Assessing the impact of acceleration-sensitive components on the 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. A preliminary proposal for LDSD is to utilise a design drift limit for multi-storey buildings of 0.5% associated with a new damage control limit state (DCLS). A previous study by the authors found that applying a 0.5% drift limit without other provisions may not deliver the expected seismic loss performance because of the higher floor accelerations (and subsequent increased acceleration-sensitive losses) which result as a side effect of needing a stiffer building. This paper emphasises the need to design acceleration-sensitive non-structural components in LDSD buildings for the damage control limit state to reduce the expected annual loss to a level intended by the advice. The response of RC wall case-study buildings of 4- and 12-storeys, designed to both current code criteria and draft LDSD criteria, are used as input for loss assessment. Loss assessments were completed for three cases: 1) conventionally designed buildings with ceilings using standard fragility functions; 2) LDSD buildings with standard ceilings; and 3) LDSD buildings with ceilings using lower fragilities (stronger components designed to DCLS accelerations). Significantly lower losses were recorded for LDSD buildings with strengthened ceilings compared to LDSD buildings with standard ceilings, bringing loss performance in line with the expectations of the draft LDSD advice and highlighting the need for design advice to consider both drift and acceleration design criteria to achieve true low-damage performance.

1 INTRODUCTION

Low damage seismic design (LDSD) has been a focal point of the structural engineering industry following the Christchurch earthquake sequence. The Ministry of Building, Innovation and Employment is developing advice on how to deliver LDSD protection from buildings, as part of their Tū Kahika: Building Resilience platform. 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.

The authors have previously investigated (John et. al 2022) how the early draft LDSD advice might affect building performance, primarily quantifying the expected monetary losses associated with post-earthquake repair of buildings designed following the advice. That study, currently under review for possible publication in the Bulletin of the NZSEE, examined four case-study commercial buildings with RC wall lateral systems. The buildings were of 4 and 12 storeys and were located in both Wellington and Christchurch (selected to consider the effects of varying building height and seismic hazard). Probabilistic loss assessments for the buildings were completed following FEMA P-58 (FEMA 2018a), using the results from non-linear time history analyses, to determine the expected losses of the case study buildings designed without the draft LDSD advice (a conventional, code compliant design) and the expected losses of the same buildings designed with the draft LDSD advice.

The previous investigation indicated that the draft LDSD advice could reduce the expected annual loss (EAL) of complying buildings to a target limit of 0.1% of the building replacement cost, with due consideration of the inherent uncertainty involved with probabilistic loss assessments. The draft LDSD advice indicates that to limit losses and achieve low-damage performance objectives, designs can be developed considering a damage control limit state (DCLS) at which, for traditional types of fit-out, the building should undergo no more than 0.5% drift at the DCLS design intensity level. The low drift limit helps to limit damage to structural and non-structural drift-sensitive components (such as plasterboard partition walls and cladding systems). Commercial buildings also contain significant quantities of non-structural components which can be damaged by high seismic accelerations, namely ceilings and services. Increasing building stiffness with the aim of reducing drifts can increase floor accelerations, and it was found that without careful design of acceleration sensitive components, increased damage and losses due to such components could undo the reduction in losses found through reducing building drift. This paper demonstrates the importance of considering acceleration-sensitive components in low-damage seismic design so that the holistic performance objectives of the draft LDSD advice are achieved.

2 OVERVIEW OF CASE STUDY BUILDINGS

The study considers a set of two case-study commercial buildings (4- and 12-storeys), designed for two locations (Wellington and Christchurch) by John et al. 2022. The case study buildings were adapted from a previous QuakeCoRE investigation by Yeow et al. (2018). Storey heights were 3.6 m, except for the first storeys (4.5 m). The IL2 office buildings all used RC wall lateral systems, with the walls varying in length and number depending on the building design. The floor systems were double tees, non-structural component quantities and weights were taken from Yeow et al. (2018) and a 3 kPa live load was assumed.

