new damage control limit state design checks. The potential impact of this design criterion on the expected annual loss due to repair costs is investigated for generic reinforced concrete wall case-study office buildings of 4- and 12-storeys in both Wellington and Christchurch. The equivalent static method, in line with NZS 1170.5 and NZS 3101, was used to design the buildings to conventional and draft LDSD specifications, representing current and future state-of-practice designs. The draft LDSD advice aims to limit the expected annual loss of complying buildings to below 0.1% of building replacement cost. This research tested this expectation. Losses were estimated in accordance with FEMA P-58, using building responses from non-linear time history analyses. Although it is found that the new drift limit alone may not limit seismic losses to the target values owing to damage to acceleration-sensitive elements, the results do support the intentions of the draft design advice to significantly reduce the expected seismic losses of complying buildings. The study also highlighted the importance of using an accurate approximation of RC wall stiffness for LDSD, and provides insight into different design strategies that could be followed to effectively limit losses in RC wall buildings as part of LDSD.

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INTRODUCTION

Assessments from the Canterbury earthquake sequence and Kaikōura 2016 earthquake found that modern capacity-designed buildings generally met structural performance expectations, but repair costs, particularly associated with damage to non-structural components, were significant [1, 2]. The Ministry of Building, Innovation and Employment as part of their Tū Kahika: Building Resilience platform is developing advice for clients and design teams on how to deliver Low Damage Seismic Design (LDSD) protection for buildings. This work is being managed by Engineering New Zealand in collaboration with sector stakeholders including engineers (SESOC, NZSEE, NZGS), architects (NZIA), insurers and property professionals. Work is currently ongoing to identify new design criteria that better limit seismically-induced monetary losses for buildings. As part of this, the draft Low Damage Seismic Design (LDSD) advice aims to limit loss by tightening design criteria (primarily drift limits). This research investigates whether the early draft LDSD proposals are likely to be effective in improving seismic performance, by reducing the expected losses (from repair costs) for reinforced concrete (RC) wall buildings.

The specific goals and objectives of the research are to:

1. Verify the draft performance objectives for low-damage buildings, which are to:
    a. Limit expected annual losses to 0.1% of the building replacement cost or less (with a 50% confidence level); and
    b. Limit the chances that an earthquake incurs a repair cost greater than 5% of the building replacement cost over a 10 year period to 1 in 25;
2. Investigate how effective the draft LDSD criteria are at reducing losses, compared to current NZ design standards, for RC wall buildings of differing heights and locations/seismic hazards; and
3. Investigate how the draft LDSD advice could affect the design of RC wall buildings, and identify potential issues with the application of the advice in practice.

The research considers four different commercial office case-study buildings with RC wall lateral systems, these being 4- and 12-storey buildings located in both Christchurch and Wellington. This sample of buildings was selected to consider the effects of varying building heights and seismic hazards. The 4- and 12-storey buildings have previously been investigated by Yeow *et al.* [3] to assess the performance of alternative lateral systems. The case study building footprint is 24 m by 40 m (4-storey) or 32 m by 48 m (12-storey), with 8 m bays in both directions (see Figure 1). Storey heights are 4.5 m for the first storey and 3.6 m for those above. Hence, building heights are 15.3 m and 44.1 m for the 4-storey and 12-storey buildings respectively. The building uses primary beams along gridlines in the short elevation, with RC double tees specified as the floor system. Non-structural component quantities and weights are taken from Yeow *et al.* A 3 kPa live load and load combination factors were selected in accordance with AS/NZS 1170.1 [4] when computing the weight of floors for design.

Various symmetrical arrangements of RC walls were designed and investigated. Designs are completed in accordance with NZ standards [5, 6] and relevant parts of the draft LDSD advice (described in the next section). Non-linear time history analyses (NLTHA) are conducted in OpenSees [7] to predict building responses to ground motions at a range of intensity levels.

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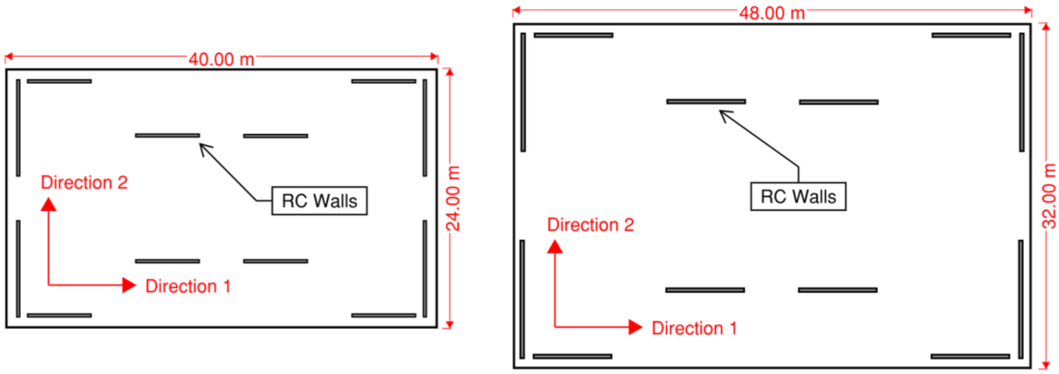

Figure 1: Plan view of case study RC wall buildings (left: 4-storey, right: 12-storey) (number of walls varies according to the design).

These responses are then used to inform loss assessments for the various building designs. Loss assessments are completed in accordance with FEMA P-58, utilising data from the Performance Assessment Calculation Tool (PACT) and QuakeCoRE. Where necessary, mechanics-based approaches are implemented to provide more appropriate loss assessment data.

SEISMIC DESIGN AND TIME HISTORY ANALYSES

Seismic Design of the Building

To represent current state-of-practice design procedures, the equivalent static method is used to produce indicative wall designs for current conventional and future LDSD specifications. Design is completed in accordance with the NZS 1170.5 equivalent static method and NZS 3101 reinforced concrete seismic design provisions [5, 6]. The hazard factor, Z, and the site subsoil class was 0.3 and D for Christchurch and 0.4 and C for Wellington buildings. The near fault factor has been determined in accordance with NZS 1170.5, with the assumption that all Wellington buildings are within 2 km from the nearest major fault.

Periods used for design were calculated by the Rayleigh method and may be found in the appendix. P-delta effects from shear and flexural deformations and accidental eccentricity were considered in the design in accordance with NZS 1170.5. All designs had symmetrical wall arrangements. Hence, torsional effects on buildings beyond the inclusion of accidental mass eccentricity were not considered in the designs. The criteria used for the two design scenarios consisted of strength, material strain and drift limit checks. The conventional design included a serviceability limit state (checking walls for yielding) and an ultimate limit state (wall strength check, material strain limits and building drift limit). LDSD also included a damage control limit state (DCLS) where storey drift limits were checked. For the conventional design, a ductility of 1.0 and 4.0 was assumed at the serviceability limit state (SLS1) and the ultimate limit state (ULS) respectively. For LDSD, a ductility of 1.0, 2.0 and 3.0 was assumed at SLS1, DCLS and ULS respectively. Even though one might query whether the use of the same ULS design ductility factor for both LDSD and conventional buildings could have resulted in similar strength and acceleration demands, other design criteria for LDSD buildings would be expected to govern the wall design. As such, LDSD RC wall buildings will tend to be stronger and stiffer, and hence will typically experience higher floor acceleration demands than conventional RC wall buildings. Wind loads were assessed with conservative parameters using AS/NZS 1170.2:2011 [8], however, lateral wind loading never governed over seismic loads.

Table 1 outlines the drift limits to which the two design scenarios adhered. While SLS1 was considered in the designs, it was found to never govern the design of the RC walls. SLS1 was considered at a return period of 25 years for the conventional designs, and at 50 years for LDSD (as suggested by NZS 1170 and the draft LDSD advice). It should be noted that the trial design criteria tested here are currently being drafted in the LDSD advice only for buildings up to six storeys. Although the 12-storey building is not covered by the LDSD recommendations, it was investigated regardless to understand the implications of extrapolating the guidance to taller buildings. Walls were designed with concrete having a characteristic compressive strength of 40 MPa and reinforcement having a characteristic yield strength of 500 MPa. All walls were designed with doubly reinforced and equally spaced rebar along the wall length.

Table 1: Design drift limits for conventional and low-damage designs.

Design case	Drift limit	Limit state	Design intensity return period
Conventional	2.5%	ULS	500 years
Low-damage	0.5%	DCLS	250 years
	2.5%	ULS	500 years

Results: Seismic Design

For LDSD, drift limits at the DCLS were found to dictate the wall design, and so a designer would aim to increase wall stiffness to reduce building drift. The approach to determine wall stiffness in NZS 3101 (Table C6.5 of the NZS 3101 commentary) only accounts for the geometric properties of the wall section and axial load, meaning a designer can only increase the wall thickness or length in order to increase wall stiffness. To increase stiffness, the stiffness provisions of NZS 3101 encourage the design of longer walls (as the most efficient way to increase second moment of area). This would likely be the method by which designers achieve the lower drift limits for LDSD when designing with RC walls. However, there are limitations with the NZS 3101 approach to determine the effective second moment of area (I_e). The cracked stiffness method in NZS 3101 ignores the effect of longitudinal reinforcement on wall stiffness. Priestley and Kowalsky [9] and Paulay [10] have shown that the nominal yield curvature of RC walls is relatively independent of their strength and hence, the cracked stiffness of a wall should be set as a function of the strength (i.e. as a function of the axial load ratio and quantity of longitudinal reinforcement). Priestley's method of determining effective stiffness in equation (1) demonstrates the relationship between section strength and stiffness:

$$E_c I_{cr} = \frac{M_N}{\phi_y} \tag{1}$$

where M_N = section moment capacity (based on expected material strengths); and ϕ_y = approximate yield curvature of rectangular RC wall.

With more reinforcement, the wall will be stronger and stiffer. For low-damage design, with drift limits being critical, the wall stiffness used in design must be accurate and so the reinforcement should be properly considered in the section. Therefore, it was proposed that the method of Priestley *et al.* [11] be used (in which the cracked stiffness depends on the flexural strength) instead of the NZS 3101 approach for determining wall stiffness.

Consequently, four designs were produced for each case-study building (two scenarios of conventional and low-damage, with each scenario designed with the two wall stiffness methods). Design details for all walls may be found in the appendix. The two designs for each scenario only differed in the method used to determine the wall stiffness; one used the NZS 3101 approach, and the other used the recommendation by Priestley (equation (1)) (with this approach considering the secant stiffness to yield for design). This was done to test how much the low-damage design could reduce losses compared to a conventional design, along with investigating the effect of using a different method for approximating wall stiffness. It was recognised that a cracked stiffness specified by Priestley's method may not always be appropriate for approximating stiffness, particularly at upper portions of the wall, at DCLS (with lower drifts and potentially more limited cracking). However, Priestley's method was still considered as a useful design tool for practitioners seeking a relatively simple and more accurate approximation to wall stiffness than the NZS 3101 method. It should allow them to rationally utilise reinforcement to stiffen walls, rather than being limited by modifying wall section geometry as encouraged by NZS 3101. One could expect that for very low intensity levels, the walls would be uncracked (even though modelled as cracked) and thus lower drifts and higher accelerations might be anticipated, reflecting an uncertainty in this work. However, a quantification of the uncertainty wall cracking has on the building response and overall losses of the building was considered out of scope in this study.

A variety of wall arrangements in the two lateral directions of loading were designed for all 4-storey buildings but not the 12-storey buildings. It was not believed that varying the wall arrangements for the 12-storey buildings would add significant value to the research beyond that of which the 4-storey buildings showed.

Drift limits (from Table 1) generally were the most onerous design criteria, but in some cases wall reinforcement was governed by the wall strength required for the assumed level of ductility (see the appendix for details). Only buildings designed conventionally (to NZS 3101, 2.5% drift limit) were partially governed by strength/ductility requirements. All LDSD and conventional building designs were governed by the DCLS and ULS respectively. Low-damage designed buildings included a larger number and/or longer walls due to the stricter drift limit. The designs produced with NZS 3101 appeared to give more walls of lower reinforcement ratio to satisfy the drift limits, compared to designs produced using Priestley's stiffness which allowed the designer to choose fewer walls with higher quantities of longitudinal reinforcement. With the NZS 3101 stiffness approach, the reinforcement quantity was only dictated by the strength/ductility requirement. With more walls, the strength demand per wall was lower, resulting in less reinforcement being required. Using Priestley's stiffness, the reinforcement quantity was a factor in achieving a stiffer wall. This meant the designer could use fewer walls (architecturally and economically preferable) with higher reinforcement quantities to achieve the required stiffness to satisfy drift limits.

The wall designs also demonstrate the effect of the stiffness assumption on the design period (found by the Rayleigh method) of the building. The design periods for the building designs assuming Priestley's stiffness are within 6% of the period found by advanced modelling (as they use the same stiffness assumption). The design periods obtained assuming the NZS 3101 stiffness were always lower than the model periods. As such, it was found that the NZS 3101 stiffness approach appears to generally overestimate the wall stiffness compared to the models. Whilst this overestimation might appear conservative for strength requirements it is non-conservative when checking drift limits.

Through a study of the effect of reinforcement ratio on Priestley's stiffness, it was found that the NZS 3101 approach would overestimate the wall stiffness (compared to the models) up to a reinforcement ratio of approximately 1.6%. The stiffnesses of 3 m and 6 m walls were determined with Priestley's method, varying the reinforcement ratio by changing wall thickness or rebar spacing. In all cases, the convergence of stiffness between Priestley's method and the NZS 3101 method occurred close to a reinforcement ratio of 1.6%, as shown by the example in Figure 2. As the buildings designed with the NZS 3101 stiffness method had reinforcement ratios less than 1.6%, the corresponding building models would be less stiff than anticipated in NLTHA. Therefore, buildings with stiffness based on NZS 3101 would not be as stiff as the Priestley stiffness method buildings, particularly given that buildings designed with the Priestley method all had high reinforcement ratios.

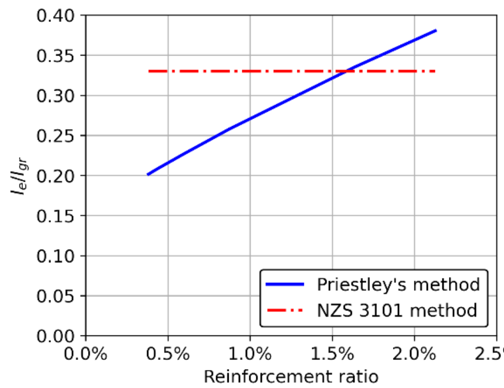

Figure 2: Comparison of effective second moment of area as determined by the NZS 3101 and the Priestley et al. (2007) methods for approximating RC wall stiffness. (Reinforcement ratio was varied by changing rebar spacing. 3 m wall length example shown, with 40 MPa concrete, 500 MPa steel, axial load ratio 0.1.)

Taller buildings gave lateral designs that required longer and a greater number of walls (primarily due to increased total seismic weights above the base of the buildings). The authors note that the size and number of walls required for some of the 12-storey designs would imply that the designs are not likely to be cost-effective or practical, but they are included in this study for completeness. In achieving proposed LDSD drift criteria for tall and slender buildings, very long or thick walls may be required, compromising available space for components such as glazing and passageways within the building. The 12-storey and 4-storey building designs in Christchurch often yielded a larger number and/or thicker walls compared to Wellington. This is because seismic demands for longer period buildings, which 4- and 12-storey buildings often represent, are larger in

Christchurch on soil type D compared to the Wellington region on soil type C (see Figure 3).

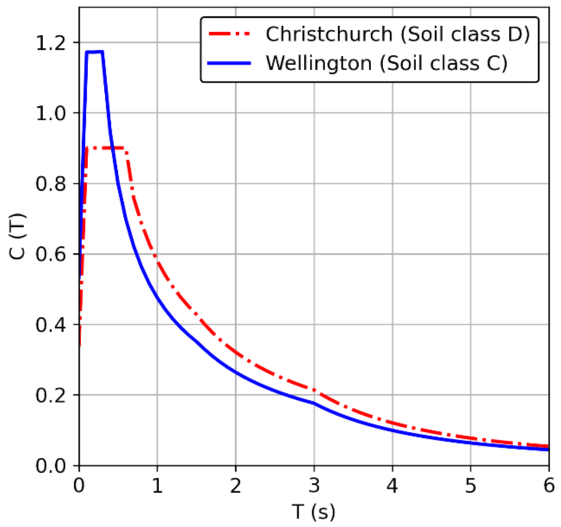

Figure 3: Design spectra for the two locations from NZS 1170.5:2004.

Non-linear Time History Analysis

Non-linear time history (NLTH) analysis is used to establish the likely seismic response of the buildings as if they were constructed in line with the results of the different design solutions. The modelling and analysis approach aims to quantify the peak storey drifts and floor acceleration demands for a range of intensity levels (return periods). This information will be used for loss assessment, as explained in the later corresponding section.

Structural Modelling

The walls were modelled with three dimensional lumped-plasticity frame elements in OpenSees for the time history analyses. Expected material strengths were used as per the recommendations of Priestley *et al.* ($1.3f'_c$ for concrete and $1.1f_y$ for steel). The walls were modelled as cantilevers, with elastic shear deformation included in the elements. The hysteretic plastic hinges at the bases of the walls were defined such that the force-displacement behaviour followed a Takeda (thin) model (Figure 4), as described by Otani [12]. Post-yield stiffness of the walls was modelled to be 5% of the elastic stiffness.

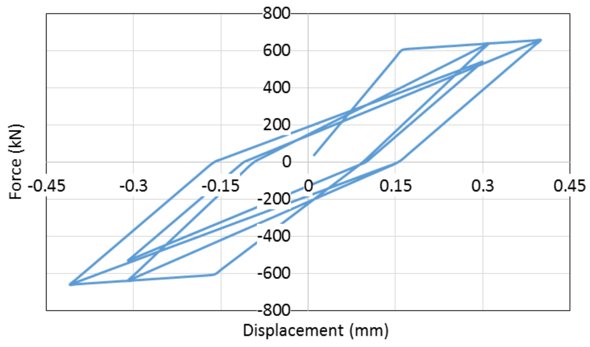

Figure 4: Example of Takeda (thin) behaviour (in a cyclic static pushover) used to model the RC walls.

The bilinear stiffness employed to approximate the nonlinear behaviour of the RC walls in loading used a cracked section stiffness (E_cI_{cr}) as the initial stiffness (see equation (1)), with the yield curvature of the rectangular walls approximated as (from Priestley *et al.* [11]):

$$\phi_y = \frac{2\varepsilon_y}{L_w} \tag{2}$$

where ε_y = yield strain of the steel; and L_w = wall length. The plastic stiffness after reaching the nominal moment was taken as 5% of the initial stiffness.

The wall shear area, A_v, was found with equation (3) to account for the reduction of effective area due to cracking:

$$A_v = \frac{I_{cr}}{I_{gross}} \frac{5}{6} A_{gross} \tag{3}$$

where I_{gross} = second moment of area of the gross section; and A_{gross} = gross area of the section.

Floors were modelled to act as rigid diaphragms in plane, fully flexible out of plane. The seismic mass of each floor was distributed to four mass nodes (placed symmetrically in the diaphragm) which were constrained (in the two horizontal directions) to the wall element nodes at each floor. P-delta effects were incorporated using dummy columns pinned between the mass nodes. The dummy columns were co-rotational truss elements carrying gravity loads from the mass nodes. When the mass nodes displaced, the axial load in the inclined truss elements induced a horizontal force on the floor diaphragms, increasing the lateral load demand on the walls.

Foundation deformations were accounted for in the model through the inclusion of elastic rotational springs at the base of each wall. The stiffness of each spring was defined such that it would rotate by 0.1% when the wall hinge reached its yield moment. This rotation was chosen as a feasible foundation stiffness based on experience from industry. This approach acknowledges the need to include foundation rotation as a factor in wall drifts but does not deal with uncertainty in foundation-soil interaction or site conditions, which was outside the scope of this research.

A Rayleigh damping model was used with 3% damping. Constants of proportionality were determined from the periods of the first and fourth modes (corresponding to the first and second modes of vibration in the longitudinal direction). The sensitivity of the building response to changes in the selected periods, damping ratios, and mass matrix constant of proportionality was tested. Changing the damping ratio of the building from 3% to 5% yielded an average 7% change in acceleration and drift. Excluding the mass matrix constant of proportionality yielded an average 5% and 9% change in acceleration and drift respectively. While these changes highlighted uncertainty in the analysis results, the sensitivity to the investigated inputs was deemed acceptable, given the focus of this study is on the impact of design criteria on the expected annual losses.

Seismic Hazard and Ground Motions

The time history analyses used 180 pairs of ground motion records to determine building responses for loss assessment, selected in a previous QuakeCoRE study by Yeow *et al.* to be hazard consistent with the sites considered. The ground motions (GMs) were divided into nine intensity levels, with two additional intensity levels (1 and 2) introduced to quantify losses at lower intensity levels (Table 2). Each ground motion was scaled to a specific spectral acceleration at the conditioning period of the ground motion (see example in Figure 5), with these spectral accelerations determined in the seismic hazard analysis by Yeow *et al*. The spectral acceleration for a ground motion depended on the location, intensity level and conditioning period (as part of the seismic hazard analysis) (see Figure 6). The Wellington seismic hazard was intended to represent a site subsoil class C and the Christchurch seismic hazard was intended to represent a site subsoil class D. The ground motion records were chosen to ensure a mix of active shallow and subduction zone type records to represent the fault rupture types which these regions might be subjected to, however, records with pulse-like properties (to capture near-fault effects) were not chosen. The seismic hazard for New Zealand is currently being re-evaluated. The hazard estimates (and consequently the designs) are likely to change in the future which will also change the loss assessment results for the case study buildings. The impact of such changes in hazard should be evaluated as part of future research.