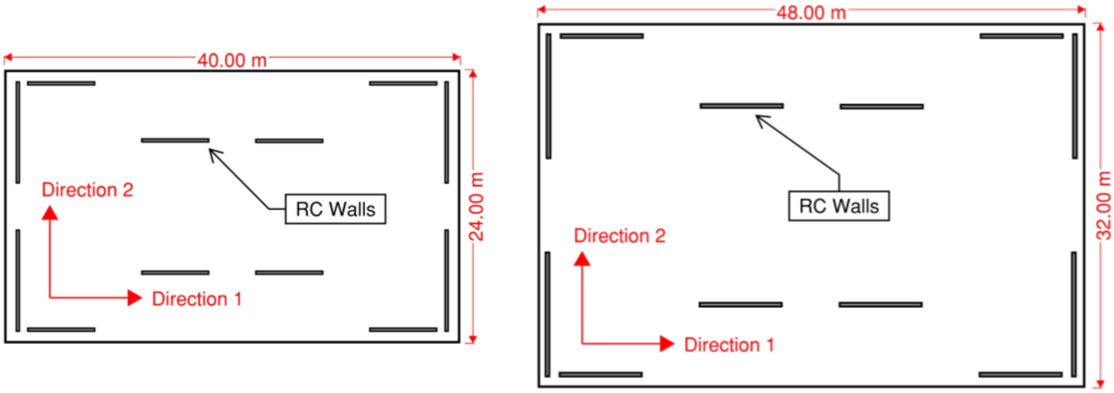

Figure 1: Plan view of case study RC wall buildings (left: 4-storey, right: 12-storey) (number of walls varies according to the design).

2.1 Seismic design of case study buildings

Building design was completed by John et al. 2022 using the equivalent static method in accordance with NZS 1170.5, along with the reinforced concrete seismic design provisions from NZS 3101 (Standards New Zealand 2004; Standards New Zealand 2006). The hazard factor, Z, was 0.3 for Christchurch (site subsoil class D) and 0.4 for Wellington (site subsoil class C). P-delta effects were considered using NZS 1170.5 provisions. For simplicity, wall arrangements were symmetrical but accidental eccentricity, and the corresponding torsional effects, were considered as per NZS 1170.5. Wind loads were considered using AS/NZS 1170.2 (Standards New Zealand 2011) but did not govern the building lateral design even when using the most conservative assumptions.

Two design scenarios were applied for each of the four buildings: a ‘conventional’ design (following current provisions from NZS 1170.5 and NZS 3101) and a ‘low-damage’ design (following the same provisions as the conventional design, but with the additional requirements from the draft LDSD advice). The conventional design included a check for wall yielding at the serviceability limit state (SLS1) and a check for inter-storey drift and wall strength at the specified ductility at the ultimate limit state (ULS). As stated earlier, the additional requirements from the draft LDSD advice introduce a damage control limit state (DCLS) at which the inter-storey drift limit was checked. The drift limits and assumed ductility for the two design scenarios are summarised in Table 1. The ULS design ductility was taken as 4.0 for the conventional design to represent a state-of-practice design, whereas the low-damage design ductility at DCLS and ULS were adopted from the draft LDSD advice. RC wall designs were generally driven by the drift limits from Table 1. Low-damage buildings required longer and more numerous walls due to the stricter drift limit at DCLS. Further discussion into the seismic design of the various buildings can be found in John et al. 2022.

Table 1: Limit states for conventional and low-damage designs.

Design case	Limit state	Drift limit	Ductility	Design intensity return period
Conventional	SLS1	N/A	1.0	25 years
	ULS	2.5%	4.0	500 years
Low-damage	SLS1	N/A	1.0	25 years
	DCLS	0.5%	2.0	250 years
	ULS	2.5%	3.0	500 years

2.2 Non-linear time history analyses

Non-linear time history (NLTH) analyses are used to establish the likely seismic response of the buildings as if they were constructed with the results of the various design solutions. These analyses aim to quantify the peak storey drifts and floor accelerations for a range of intensity levels (return periods), with this data being used for the loss assessment procedure (described later).