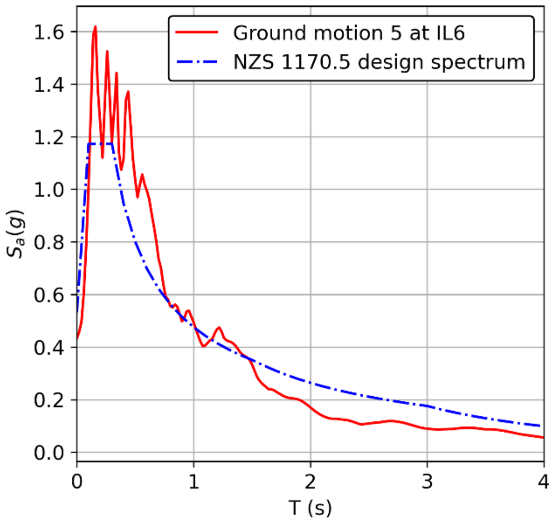

Figure 5: An example ground motion from intensity level 6, conditioned to a period of 1.0 s, compared to the NZS 1170.5:2004 design spectrum for Wellington (site subsoil class C).

Intensity levels 1 and 2 were introduced to capture losses at lower intensity events (higher probabilities of occurrence). These intensity levels were approximated with a simplified approach: NLTHA results from intensity level 3 were scaled down to give the building response for intensity levels 1 and 2. All building models responded elastically at the lowest intensity ground motions (intensity level 3). A hyperbolic fit was applied to the seismic hazard data, and the building responses were scaled down by considering the mean annual frequency of exceedance of the new intensity levels. The combined contribution of the intensity level 1 and 2 losses to the overall losses was found to be low (up to 10% and 18% for Wellington and Christchurch buildings respectively). While the authors recognise this approximation does not fully capture how a multiple degree of freedom structure may respond at more frequent events in these locations, it should not significantly affect the results and conclusions of the study. The inclusion of intensity levels 1 and 2 effectively gave 220 building responses to ground motions over eleven intensity levels for use in the loss assessment. Intensity level 5 was the design intensity for DCLS, and intensity level 6 was the design intensity for ULS.

The largest integration time step used in the time history analyses was 0.02 s. In the cases where the ground motion record was digitised in shorter time steps, this shorter time step was adopted for the analyses. Sensitivity analyses were carried out for representative cases where the largest time step (0.02 s) was used. Peak drifts varied by 6% or less, and peak floor accelerations varied by 15% or less, which was deemed acceptable for the purposes of this loss-assessment study. Ground motion records for each building were chosen with respect to the location of the building and the period of the building. The suite of ground motions with conditioning period closest to that of the building was selected for a given building. This was done to reduce variability of the building response across an intensity level. Conditioning periods selected for each building can be found in the appendix.

Table 2: Ground motion intensity levels with corresponding probabilistic descriptions.

Intensity Level	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
Probability of exceedance in 50 yrs (%)	99	90	80	50	20	10	5	2	1	0.5	0.2
Mean annual frequency of exceedance (MAFE)	0.0921	0.0461	0.0322	0.0139	0.0045	0.0021	0.0010	0.0004	0.0002	0.0001	0.00004
Return period (years)	11	22	31	72	224	475	975	2475	4975	9975	24975

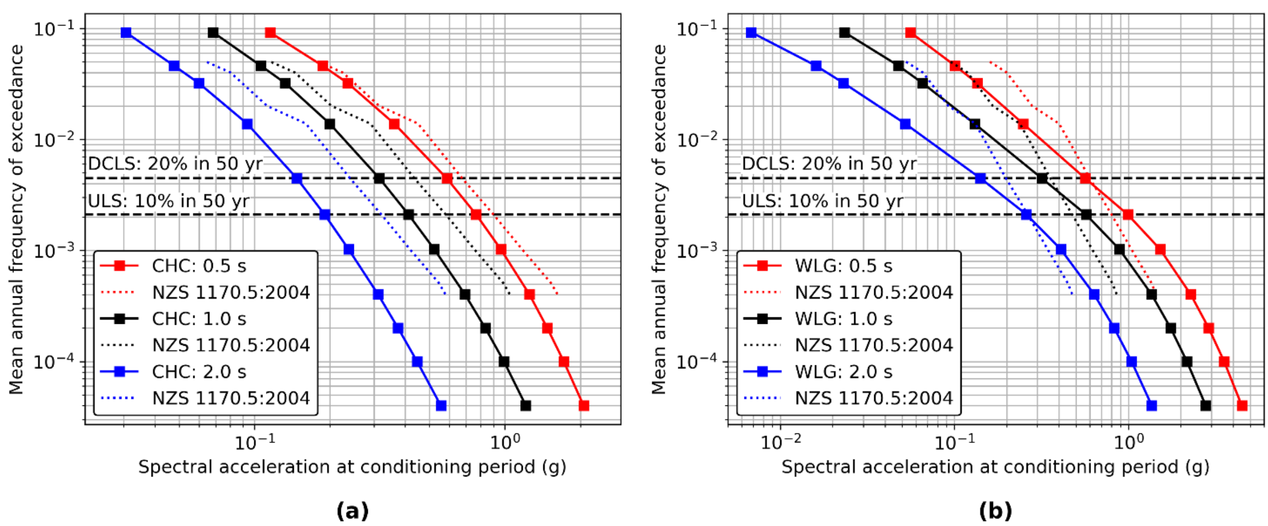

Figure 6: Hazard curves for the ground motion suites used for NLTHA of buildings in Christchurch (a) and Wellington (b), compared to design spectra from NZS 1170.5:2004.

Results: Non-linear Time History Analysis

The results from the time history analyses of the two design scenarios (conventional and LDSD) are summarised by the peak roof accelerations and drifts at the top storey of the various buildings (Figures 7-10). The building responses at these heights were generally the largest for the building, especially for buildings in which the first mode shapes dominated (most 4- storey buildings), and hence give a good representation of the building performance in the time history analyses. The plots show the medians of the peak building responses for the 20 ground motions at each intensity. Results plotted in Figures 7-10 show the peak building response for a single direction. The bidirectional ground motions generally gave similar median building response in the two perpendicular directions when the same wall arrangements were used in both directions. For further reference, Figure 11 shows drift and acceleration profiles up the building height for the Wellington 4-storey buildings at two intensity levels.

The top storey drifts shown in Figures 7-10 demonstrate that the stiffer buildings recorded lower drifts. As expected, the stiffer LDSD models experienced lower drifts than their conventionally-designed counterparts. Buildings designed with the Priestley stiffness method also appeared to be stiffer (recorded lower drifts) than those designed with the NZS 3101 method. Drifts for Wellington buildings appear slightly higher than those recorded for their Christchurch equivalents. For example, for the conventional buildings at the 1-in-475 year intensity (intensity level 5), the peak storey drifts for the 4- and 12-storey buildings were 2.1% and 1.9% respectively for Wellington whereas for Christchurch the peak drifts were 1.65% and 1.35% respectively. This is likely due to the seismic hazard predicted by NZS 1170.5 design spectra being less conservative for the Wellington seismic hazard than the Christchurch hazard predicted by Yeow *et al*.

Drifts appear to reduce, particularly for conventional models, as the building height increases. This could be attributed to design conservatism associated with the method for consideration of P-delta effects [13], which would be more pronounced for taller buildings. Other conservatisms may have arisen from the use of the equivalent static method. The difference between the design spectra and ground motion spectral accelerations could account for some discrepancy. Conventional models also appear to drift less than the ULS drift limit by an amount which is considerably more than the amount by which LDSD models drift less than the DCLS drift limit. Again, this could be due to conservatism introduced with the P-delta design method, which becomes more significant when the building drifts more. Generally, LDSD buildings drifted close to, or less than, the 0.5% drift limit at the nominal DCLS intensity level.

The roof accelerations shown in Figures 7-10 suggest that the stiffer buildings experienced higher accelerations. The general trend was that the stiffer LDSD models experienced higher accelerations than the conventional models, and the models representing designs obtained via Priestley's stiffness approach also experienced higher accelerations than their 'NZS 3101 compliant' counterparts. As previously discussed, accelerations for Wellington buildings were higher than their Christchurch counterparts and taller buildings experienced higher accelerations at the roof. Consider the results for the LDSD designs using the Priestley stiffness method shown in Figures 7-10. For Christchurch buildings at intensity level 5, which corresponds to a 1-in-475 year event, the median peak accelerations were 1.35g and 1.5g for the 4- and 12-storey buildings respectively. For Wellington buildings at the same return period event, the median peak accelerations were considerably higher, being 2.15g and 2.5g for the 4- and 12-storey buildings respectively.

Some may consider the intensity-based comparison of engineering demand parameters for different buildings in Figures 7-10 to be inappropriate, given that ground motions were selected with reference to a conditioning period and all the buildings have different periods. The effect of variation in building response due to difference in the building periods and ground motion conditioning periods was minimised where practicable. This was done by adopting ground motion records with conditioning periods of 0.5 s, 1.0 s and 2.0 s and using the records whose conditioning period best matched the building period for the time history analysis of a given building. However, it was not always possible to closely match the building and conditioning periods, particularly for the 12-storey buildings. This did result in some variation in the building drifts, such as that seen in Figure 7(a) at the high intensity levels. For the example of the Christchurch 4-storey LDSD building, whose median drift reduces at intensity level 11, the dispersion of drifts is 0.4 to 0.5 at the three highest intensity levels, and 0.25 to 0.33 for the lower intensity levels. Indeed, one could expect the dispersion in demands at different intensity levels to vary quite significantly between buildings, and this dispersion is accounted for as part of the rigorous loss assessment process described in the following section. Nevertheless, Figures 7-10 do provide valuable insight into the typical (median) magnitude of peak drift and acceleration demands associated with different intensity levels which is likely to be of interest to practitioners.

Peak residual drifts were also recorded along with the primary engineering demand parameters (peak drifts and accelerations). Residual drifts were obtained from NLTHA by allowing the building to return to a resting state after experiencing a ground motion and then recording the largest residual inter-storey drift. Residual drifts were larger for buildings which experienced higher peak drifts during ground motion excitation.

The structural models for NLTH analyses only included the RC walls in their lateral stiffness: no stiffness of secondary elements, such as gravity structure, was modelled. The inclusion of secondary stiffness and the resulting increase in post-yield stiffness of the models would likely reduce residual drifts significantly [14]. A sensitivity check was completed for conventional buildings in Wellington (4- and 12-storeys, designed with Priestley's stiffness) by increasing the post-yield stiffness from 5% to 10%. Increasing post-yield stiffness reduced residual drifts significantly (for example, median residual drift at the 2500-year return period intensity level reduced by 1.60 and 1.14 times for the 4- and 12-storey buildings respectively). Strength degradation effects were not included in the modelling and if included could have resulted in larger residual drifts. The uncertainty in the recorded residual drifts is acknowledged in the loss assessment but does not impact the conclusions of the study.