The RC walls were modelled with three dimensional lumped-plasticity frame elements in OpenSees (McKenna et al. 2006) for the NLTH analyses. Expected material strengths were as set in line with the recommendations of Priestley et al. (2007). 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 using a Takeda (thin) model, as described by Otani (1981). Post-yield stiffness of the walls was modelled to be 5% of the elastic stiffness. The bilinear stiffness used to approximate the nonlinear behaviour of the RC walls used a cracked section initial stiffness given by Priestley et al. (2007):

$$E_c I_{cr} = \frac{M_N}{\phi_y} \quad (1)$$

where M_N = section moment capacity (based on expected material strengths); and ϕ_y = nominal yield curvature of the rectangular RC wall (defined using approximate expressions in Priestley et al. 2007). Floors were modelled to act as rigid diaphragms in-plane, fully flexible out-of-plane. P-delta effects were incorporated into the model using dummy columns (co-rotational truss elements) constrained to move laterally together with the central nodes of the structure. An elastic rotational spring at the base of each wall accounted for foundation deformations. This spring stiffness was defined such that the spring would rotate 0.1% when the wall yielded. The first and fourth modes of vibration were damped at 3% using a Rayleigh damping model.

The NLTH analyses used 20 pairs of ground motion records at 11 intensity levels, selected in a previous QuakeCoRE study by Yeow et al. (2018) to be hazard consistent with the locations considered. 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 seismic hazard for New Zealand is currently being re-evaluated and hence, the hazard estimates and 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. The largest integration time step used was 0.02 s, which was deemed appropriate after considering the results of a sensitivity analysis.

Buildings located in Wellington typically experienced higher drifts and floor accelerations at higher intensity levels than those in Christchurch. Low-damage buildings were stiffer and drifted significantly less than their conventional counterparts. However, the stiffer low-damage buildings also generally experienced higher floor accelerations. Figure 2 shows NLTH results for the 4-storey buildings, which are generally representative of the major trends observed across all buildings.