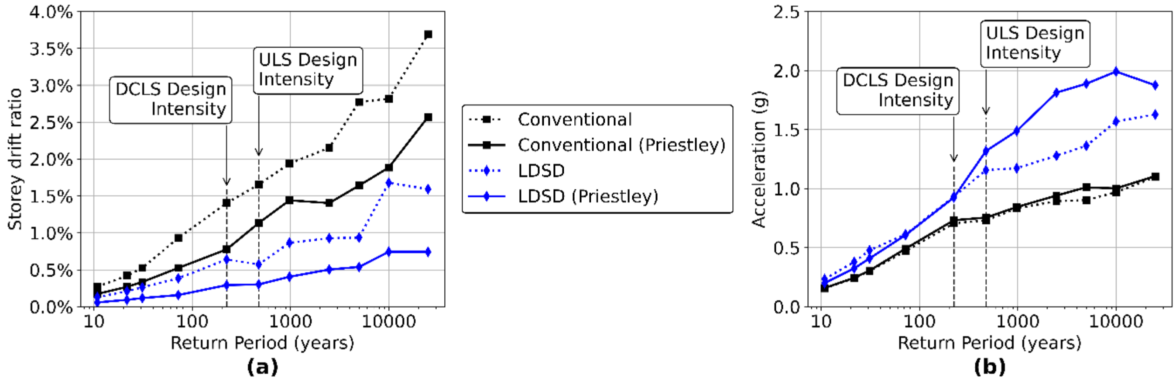

Figure 7: Median values of (a) peak top storey drifts and (b) peak roof accelerations across the eleven intensity levels, recorded from time history analyses of the 4-storey buildings in Christchurch.

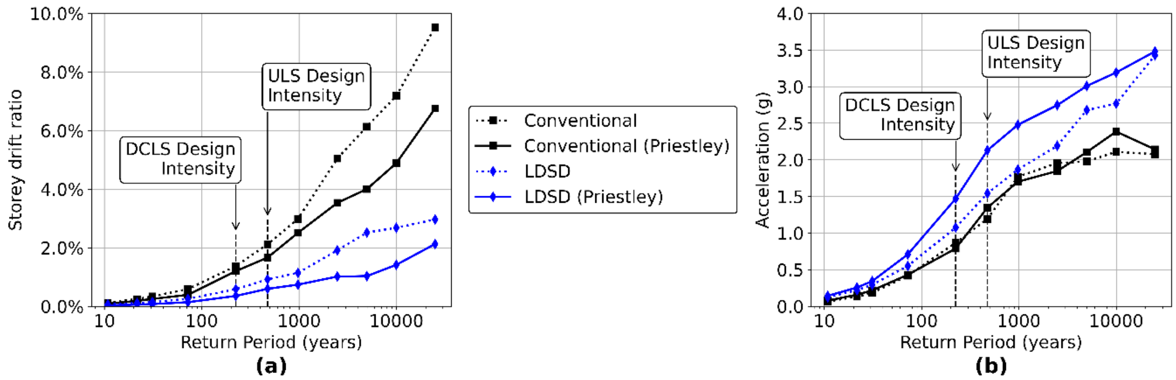

Figure 8: Median values of (a) peak top storey drifts and (b) peak roof accelerations across the eleven intensity levels, recorded from time history analyses of the 4-storey buildings in Wellington.

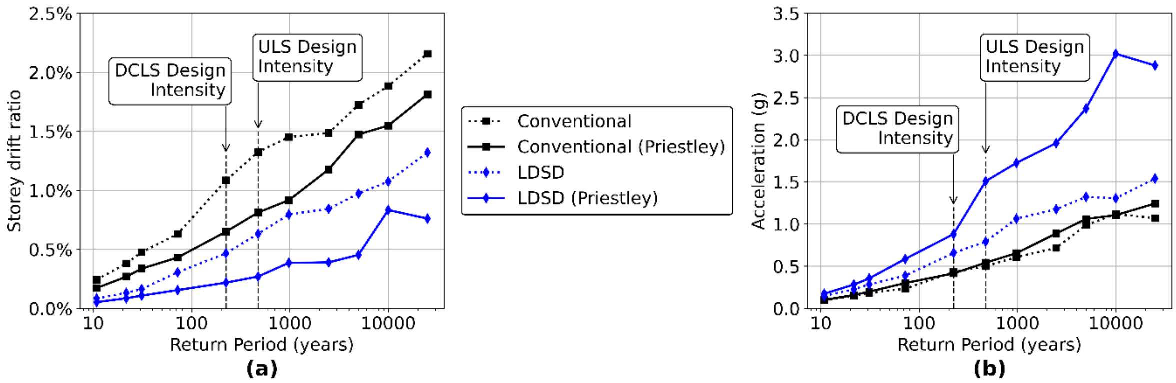

Figure 9: Median values of (a) peak top storey drifts and (b) peak roof accelerations across the eleven intensity levels, recorded from time history analyses of the 12-storey buildings in Christchurch.

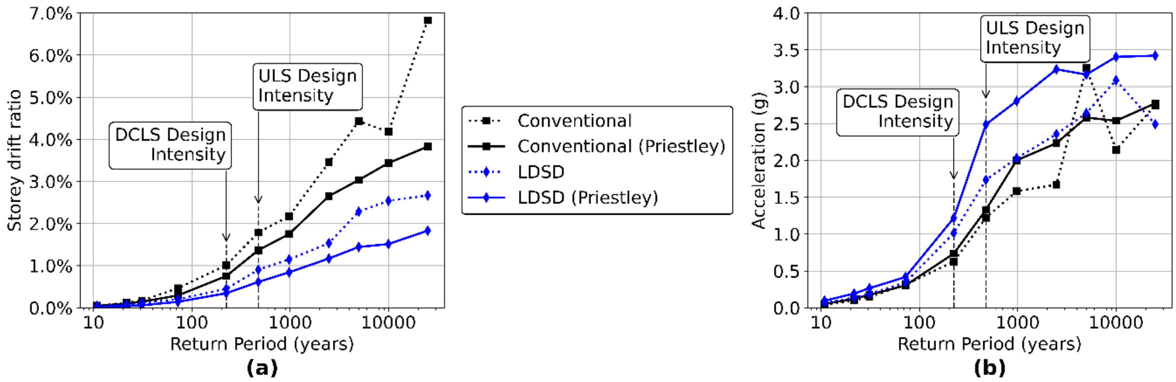

Figure 10: Median values of (a) peak top storey drifts and (b) peak roof accelerations across the eleven intensity levels, recorded from time history analyses of the 12-storey buildings in Wellington.

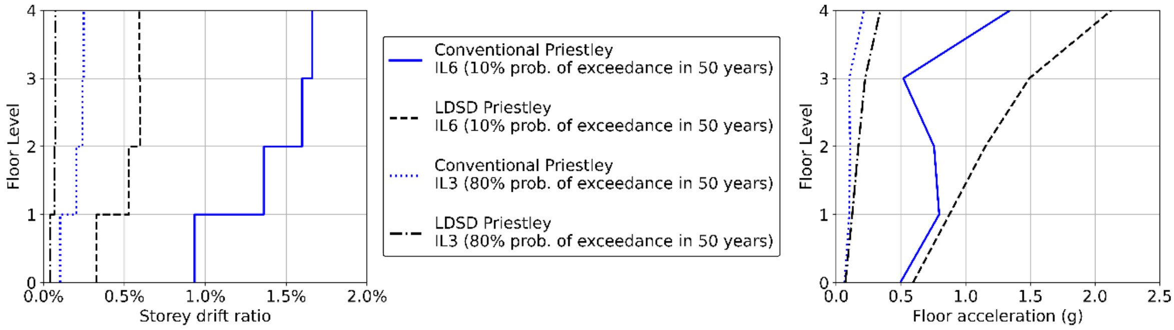

Figure 11: Median values of peak storey drifts and peak floor accelerations up the building height, recorded from time history analyses of the for the 4-storey buildings in Wellington.

LOSS ASSESSMENT

Loss assessments were completed using the Performance Assessment Calculation Tool (PACT) software [15] in accordance with the performance-based earthquake engineering method developed by the Pacific Earthquake Engineering Research centre and outlined in FEMA P-58 [16]. The governing equation behind the loss estimation methodology is:

$$\lambda(DV) = \iiint G\langle DV|DM\rangle \cdot dG\langle DM|EDP\rangle \cdot dG\langle EDP|IM\rangle \cdot d\lambda(IM) \quad (4)$$

where *IM* is the intensity measure (i.e. measure of ground shaking); *EDP* is the engineering demand parameter (i.e. building structural response); *DM* is the damage measure (i.e. measure of the component damage); *DV* is the decision variable (i.e. monetary losses); $\lambda(DV)$ and $\lambda(IM)$ are the annual rates of exceeding a given value of *DV* and *IM*, respectively; and $G\langle DV|DM\rangle$, $G\langle DM|EDP\rangle$ and $G\langle EDP|IM\rangle$ are probabilities of *DV*, *DM*, and *EDP* conditioned to *DM*, *EDP*, and *IM*, respectively.

Loss estimation is performed in four stages; (i) probabilistic seismic hazard analysis to derive $\lambda(IM)$; (ii) structural analyses using ground motions consistent with the seismic hazard to estimate $G\langle EDP|IM\rangle$; (iii) use of fragility functions to obtain $G\langle DM|EDP\rangle$; and (iv) use of loss distributions, $G\langle DV|DM\rangle$, to predict losses [17]. The PACT software estimated losses using equation (4), with the independent variables/inputs being:

- A hazard curve for the specified intensities (from Yeow *et al.*) (step (i));
- Peak inter-storey drifts, residual drifts, and peak floor accelerations from the non-linear time history analyses using the 220 ground motions, over the eleven hazard levels/intensity measures (step (ii)); and
- A building inventory of damageable components, with associated fragility and consequence functions (steps (iii) and (iv)).

Fragility functions specify the likelihood of reaching a damage state as a function of the peak drift or acceleration demand. Consequence functions indicate the likely repair cost associated with a given damage state. The case-study building inventory (and fragility and consequence functions) from Yeow *et al.* was used and updated to reflect the structural system adopted in this research. The inventory consisted of structural and non-structural components. The damageable components considered were: RC walls, glazing partitions (interior and exterior), precast cladding, plasterboard partition walls (full and partial height), stairs, suspended ceilings, braced ceilings, lighting, a traction elevator, water pipe distribution, sanitary pipe distribution, fire sprinklers, HVAC components, and transformers (100 to 350 kVA).

RC Wall Fragility and Consequence Functions

The information available for RC walls from FEMA P-58 did not recognise the effect of wall length on wall fragility functions. Fragility functions have also been proposed for RC walls by Velez [18] but these also were not set as a function of wall length, despite the work of Grammatikou *et al.* [19] and others indicating that deformation capacity does depend on wall length (or more specifically on shear span to wall length aspect ratio, but for a given building height this ratio is proportional to wall length). In line with the guidance of Priestley et al. (2007), damage states for RC walls are assumed to be governed by steel (tension) or concrete (compression) strains, and hence for different wall lengths a given damage state (a given strain) will occur at different wall curvatures. Therefore, damage states for different wall lengths will be triggered at different inter-storey drifts. Given the apparent limitations with the existing fragility functions for walls, a mechanics-based approach was implemented to determine set tentative fragility functions for the different lengths of walls. These were based on damage states and corresponding concrete and steel strains which were taken from the literature and informed by industry experience following the Christchurch earthquakes (Table 3).

Table 3: Damage states (DS) assumed for RC walls in this study.

DS	Damage description*	Median Strain
1	Light cracking: approximately 4 × L_w** of cracks requiring repairs	$\varepsilon_s = 2\varepsilon_y = 0.0055$ (steel in tension) (i.e. twice the yield strain)
2	Moderate cracking: approximately 20 × L_w** of cracks requiring repairs and minor spalling	0.004 (concrete in compression) (from Priestley and Kowalsky [9])
3	Extensive cracking, spalling, bar buckling and/or concrete crushing	0.06 (steel in tension) (from Priestley and Kowalsky [9])

**see Table 6 for repair works required. **L_w = length of RC wall.*

Damage state 1 is reached at a point where cracking in the RC wall is visible to the naked eye but remains under 0.5 mm. Damage state 2 occurs at a point where cover concrete crushes and crack filling is more extensive, but it remains feasible for the wall not to be replaced. Damage state 3 is reached once a wall section has undergone sufficient damage for it to be cut out entirely and replaced within the wall.

A corresponding strain at which damage state 1 occurs, being a function of crack spacing across the plastic hinge of the wall, is difficult to estimate. Based on industry experience, it was assumed that the damage state would be reached at a steel strain equal to twice the yield strain of steel (0.0055 for 500 MPa reinforcing steel). A corresponding total crack length requiring epoxy injection of four times the wall length was assumed and deemed a reasonable estimate. Similarly, damage state 2 assumes a total crack length requiring epoxy injection of 20 times the wall length and spalling of cover concrete that would require repair. The corresponding fragility function is determined based on a concrete strain of 0.004 in the compression zone, which was assumed to be representative of the onset of spalling [9].

The damage state strains informed corresponding inter-storey drifts, forming the mechanics-based RC wall fragility functions. For each length of wall (3 m, 6 m, 9 m, 16 m), the following process was utilised to determine the drifts triggering each damage state:

1. Moment-curvature analysis is used to convert the damage state strain to a section curvature limit, ϕ, at the base of the wall.
2. The wall displacement profile at the limiting curvature is found using equation (5) from Sullivan et al. (2012).

$$\Delta_i = \frac{\phi}{2} H_i^{\,2}\left(1-\frac{H_i}{3H_n}\right) + \theta_p H_i + \theta_f H_i \tag{5}$$

   where Δ_i = displacement at the i[th] storey; H_i = wall height at the i[th] storey; H_n = total wall height; θ_p = plastic hinge rotation; and θ_f = foundation rotation. In equation (5), the curvature, ϕ, was capped at an expected yield curvature, ϕ_y, which for rectangular RC walls was taken from equation (2).

   The foundation rotation was computed according to the moment demand, which was obtained as a function of the wall base curvature.

   The plastic hinge rotation was given by equation (6) (from Sullivan et al [20]) if the wall had yielded.

$$\theta_p = \left(\phi - \phi_y\right) L_p \tag{6}$$

   where L_p = plastic hinge length, determined from the method first proposed by Berry *et al.* [21] and modified by Cordero *et al.* [22] (based on analysis of a range of past experimental results) to:

$$L_p = 0.058 H_n + 0.1\frac{f_y d_b}{\sqrt{f'_c}} \tag{7}$$

   where f_y = steel yield strength; d_b = longitudinal reinforcement diameter; and f'_c = concrete strength.

3. The wall displacement profile is converted into inter-storey drifts to define the median drift capacity for the RC wall fragility function (with the bottom storey drift taken as the fragility).

This method to set fragility functions for the RC walls should be verified and improved using the results of experimental testing as part of future research. However, it is argued that the preliminary fragility functions better capture the effects of wall length on losses than existing fragility functions, and hence are a more useful tool for the purposes of this investigation. Furthermore, note that because the wall length affects the curvature results in step 1 and the wall height affects the displacements in step 2 (and equation (7)), the relevance of the wall shear span ratio to the deformation capacity of walls (observed from a large experimental database by Grammatikou *et al.*) becomes evident.

Upper and lower bound drift capacity values for the RC walls were obtained by the method described above. The variables considered to obtain upper and lower bounds were axial load, reinforcement ratio, and wall height. These variables were bounded by the limiting values from the 16 building designs. For example, the bounds of the 16 m walls were derived from the axial loads, reinforcement ratios and wall height from the 12-storey building alone (only building height with 16 m walls). An average of the upper and lower bound drift capacities was assumed to be the median drift for the preliminary fragility functions (Table 4). It was noted that generally, wall fragility increased with wall length (lower drift limits for each damage states). This was because for longer walls, a lower section curvature (and consequently lower drifts) was required to reach the governing damage state strain. Due to the natural variability and uncertainty in the characteristics and behaviour of the walls, a dispersion of 0.3 was assumed for all wall fragility functions.

RC wall fragility functions determined from this mechanics-based approach were paired with consequence (cost) functions. These were derived from expected repair methods and corresponding costs in the NZ industry (2020/21), published in Fox *et al.* [23]. Repair methods for damage states 1-3 for RC walls are described in Table 5. Corresponding costs and the associated coefficient of variations are summarised in Table 6. The RC wall fragility functions were 'triggered' via drifts at the ground floor (only), with corresponding consequences representing full repair of the wall for given damage stages.

Table 4: Preliminary fragility functions of RC walls from the mechanics-based approach (dispersion of 0.3 assumed for each fragility function). To be validated and refined as part of future research.

	Median bottom storey drift capacity for 4-storey buildings			Median bottom storey drift capacity for 12-storey buildings	
DS	***3 m wall***	***6 m wall***	***9 m wall***	***9 m wall***	***16 m wall***
1	0.0049	0.0029	0.0026	0.0026	0.0019
2	0.0111	0.0063	0.0066	0.0066	0.0049
3	0.0321	0.0149	0.0199	0.0199	0.0210

Table 5: Assumed repair methods for the damage states of an RC wall (after Fox et al.).

DS	Repair description
1	Repair cracks using a high-pressure low viscosity crack injection epoxy (e.g. Sikadur 52) in accordance with the manufacturer's instructions. Allow to grind back injection ports and crack lines, skim coat wall to a uniform finish. Painting/recoating of finishes.
2	Repair cracks using a high-pressure low viscosity crack injection epoxy (e.g. Sikadur 52) in accordance with the manufacturer's instructions. Break back spalled concrete to competent concrete and repair using a high build structural repair mortar (e.g. Sika MonoTop 352) in accordance with the manufacturer's instructions and ensuring adequate bond to the reinforcing. Allow to grind back injection ports and crack lines, skim coat wall to a uniform finish. Painting/recoating of finishes.
3	Break out damaged section of wall to competent concrete. Mechanically splice replacement reinforcing with existing longitudinal reinforcing and recast wall using high-strength concrete with 10 mm aggregate.

Table 6: Assumed costs and corresponding coefficients of variation associated with damage states in RC walls.

Wall length:	3 m		6 m		9 m		16 m	
Damage state	*Cost (NZD)*	*COV*	*Cost (NZD)*	*COV*	*Cost (NZD)*	*COV*	*Cost (NZD)*	*COV*
1	$13,683	0.61	$16,083	0.58	$18,483	0.56	$24,083	0.52
2	$28,340	0.49	$40,340	0.46	$52,340	0.45	$78,756	0.43
3	$46,048	0.20	$53,596	0.27	$61,144	0.33	$78,756	0.42

The wall repair costs reported in Fox *et al.* were developed by an experienced contractor and consulting engineering company both involved in the rebuild following the Canterbury earthquakes. The values reported in Table 6 have been amended to account for the estimated crack lengths to be repaired for different lengths of walls. Surrounding works including removal of partitions, floor finishes, and scaffolding are included in these costs. The coefficients of variation were determined from costs and crack lengths assumed at the 10th and 90th percentiles reported in Fox *et al*. Further variation was included by assuming 10th and 90th percentile crack lengths of 50% more and less than the mean assumed crack lengths respectively. That is, two times and six times the wall length for damage state 1 and 10 times and 30 times the wall length for damage state 2. This increased variation was included to better acknowledge the large uncertainty in the crack lengths, expected damage and the associated repair costs for RC walls.

The authors acknowledge the significant uncertainty in the fragility and consequence functions of RC walls, primarily associated with the extent of wall cracking. Absolute loss values for RC walls determined through this research should be used with care and future research should be undertaken to better define RC wall fragility functions. The fragility and consequence functions defined here are intended to better capture the effects of wall length and shear span ratio on expected losses compared to those provided in FEMA P-58 and elsewhere in the literature. Despite the uncertainty in these functions and hence in the loss results, it is expected that the relative differences in expected losses between the various building designs undertaken would not be affected. Consequently, recommendations and observations listed throughout the paper are formed from relative results only, which are likely to maintain their validity despite the uncertainty in the absolute loss results associated with the RC walls.

Building Values

The 4-storey and 12-storey building values assumed in accordance with the research by Yeow et al. (2018) are shown in Table 7. The building replacement cost was assumed to correspond to 125% of the building value, and the core and shell replacement cost corresponded to 60% of the building value (from FEMA P-58 [16]). The 4-storey and 12-storey buildings have the same assumed cost/m^2.

Table 7: Assumed value of the 4-storey and 12-storey buildings (NZD, 2020).

Christchurch		Wellington	
4-storey	*12-storey*	*4-storey*	*12-storey*
$ 9.32 m	$ 44.5 m	$ 8.59 m	$ 41.2 m

Total Loss Threshold and Residual Drifts

The loss assessment included a total loss threshold and residual drifts beyond which the full building replacement cost was triggered. A total loss threshold of 50% of the total core and shell replacement cost was assumed reasonable event though it is noted that Elwood et al. [24] reports lower values were observed in the Canterbury earthquakes. A median irreparable residual story drift ratio of 0.01 was specified in the model with a corresponding dispersion of 0.3. Where the total repair costs or residual drift of the building for a given earthquake exceeded these respective values, the building was assumed to be replaced in full. These limits were based on recommendations produced in FEMA P-58.