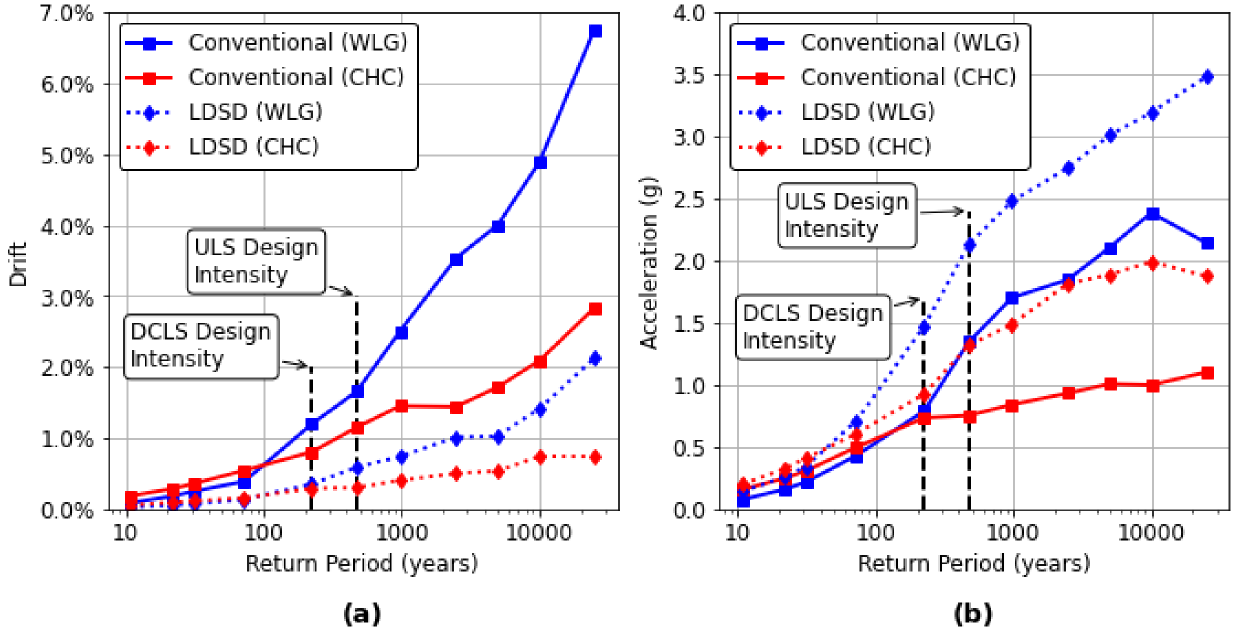

Figure 2: Median values of (a) peak top storey drifts and (b) peak roof accelerations across the eleven intensity levels, recorded from time history analyses of the 4-storey buildings.

3 LOSS ASSESSMENT

Loss assessments were completed with the Performance Assessment Calculation Tool (PACT) software (FEMA 2018b) in accordance with the performance-based earthquake engineering method developed by the Pacific Earthquake Engineering Research centre and outlined in FEMA P-58 (FEMA 2018a). Losses are estimated from the following inputs:

- Peak inter-storey drifts and peak floor accelerations (engineering design parameters) from the NLTH analyses (using the 20 pairs of ground motions over the eleven intensity levels),
- The hazard curve for the specified intensities (probabilities of occurrence of intensities), and
- A building inventory of damageable components, with associated fragility and consequence functions.

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 (structural and non-structural items) was adopted from Yeow et al. (2018) and updated to reflect the structural system used in this study.

The investigation of John et al. 2022 included loss assessments for the four buildings, when designed conventionally and when designed following the draft LDSD advice. While all LDSD buildings had lower drift-sensitive losses compared to conventional buildings, three of the four LDSD buildings showed higher overall losses due to increased acceleration-sensitive losses (in terms of expected annual loss, or EAL) (see Figure 4 and Figure 5). Low-damage buildings experienced higher floor accelerations due to being stiffer, which increased acceleration-sensitive losses when compared to the corresponding conventionally designed building. It was concluded that following the draft advice should give the expected reduction in losses from drift-sensitive components, however, acceleration-sensitive losses should also be reduced to ensure overall losses are meeting low-damage expectations. Consequently, this paper investigates how a reduction in acceleration-sensitive losses for LDSD buildings could be achieved.

3.1 Acceleration-sensitive components

Of the acceleration-sensitive losses, ceilings were identified as the predominant contributor for LDSD buildings (Figure 3). Ceiling fragility functions have been relatively well-investigated by other researchers. For these reasons, ceilings were identified as the best candidate for testing how acceleration-sensitive losses in LDSD buildings could be reduced. In New Zealand, ceilings are typically designed for strength at SLS1-level (1 in 25 years) seismic events (Dhakal et al. 2016) for buildings below IL4. This means ceilings have the potential to be highly vulnerable, which reflects the large losses attributed to ceilings in the original investigation. It is expected that by designing ceilings to the DCLS design intensity level (taken as 1 in 250 years), the corresponding losses should be well limited even for low-damage buildings despite the potential amplification of floor accelerations from stiffer structural response. This approach is tested in this study by using the results from the original NLTH analyses to complete new loss assessments for the LDSD buildings in which ceilings with lower fragility (i.e., those designed to a DCLS-level event) are specified.

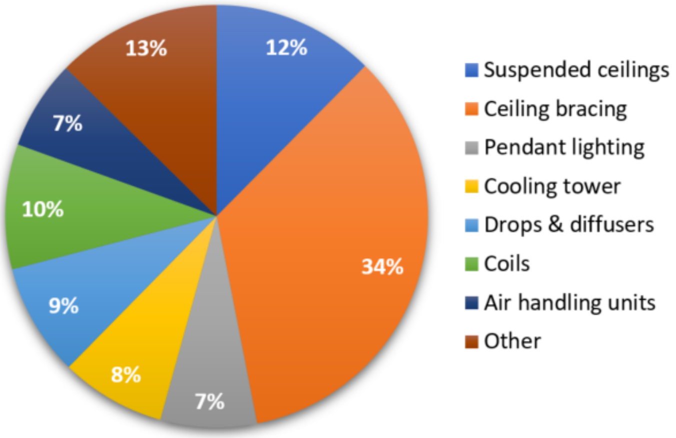

Figure 3: Contributions to the expected annual losses of acceleration-sensitive components (for the 4-storey LDSD building in Christchurch).