The effect of the inclusion of a total loss threshold in the loss assessment was investigated by sensitivity analyses. Varying total loss thresholds (80%, 65%, 50%, 25%, 15%) were specified for the 4-storey Christchurch building with the conventional design. For each building, the corresponding expected annual loss (EAL) was recorded (see Table 8).

Table 8: Expected annual losses for varying total loss thresholds (4-storey, conventional design, Christchurch).

Total loss threshold (%)	Expected annual loss (%)
80	0.15
65	0.15
50*	0.17
25	0.19
15	0.31

**suggested by FEMA-P58*

The results show little change in the EAL of the building at loss thresholds set higher than the recommended 50%. An exponential increase is noted as the loss threshold decreases. However, significant increases in the EAL were noted below a loss threshold of 25% only. A 50% total loss threshold was hence deemed to be appropriate for this study.

The effect of the inclusion of residual drifts in the loss model on the EAL of a building was also investigated. The residual drift limit was considered as a trigger for the full replacement cost of the building (the residual drift at which the building may be replaced). This comparison was made for all buildings designed with Priestley's stiffness approach. The corresponding results are shown in Table 9 (Christchurch buildings) and Table 10 (Wellington buildings).

Table 9: Expected annual losses for different Christchurch designs (Priestley's stiffness), with and without a residual drift limit to trigger full replacement cost.

	4-Storey		12-Storey	
Residual drift limit included in loss model	***Conv.***	***LDSD***	***Conv.***	***LDSD***
No	0.15	0.18	0.06	0.10
Yes	0.16	0.18	0.06	0.10

Table 10: Expected annual losses for different Wellington designs (Priestley's stiffness), with and without a residual drift limit to trigger full replacement cost.

	4-Storey		12-Storey	
Residual drift limit included in loss model	***Conv.***	***LDSD***	***Conv.***	***LDSD***
No	0.20	0.22	0.10	0.11
Yes	0.22	0.22	0.11	0.11

The inclusion of residual drifts in the loss assessment appears to increase the EAL, particularly for buildings of higher drift (conventional buildings). However, these increases generally appear to be low. It was noted that there was large uncertainty in the residual drifts recorded from time history analyses, and there is also contention around the residual drift limit at which a building might be replaced. Due to the considerable uncertainty in this approach, and due to not being critical to the outcomes of this study, residual drifts were not included in the detailed loss assessment.

Loss Assessment Results and Discussion

Loss assessments were completed for the 16 building designs based on the NLTHA results. The key results from the designs produced using Priestley's stiffness method are shown in Table 11 (Christchurch) and Table 12 (Wellington).

Table 11: Loss results for conventional and low-damage designs using Priestley's stiffness method, Christchurch.

	4-Storey		12-Storey	
	Conv.	***LDSD***	***Conv.***	***LDSD***
Total Expected Annual Loss*	0.15%	0.18%	0.06%	0.10%
Expected Annual Loss from drift-sensitive components	0.05%	0.01%	0.02%	0.01%
Chance of a repair cost greater than 5% of building replacement cost over 10 years**	1/12	1/10	1/51	1/23

**Should be 0.1% or less for a low-damage building, according to draft LDSD advice.*

***Should be a 1/25 chance, or less likely, for a low-damage building, according to draft LDSD advice.*

Table 12: Loss results for conventional and low-damage designs using Priestley's stiffness method, Wellington.

	4-Storey		12-Storey	
	Conv.	***LDSD***	***Conv.***	***LDSD***
Total Expected Annual Loss*	0.20%	0.22%	0.10%	0.11%
Expected Annual Loss from drift-sensitive components	0.06%	0.01%	0.03%	0.01%
Chance of a repair cost greater than 5% of building replacement cost over 10 years**	1/10	1/8	1/18	1/16

**Should be 0.1% or less for a low-damage building, according to draft LDSD advice.*

***Should be a 1/25 chance, or less likely, for a low-damage building, according to draft LDSD advice.*

LDSD building results shown in Table 11 and Table 12 do not exactly align with the performance objectives stated by the design advice. 4-storey and 12-storey buildings in Christchurch and Wellington returned an EAL of 0.18%, 0.10%, 0.22%, and 0.11% respectively. This is up to 120% larger than the EAL stated as a performance objective of the draft LDSD advice (0.1%). The chances of incurring a repair cost greater than 5% of the building replacement cost over 10 years appears to be between 1/10 and 1/8 for 4-storey buildings (compared to the performance objective of 1/25), suggesting a significantly larger loss probability than desired.

An exact alignment of the LDSD results to the performance objectives was not expected given the uncertainty of loss assessment and because absolute EAL results are very sensitive to the assumed building cost, building inventory, the lateral system used, and the inherent uncertainty of the loss model. Moreover, the building inventory used in this study assumes current/conventional design strengths of acceleration sensitive components whereas the draft LDSD advice proposes increasing acceleration demands for the design of these components, which should further reduce losses (as discussed in later sections). As such, the best indicator of the loss for the LDSD buildings are arguably the drift-sensitive losses shown in Tables 11 and 12 which does look promising. However, the research is unable to confirm the absolute numerical performance objectives stated and demonstrates that for the draft low-damage design advice to reduce losses as per its intentions, the drift limit at DCLS cannot be solely relied upon. It is therefore equally important that the design acceleration demands of non-structural components are evaluated using suitable methods which are expected to be included in the LDSD advice (discussed further in later sections).

The results show a similar and higher total EAL for the low-damage design compared to the conventional design for the 4-storey and 12-storey buildings respectively. In terms of overall losses, the research does not show a clear benefit in LDSD compared to conventional buildings. However, LDSD buildings yielded significantly lower EAL for drift-sensitive components compared to the conventional design buildings (with the opposite true for acceleration-sensitive losses). All buildings appear to show an average proportional decrease of 60-80% in total and drift-sensitive losses from conventional to LDSD designed buildings. Observed trends do not appear to vary significantly between buildings in Wellington and Christchurch (though Wellington buildings tended to experience higher acceleration demands and acceleration-sensitive losses).

It should be noted that the conventional buildings drifted much less than the 2.5% limit in the non-linear time history analyses (Figure 7(a) to Figure 10(a)). Much higher drift-sensitive losses would be expected in buildings that drift closer to the 2.5% limit. Therefore, there may be an increased reduction in drift-sensitive damage from conventionally designed buildings to LDSD buildings.

It is noted that, counterproductively, the chances of incurring a repair cost greater than 5% of the building replacement cost over 10 years tend to increase for LDSD buildings. LDSD buildings thus appear to have a larger probability to incur small building repair costs compared to conventional buildings. Conventional buildings appeared to generally incur significantly larger losses during less frequent, high intensity earthquakes compared to LDSD buildings. On the contrary, LDSD buildings appeared to incur damages exceeding 5% of the building's replacement cost more frequently. This is due to increased acceleration-sensitive losses for lower intensity earthquakes, which is particularly pronounced in LDSD buildings. This observation is further explained in the following sections.

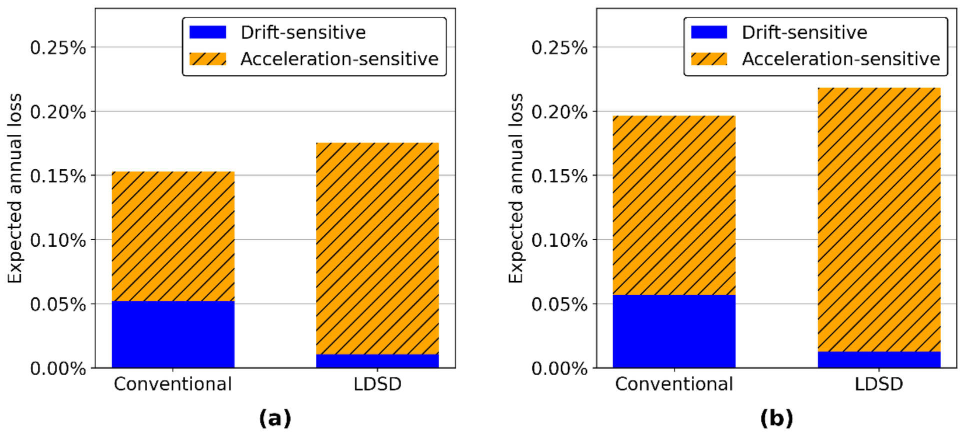

Figure 12: Summary of expected annual losses for the two 4-storey building designs (using Priestley's stiffness) in Christchurch (a) and Wellington (b), showing contributions from drift and acceleration sensitive components.

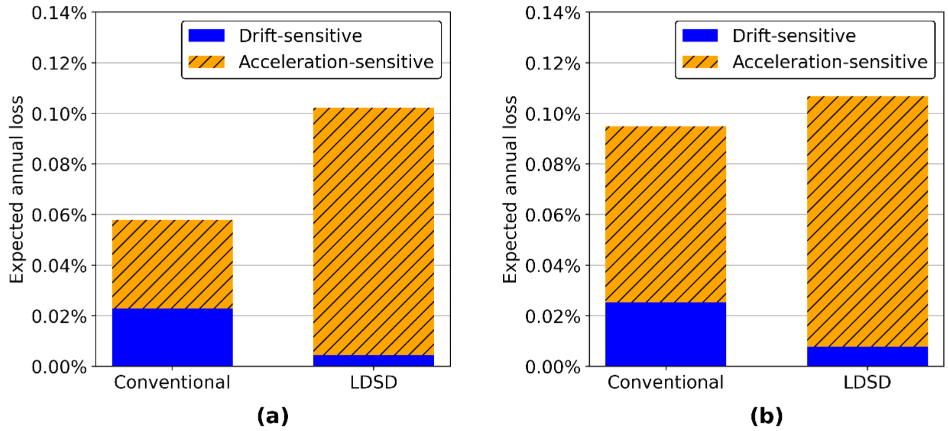

Figure 13: Summary of expected annual losses for the two 12-storey building designs (using Priestley's stiffness) in Christchurch (a) and Wellington (b), showing contributions from drift and acceleration sensitive components.

Breakdown of EALs and Acceleration-Sensitive Losses

A summary of the total expected annual losses for the 8 building designs using Priestley's stiffness is shown in Figure 12 and Figure 13 for the 12- and 4-storey buildings respectively. The acceleration- and drift-sensitive losses within buildings are separated to demonstrate relative loss contributions of these components. Notably, acceleration-sensitive losses appear to form the majority of the EAL for low-damage RC wall designs. Whilst the building drift and its associated losses decrease with a lower drift limit, corresponding floor accelerations and their associated losses appear to generally increase (see Figure 7(a) to Figure 10(a)). This highlights a potential downside of limiting drift in a low damage design: stiffer buildings can experience higher floor accelerations and this needs to be accounted for when designing or verifying acceleration-sensitive components. It is noted that differences in proportions of acceleration-sensitive losses might be expected for other building types, such as frame buildings, due to their inherently different structural behaviour. Quantifying this difference is not within the scope of this research.