A similar approach could be suggested for reducing the losses of other acceleration-sensitive components that are designed for events less severe than at the damage control limit state. It should be noted that the losses from ceilings will not represent those of all acceleration-sensitive components (particularly components already designed for ULS-level events) but may offer useful insight into how acceleration-sensitive losses could be reduced in LDSD buildings and if the proposed approach is likely to be effective.

3.1.1 Derivation of current ceiling fragilities

The ceiling fragilities used in the original loss assessments by the authors were adapted from the Yeow et al. 2018/QuakeCoRE investigation. The fragilities represent conventionally designed ceilings (i.e., to SLS). The QuakeCoRE fragilities for ceilings were derived from testing at the University of Canterbury, using the testing data and methodology from Dhakal et al. 2016. The testing of ceilings at UC found the forces at which various ceiling systems would be expected to reach certain damage states, which was divided by the ceiling tributary masses to obtain the lateral accelerations required to reach the damage states. These lateral accelerations are peak ceiling grid accelerations (PCGA) at which failure occurs. For use in the loss assessments, the fragilities must be defined in terms of peak floor acceleration (PFA). Yeow et al. 2018 achieved this conversion by a statistical analysis from existing data relating PCGA to PFA. It is acknowledged that the conversion from PCGA to PFA may not be exact because the PCGA depends on the spectral response of the ceiling system to the PFA, which will depend on the dynamic characteristics of the ceiling system (likely to vary from ceiling to ceiling). However, as a means of capturing the expected/average response of the ceilings, the authors deemed this approach acceptable given the scope of this study.

3.1.2 Adjustment of ceiling fragilities

It was assumed that the ceiling bracing would be designed considering the loads defined in the parts and components/section 8 approach from NZS 1170.5 (Standards NZ 2004). Therefore, capacity of the ceilings can be assumed to be proportional to the design acceleration from the parts and components loading methodology. Consequently, the fragilities were adjusted for location by the ratio of the hazard factors (Z) for the two locations (0.4 for Wellington and 0.3 for Christchurch). To obtain representative fragilities of ceilings that were designed for DCLS rather than SLS, the fragilities were increased by the ratio of the return period factors (R) between SLS1 and DCLS (0.25 for a 1-in-25 year SLS1 event and 0.75 for a 1-in-250 year DCLS event). The fragilities and dispersions adopted in this study for the different ceiling groups within the case-study building are shown in Table 2.

As the fragilities inherently capture the material expected strengths (as they were derived from testing), the fragilities need not be adjusted for any difference between design and expected strength. This approach for adjusting the ceiling fragilities was deemed appropriate to obtain fragilities that could illustrate the effect of designing ceilings to higher accelerations in a low-damage design on building losses in the context of this study, however, it is suggested that further investigation of ceiling fragilities would be required for more rigorous loss assessments.

Table 2: Fragilities of various ceilings in the case study building, derived for the two cases where ceilings designed to SLS or DCLS.

Room	Median PFA (g)				Dispersion
	CHC, SLS	WLG, SLS	CHC, DCLS	WLG, DCLS	
Staff room, 8 m^2 meeting room and 5 m^2 quiet room combined	0.46	0.61	1.38	1.84	0.4
32 m^2 meeting room	0.70	0.93	2.10	2.80	0.4
Other enclosed rooms	0.72	0.96	2.16	2.88	0.4
Braced ceilings	0.72	0.96	2.16	2.88	0.4

3.2 Loss assessment results

Key loss results are shown in Figure 4 (Christchurch) and Figure 5 (Wellington). It is apparent that LDSD buildings with ceilings designed to DCLS appear to incur significantly lower losses compared to LDSD buildings with ceilings designed to SLS (a reduction of 85-90% in ceiling losses and 25-40% in overall losses was found). All four LDSD building designs with ceilings designed to DCLS returned lower overall losses compared to conventional buildings, while only one of the LDSD buildings with ceilings designed to SLS1 achieved this (and only marginally). This demonstrates the effectiveness of increasing the design strength requirements of ceiling systems (and acceleration-sensitive components in general) on the reduction of overall building losses and enables the LDSD buildings to achieve an EAL, on average, of 0.1% or less, as targeted by the draft LDSD advice.

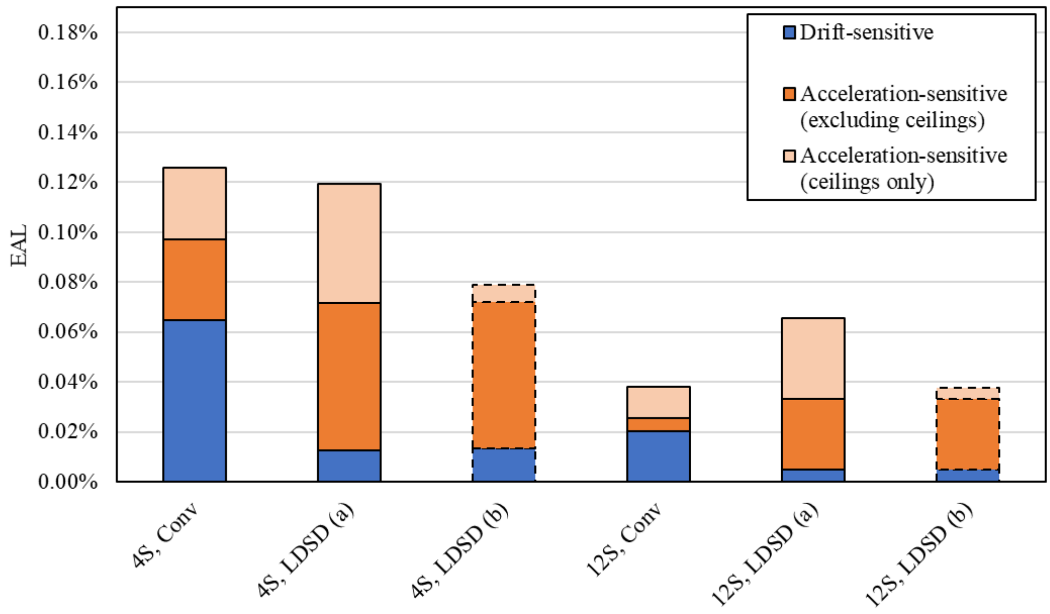

Figure 4: Expected annual losses for conventional and low-damage designs, Christchurch. ("Conv" refers to a conventional design, "LDSD (a)" is low-damage design without a specific ceiling design, and "LDSD (b)" is low-damage design with the DCLS ceiling design).