The composition of typical acceleration-sensitive losses was interrogated. A corresponding breakdown of the contribution of components to these acceleration-related losses is shown in Figure 14 (for the 4-storey, Wellington, conventional building with Priestley stiffness). Insignificant variation to these results is found between the 8 designs. Notably, damage to suspended and braced ceilings form the majority of these losses (~35-50% for Wellington and ~50-65% for Christchurch buildings). A breakdown of the cumulative contribution of building losses to the EAL over the 11 earthquake intensities is shown in Figure 15 (for the 4-storey, Wellington building designed with Priestley stiffness). The rate of accumulation of acceleration-sensitive losses at low return periods (around 20 to 500 years) is significantly larger than drift-sensitive losses. While variations in this accumulation are noted between different buildings, this observation explains the following tendency found through this study: that the chances of incurring a repair cost greater than 5% of the building replacement cost over 10 years tends to be larger for LDSD buildings. The significantly larger proportion of acceleration losses for the LDSD (compared to conventional) buildings appears to increase overall building losses at low intensities which frequently exceed 5% of the building repair cost.

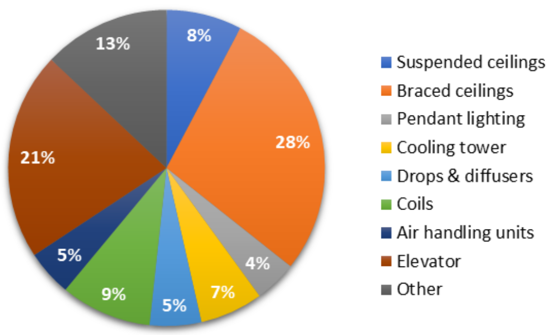

Figure 14: Components' contribution to the EAL due to acceleration (4-storey, Wellington, conventional design with Priestley stiffness).

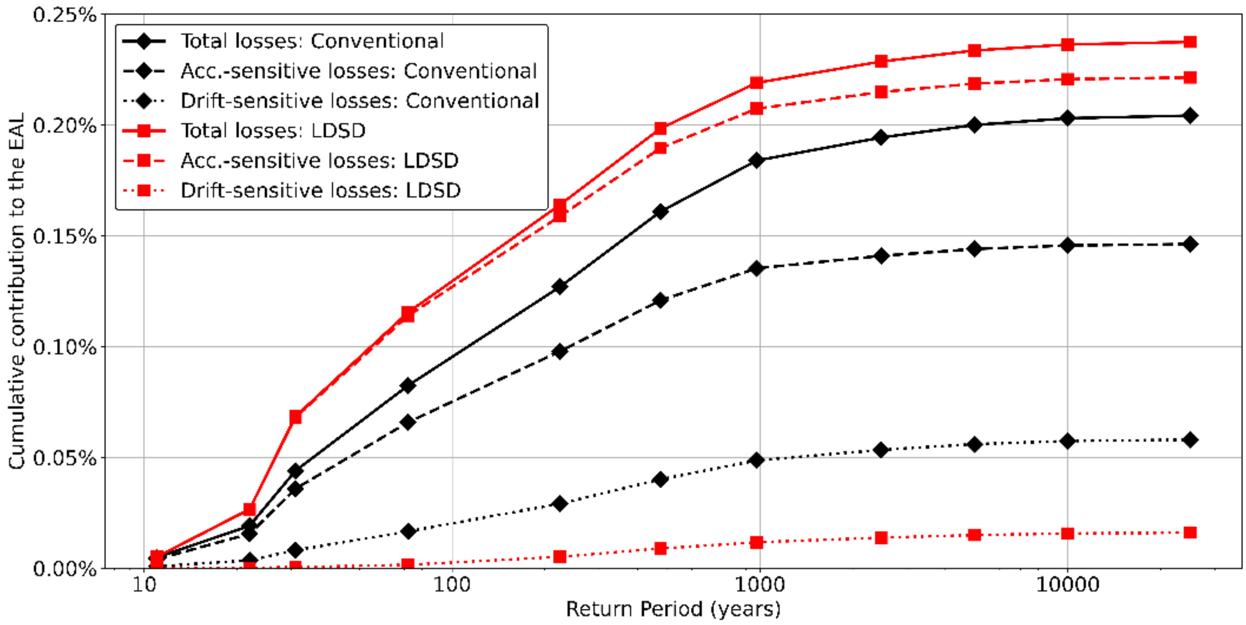

Figure 15: Cumulative contribution of building losses to the EAL over the 11 earthquake return periods (4-storey, Wellington, design with Priestley stiffness).

The large proportional losses of acceleration-sensitive components, particularly from lower intensity earthquakes, raises questions around the suitability of their current design criteria. Results in the research suggest that large reductions in the components' probable annual repair costs may be made by increasing the strength criteria of these components (particularly ceilings and air handling units). The ceiling fragilities reflected a design to SLS1 criteria [25], which was typical in NZ until recently. LDSD guidelines, in comparison, recommend acceleration-sensitive components be designed to DCLS loading at a ductility factor of 1.0. This research hence supports the intention of the LDSD guidelines to increase the strength requirement of acceleration-sensitive components under LDSD. Such requirements, in parallel to the lowering drifts limits, may provide an economical solution to further decreasing building losses. One other solution for reducing losses from ceilings is to remove ceilings from the fitout, where appropriate. This recommendation should however consider the wider context and desired functionality of the building structure (such as architectural, fire, or acoustic performance which may require ceilings). A related study on the impact of acceleration-sensitive components on the seismic losses of multi-storey office buildings was published by the authors in support of these recommendations [26].

Amongst the significant uncertainties in loss estimation is the uncertainty in the dynamic response of non-structural acceleration-sensitive components given possible variations in the frequency content of floor response. The uncertainty in the relationship between peak floor acceleration and component damage is reflected in the high dispersion values that characterise the fragility functions of acceleration-sensitive non-structural components. To reduce such uncertainty, future research could aim to identify other EDPs that result in fragility functions with lower dispersions.

Overall, the study has shown that low-damage seismic design criteria could be an effective way of limiting drift-sensitive damage and losses only. Acceleration-sensitive losses must be addressed and effectively reduced such that a reduction of overall losses can be expected. One should consider the differences in capital cost of conventional to LDSD design solutions to ultimately decide which design option is financially the most optimum. This consideration is, however, outside the scope of this research and ultimately the choice for adopting LDSD may not be dictated just by costs.

Losses in 12 storey Buildings

It is noted that 12-storey LDSD buildings tend to result in larger overall EALs compared to the conventional designs. Other loss-assessment studies in the literature have also observed that the EAL for tall buildings tends to be low since damage does not occur over all storeys and the EAL is a measure of repair costs relative to building replacement cost. 12-storey buildings tend to have longer periods of vibration which would reduce first mode acceleration demands but higher mode effects are more likely to impact the response of taller buildings. Peak accelerations felt across all storey levels hence tend to be larger on average relative to storey drifts in comparison with smaller buildings. These factors cause the ratio between acceleration-sensitive and drift-sensitive losses as a contribution to the total EAL to be larger in taller compared to shorter buildings. Within 12-storey buildings, acceleration-sensitive losses tend to form the majority of the EALs.

As the increases in stiffness for LDSD buildings results in an increase of acceleration-sensitive losses, 12-storey LDSD buildings are found to yield higher overall EALs compared to conventional buildings (despite a general decrease in drift-sensitive losses). The draft LDSD advice covers buildings up to and including 6 storeys high. Given that acceleration-sensitive losses appear to dominate taller buildings, the importance of following the proposed methodology to design acceleration-sensitive components to the DCLS becomes critical. If acceleration-sensitive components are strengthened sufficiently for taller buildings, the draft LDSD advice (and related performance objectives) could be extended to taller buildings. However, it is suggested that sufficient consideration must be given to the more complex structural behaviour for taller buildings when determining floor acceleration demands for the DCLS design. It remains important for the designer not to blindly stiffen the building to reduce drifts, as this may hinder their ability to sufficiently control accelerations.

Table 13: Contribution of RC walls as a proportion of the total EAL of the building designs for 4-storey building designs (using Priestley's stiffness).

Location	Wellington				Christchurch			
Design	*Conventional*		*LDSD*		*Conventional*		*LDSD*	
Length of RC walls (number, length)	6 x 3 m	2 x 6 m	6 x 6 m	2 x 9 m	6 x 3 m	2 x 6 m	6 x 6 m	2 x 9 m
EAL contributed by RC walls (% of building replacement cost)	0.010%	0.008%	0.003%	0.003%	0.005%	0.011%	0.003%	0.004%

Table 14: Loss results for low-damage designs, Christchurch.

	Total Expected Annual Loss (%)* for design with effective stiffness method:		Expected Annual Loss from drift-sensitive components (%) for design with effective stiffness method:	
Building height	*Priestley*	*NZS 3101*	*Priestley*	*NZS 3101*
4-storey	0.18%	0.22%	0.01%	0.04%
12-storey	0.10%	0.05%	<0.01%	0.01%

**Should be 0.1% according to draft LDSD advice.*

Table 15: Loss results for low-damage designs, Wellington.

	Total Expected Annual Loss (%)* for design with effective stiffness method:		Expected Annual Loss from drift-sensitive components (%) for design with effective stiffness method:	
Building height	*Priestley*	*NZS 3101*	*Priestley*	*NZS 3101*
4-storey	0.22%	0.20%	0.01%	0.03%
12-storey	0.11%	0.10%	0.01%	0.02%

**Should be 0.1% according to draft LDSD advice.*

RC Wall Selection and Associated Losses

Results from the loss assessments gave further insight into the effectiveness of various RC wall arrangements with respect to their contribution to the total EAL. This was obtained where buildings were designed with varying arrangements of RC walls in each direction (4-storey buildings only). A summary of the EAL contribution of RC walls to the total EAL of the building is shown in Table 13.

Losses of different RC wall systems appeared to be variable, with their performance being dependant on a range of factors linked to the intrinsic behaviour of wall systems and that of the building as a whole. These factors include the number of walls present, the length of the walls, and the levels of drift experienced by the building. For a given drift and wall length, losses due to RC wall damage increase with the number of walls present. At higher drifts, walls tend to experience higher levels of damage. Longer walls have a higher fragility and repair cost for a given damage state compared to shorter walls. Longer walls reach damage states at lower drifts and carry higher corresponding repair costs due to their larger damageable area. While buildings with long walls may be expected to drift less for the same intensity, this is not ensured through the design process because it is targeting a 0.5% drift limit at the DCLS intensity. Furthermore, longer walls may incur additional costs due to the larger foundation requirements that may result. The losses experienced by different wall systems are a function of these interlinked factors which should all be considered in the specification of RC walls.