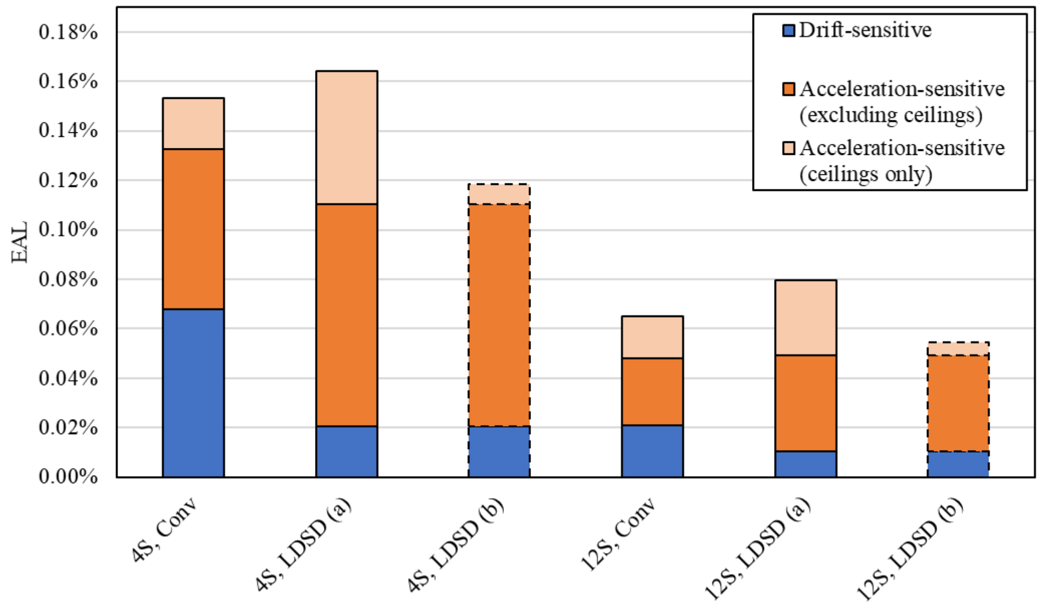

Figure 5: Expected annual losses for conventional and low-damage designs, Wellington. ("Conv" refers to a conventional design, "LDSD (a)" is low-damage design without a specific ceiling design, and "LDSD (b)" is low-damage design with the DCLS ceiling design).

Figure 6 shows a breakdown of the acceleration-sensitive losses for a LDSD building with and without ceilings designed to DCLS. Little variation to these results was found between the different buildings. When ceilings were designed to SLS1, they were found to be the predominant contribution to the total acceleration-sensitive losses. However, when designed to DCLS the loss contribution from the ceiling systems reduces to a level where is it appears to be proportional to a variety of other acceleration-sensitive components (including drops & diffusers, coils, cooling tower, air handling units, and pendant lighting). This suggests that DCLS could be a sufficient limit state to which ceilings can be designed to reduce the expected annual losses of LDSD buildings to 0.1% or less, on average. Further investigation is recommended to determine if designing ceilings to DCLS is practicable and if there is a lower return period for ceiling design that could still yield acceptable losses.

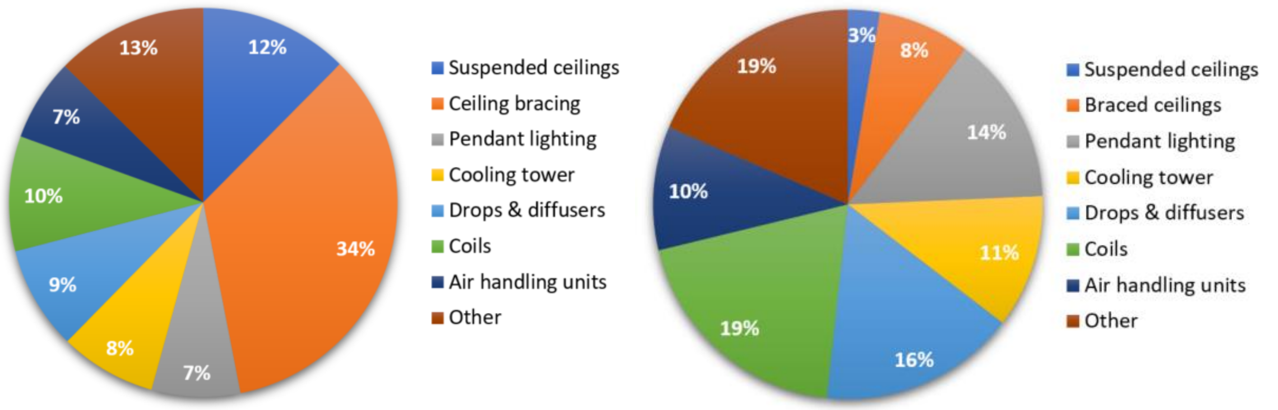

Figure 6: Contributions to the expected annual losses of acceleration-sensitive components (for the 4-storey LDSD building in Christchurch without (left) and with (right) the DCLS ceiling design limit state).

4 CONCLUSIONS

This study has demonstrated that a significant reduction in monetary losses (from likely post-earthquake repair costs) can be achieved for LDSD buildings by considering the acceleration demands at the DCLS intensity level for the design of restraints of ceilings. This reduction was shown to bring overall losses down to the level intended by the draft LDSD advice for RC wall buildings, negating the effect of high acceleration-sensitive losses from increased floor accelerations which resulted as a side effect of applying a 0.5% design drift limit for a DCLS design intensity level. More generally, the paper highlights the importance of carefully designing acceleration-sensitive non-structural elements and the need for a holistic approach to low-damage design. The relationship between drift limits, building stiffness and floor accelerations must be considered in a low-damage building design, and preventative measures should be taken to protect acceleration-sensitive components, along with drift-sensitive components.

5 ACKNOWLEDGEMENTS

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REFERENCES

Dhakal, R., Macrae, G., Pourali, A., and Paganotti, G. 2016. Seismic fragility of suspended ceiling systems used in NZ based on component tests, *Bulletin of the New Zealand Society for Earthquake Engineering*, Vol 49 45-63. DOI: 10.5459/bnzsee.49.1.45-63

FEMA. 2018a. *Seismic Performance Assessment of Buildings Volume 1 - Methodology*. Washington, D.C.: Federal Emergency Management Agency. ⟨https://www.fema.gov/media-library-data/1557508320921-d67f745e88e04e54a1f40f8e94835042/FEMA_P-58-1-SE_Volume1_Methodology.pdf?fbclid=IwAR3RDQhKHPGdDWJxEEd4_7YEqY0LdXddTCRH_-YSOZDM1QNl_LylBw8g12Q⟩

FEMA. 2018b. Appendix C: PACT User Manual, *Seismic Performance Assessment of Buildings Volume 2 - Implementation Guide*. Washington, D.C.: Federal Emergency Management Agency. ⟨https://femap58.atcouncil.org/documents/fema-p-58/25-fema-p-58-volume-2-implementation-second-edition/file⟩

John, L., Williamson, M.B., Sullivan, T.J. 2022. Investigating the impact of design criteria on the expected seismic losses of multi-storey office buildings, *Bulletin of the New Zealand Society for Earthquake Engineering*, under review.

McKenna, F., Fenves, G.L., Scott, M.H., and Mazzoni, S. 2006. *OpenSees Command Language Manual*. Berkeley, USA: University of California, Berkeley. ⟨https://opensees.berkeley.edu/OpenSees/manuals/usermanual/OpenSeesCommandLanguageManualJune2006.pdf⟩

Otani, S. 1981. Hysteretic Models for Reinforced Concrete for Earthquake Analysis, *Facility of Engineering*, Vol XXXVI (2): 125-156. DOI:

Priestley, M.J.N., Calvi, G.M., and Kowalsky, M.J. 2007. *Displacement-Based Seismic Design of Structures*. Pavia, Italy: IUSS Press.

Standards New Zealand. 2004. *NZS 1170.5:2004. Structural design actions: Part 5: Earthquake actions*. Wellington: Standards New Zealand.

Standards New Zealand. 2006. *NZS 3101.1:2006. Concrete structures standard*. Wellington: Standards New Zealand.

Standards New Zealand. 2011. *NZS 1170.2:2011. Structural design actions: Part 2: Wind actions*. Wellington: Standards New Zealand.

Yeow, T.Z., Orumiyehei, A., Sullivan, T.J., MacRae, G.A., Clifton, G.C., and Elwood, K.J. 2018. Seismic performance of steel friction connections considering direct-repair costs, *Bulletin of Earthquake Engineering*, Vol 16 (12): 5963-5993. DOI: 10.1007/s10518-018-0421-x