In general, stiffer RC wall buildings tend to be more expensive to fix at given building drifts because they either require longer walls (more fragile) or more walls (more area to fix). Thus one might question the suitability of RC walls to serve as an effective lateral load resisting system (LLRS) for LDSD buildings and yet the results in this study suggest that it is possible. Nevertheless, an investigation into losses of RC walls compared to other lateral load resisting systems is recommended as part of future research.

Losses of Buildings Designed with the NZS 3101 Stiffness Method

Key loss assessment results for the building designs using the NZS 3101 stiffness method (the 'code-compliant' designs) are compared to those using the Priestley stiffness method in Table 14 (Christchurch) and Table 15 (Wellington). While similar relative performances are noted across the respective building designs, a larger magnitude in drift-losses was generally obtained for buildings designed using the NZS 3101 stiffness compared to those designed with Priestley's stiffness approach. This was because the buildings designed with Priestley's stiffness generally drifted less (Figure 7(a) to Figure 10(a)), as the design stiffness was lower than the NZS 3101 design stiffness (which overestimated the wall stiffness).

Buildings designed using Priestley's stiffness appear to yield lower drift-sensitive losses compared to buildings designed with the NZS 3101 stiffness approach. While the results show large variations, buildings designed via Priestley's stiffness on average returned around 50-80% lower EAL results than buildings designed via the NZS 3101 stiffness approach for conventional and LDSD buildings (all heights) respectively.

The reduction in the losses of drift-sensitive components can partly be attributed to the significantly lower losses in RC walls designed with Priestley's stiffness. When designed with Priestley's stiffness, RC walls generally contributed 25% to 75% less to the total EAL compared to RC walls designed with the NZS 3101 method. As explained previously, Priestley's stiffness approach allowed for a lower number of walls (with higher reinforcement content) to be specified compared to the NZS 3101 stiffness approach (by accounting for the effect of reinforcement content on wall stiffness). This resulted in an overall reduction in the contribution of the wall system to the building losses, which further encourages the adoption of Priestley's stiffness as a means of designing RC walls under LDSD.

CONCLUSIONS

The effectiveness of draft LDSD advice for reducing earthquake-induced losses for RC wall buildings was investigated by design and assessment of 4- and 12-storey RC wall buildings, located in Wellington and Christchurch. The results of the study did not exactly align with the performance objectives of the LDSD advice (0.1% EAL and at most a 1 in 25 chance of a repair cost greater than 5% of the building replacement cost for a 10-year period). Considering the uncertainty in the loss assessment and the comparison of relative results between conventional and LDSD buildings showed that the study can holistically support the intent of the draft LDSD advice in reducing drift-sensitive losses (as no trend in the reduction of total losses was noted). Losses attributed to acceleration-sensitive non-structural elements appeared to increase significantly after imposing the 0.5% interstorey design drift limit at the new DCLS, counteracting and in most cases exceeding the reduction in drift-sensitive losses. With improved acceleration-sensitive component design provisions for LDSD, acceleration-losses could be appropriately controlled and significantly reduced. Reductions in total losses could thereby likely be achieved, to a level expected by the LDSD performance objectives. These conclusions hold true for buildings in both Wellington and Christchurch, although Wellington buildings tended to experience higher acceleration demands and acceleration-sensitive losses.

The investigation found the proposed LDSD criteria to be effective at reducing drift-losses only when compared to the current NZ design standards for RC wall buildings. To achieve an overall reduction of building losses, the study produced some recommendations to improve the performance of LDSD buildings:

1. To achieve the desired loss performance, strengthening of acceleration-sensitive components should be considered as per the recommendations of the draft LDSD guidance advice. Such components have often been designed for SLS1. These should instead be designed for higher acceleration demands (perhaps at the DCLS) to ensure losses are minimised for more frequent seismic events. This is particularly important for low-damage RC wall buildings which are likely to be much stiffer to meet drift criteria, and hence more susceptible to very high floor accelerations. For example, ceilings could be better considered in low-damage buildings by strengthening them, or omitting them where appropriate, to obtain greater reductions in loss.
2. For RC wall buildings, detailed consideration should be given to the method by which RC wall stiffness is determined. This becomes particularly important when drift becomes the primary building performance measure. The NZS 3101 method of approximating wall stiffness was found to be oversimplified as it did not consider the effect of reinforcement on the stiffness and may overestimate effective wall stiffness at low reinforcement ratios. Priestley's method [11] of determining wall stiffness as a function of wall strength is recommended. The method is a better approximation of wall stiffness, which is critical when building drift is the governing design criterion. It also allows for fewer walls of higher reinforcement content to be designed.
3. Losses of RC wall systems depend on a range of factors including the number of walls present, the length of the walls, and the levels of drift experienced by the building. For given drift and wall length, losses due to RC wall damage increase with the number of walls present. At higher drifts, walls tend to experience higher levels of damage. Longer walls have higher repair costs per wall, because of their larger repair cost at given damage states and higher fragilities. Losses experienced by different wall systems are a function of these interlinked factors which should all be considered in the specification of RC walls. An effective method found to minimise the contribution of RC walls to overall building losses is to use a high reinforcement ratio, thus reducing the number of walls required to achieve the required lateral stiffness, provided that maximum quantities of longitudinal reinforcement are respected and capacity design provisions can be satisfied. The ability of the designer to do this will be aided by using an approximation for RC wall stiffness such as Priestley's method, which acknowledges the relationship between strength and stiffness.
4. Taller RC wall buildings could be included in the draft LDSD advice (which only applies for buildings up to six stories). The drift-sensitive losses of taller RC wall buildings, as a fraction of the building replacement cost, are lower because the peak storey drift demands tend to vary quite significantly over the building height (a lower proportion of storeys reach peak storey drift). Provided the first recommendation is considered and acceleration-sensitive losses are minimised (by strengthening acceleration-sensitive components), it is suggested that the loss performance of taller RC wall buildings should meet the performance objectives set by the draft LDSD advice.

   While the research was able to produce reasonable RC wall LDSD designs for 4-storey buildings, the large number of long walls required for the 12-storey LDSD designs was noted. One might hence question the suitability of RC walls as the LLRS for very tall LDSD buildings as very long or thick walls may compromise available space for components such as glazing and passageways within the building. For such building types, it is recommended to consider other LLRS options.

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APPENDIX: CASE STUDY BUILDING WALL DESIGNS

Information on the wall designs and modelling parameters for the case-study buildings are summarised in Table 16 to Table 21.

Table 16: 4-storey, Christchurch: Wall designs for the four cases, in direction 1.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Six 3 m x 0.49 m	0.60%	1.2%	0.87 s	1.17 s	Drift limit (ULS)	1.0 s
Low-damage	Six 6 m x 0.35 m	0.39%	1.2%	0.39 s	0.61 s	Drift limit (DCLS)	0.5 s
Conventional (Priestley stiffness)	Six 3 m x 0.40 m	1.84%	1.2%	0.78 s	0.72 s	Drift limit (ULS)	1.0 s
Low-damage (Priestley stiffness)	Six 6 m x 0.28 m	2.06%	1.8%	0.38 s	0.38 s	Drift limit (DCLS)	0.5 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*

Table 17: 4-storey, Christchurch: Wall designs for the four cases, in direction 2.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Two 6 m x 0.24 m	0.92%	3.9%	0.79 s	0.86 s	Drift limit (ULS)	1.0 s
Low-damage	Two 9 m x 0.32 m	0.56%	2.4%	0.41 s	0.56 s	Drift limit (DCLS)	0.5 s
Conventional (Priestley stiffness)	Two 6 m x 0.20 m	1.92%	4.5%	0.83 s	0.82 s	Drift limit (ULS)	1.0 s
Low-damage (Priestley stiffness)	Two 9 m x 0.24 m	2.14%	2.9%	0.41 s	0.41 s	Drift limit (DCLS)	0.5 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*

Table 18: 4-storey, Wellington: Wall designs for the four cases, in direction 1.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Six 3 m x 0.50 m	0.54%	1.2%	0.85 s	1.20 s	Strength (ULS)	1.0 s
Low-damage	Six 6 m x 0.44 m	0.39%	1.6%	0.35 s	0.55 s	Drift limit (DCLS)	0.5 s
Conventional (Priestley stiffness)	Six 3 m x 0.37 m	2.09%	1.2%	0.86 s	0.82 s	Drift limit (ULS)	1.0 s
Low-damage (Priestley stiffness)	Six 6 m x 0.34 m	2.14%	1.6%	0.34 s	0.34 s	Drift limit (DCLS)	0.5 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*

Table 19: 4-storey, Wellington: Wall designs for the four cases, in direction 2.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Two 6 m x 0.20 m	0.94%	4.5%	0.86 s	0.97 s	Strength (ULS)	1.0 s
Low-damage	Two 9 m x 0.32 m	0.59%	1.3%	0.41 s	0.55 s	Drift limit (DCLS)	0.5 s
Conventional (Priestley stiffness)	Two 6 m x 0.20 m	1.77%	4.5%	0.91 s	0.87 s	Drift limit (ULS)	1.0 s
Low-damage (Priestley stiffness)	Two 9 m x 0.29 m	2.11%	1.3%	0.38 s	0.39 s	Drift limit (DCLS)	0.5 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*

Table 20: 12-storey, Christchurch: Wall designs for the four cases, in both directions.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Six 9 m x 0.48 m	0.36%	3.3%	1.80 s	2.73 s	Strength (ULS)	2.0 s
Low-damage	Four 16 m x 0.87 m	0.32%	3.2%	0.70 s	1.18 s	Drift limit (DCLS)	1.0 s
Conventional (Priestley stiffness)	Six 9 m x 0.38 m	2.12%	3.5%	1.79 s	1.69 s	Drift limit (ULS)	2.0 s
Low-damage (Priestley stiffness)	Four 16 m x 0.75 m	2.14%	3.3%	0.66 s	0.68 s	Drift limit (DCLS)	0.5 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*

Table 21: 12-storey, Wellington: Wall designs for the four cases, in both directions.

Design case	Wall arrangement	Reinforcement ratio*	Axial load ratio	Design period	Period of NLTHA model	Critical design criteria**	Ground motion conditioning period
Conventional	Six 9 m x 0.42 m	0.32%	3.4%	1.93 s	2.99 s	Strength (ULS)	2.0 s
Low-damage	Four 16 m x 0.63 m	0.32%	3.4%	0.82 s	1.38 s	Drift limit (DCLS)	1.0 s
Conventional (Priestley stiffness)	Four 9 m x 0.50 m	2.14%	3.6%	1.91 s	1.80 s	Drift limit (ULS)	2.0 s
Low-damage (Priestley stiffness)	Four 16 m x 0.53 m	2.14%	3.5%	0.79 s	0.81 s	Drift limit (DCLS)	1.0 s

**0.32% minimum in accordance with NZS 3101.*

***Design criteria which most closely dictated the wall design in number, reinforcement ratio or geometry.*