# Investigating the impact of design criteria on the expected seismic losses of an office building

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## ABSTRACT

Low Damage Seismic Design (LDSD) guidance material being developed by Engineering NZ is considering a design drift limit for multi-storey buildings of 0.5% at a new damage control limit state (DCLS). The impact of this new design requirement on the expected annual loss due to repair costs is investigated for a four-storey office building with reinforced concrete walls located in Christchurch. The LDSD guidance material aims to reduce the expected annual loss of complying buildings to below 0.1% of building replacement cost. The research tested this expectation. Losses were estimated in accordance with FEMA P-58, using building responses from non-linear time history analyses (performed with OpenSees using lumped plasticity models). The equivalent static method, in line with NZS 1170.5 and NZS 3101, was used to design the building to LDSD specifications, representing a future state-of-practice design. The building designed to low-damage specification returned an expected annual loss of 0.10%, and the building designed conventionally returned an expected annual loss of 0.13%. Limitations with the NZS 3101 method for determining wall stiffness were identified, and a different method acknowledging the relationship between strength and stiffness was used to redesign the building. Along with improving this design assumption, the study finds that LDSD design criteria could be an effective way of limiting damage and losses.

## 1 INTRODUCTION

Assessments following the Christchurch earthquake sequence found that modern capacity-designed buildings generally met structural performance expectations, but repair costs, associated with damage to non-structural components, were unacceptably high (Dhakal 2010). Work is currently on-going to identify new design criteria that better limit seismically-induced building loss. As part of this, Engineering NZ is proposing Low Damage Seismic Design (LDSD) guidance material that aims to limit loss by tightening design criteria (primarily drift limits). This research will investigate whether the LDSD proposals are likely to be effective in reducing the expected losses for a reinforced concrete wall building.

The specific goals and objectives of the research are to:

- Verify the following proposed objectives of the LDSD guidance material;

1. LDSD buildings achieve expected annual losses of 0.1% of the building replacement cost or less (with 50% confidence level).
2. For a LDSD building, there is less than a 1/25 chance that an earthquake incurs a repair cost greater than 5% of the building replacement cost over a 10 year period.

- Investigate how effective the LDSD criteria are at reducing losses, compared to current NZ design standards, for a RC wall building; and
- Investigate how the proposed new LDSD guidance material will affect the design of RC wall buildings, and identify potential issues with the application of the guidance material by structural designers.

The research considers a four storey commercial office case-study building with RC wall lateral systems (located in Christchurch), previously investigated by Yeow et al. (2018) and shown in Figure 1. The building footprint is 24 m by 40 m, with 8 m bays in both directions. Storey heights are 4.5 m for the first storey and 3.6 m for those above. The building uses primary beams along gridlines in the short elevation, with RC double tees specified as the floor system. Non-structural component weights are taken from Yeow et al. (2018). A 3 kPa live load in accordance with AS/NZS 1170.1 is included in the seismic weight for floors.

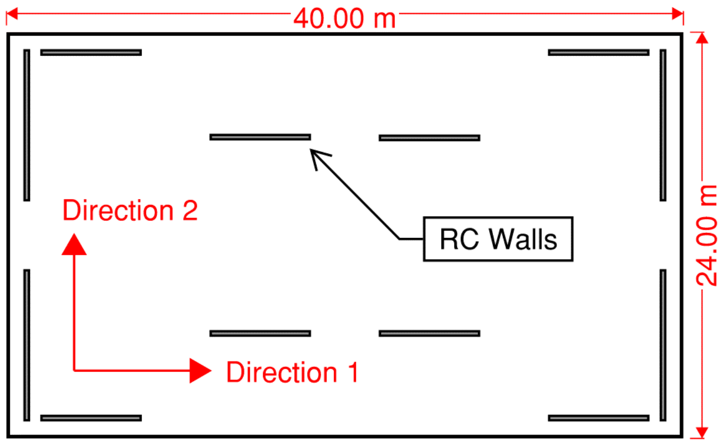


*Figure 1: Plan view of case study RC wall building (number of walls varies according to the design).*

Various symmetrical arrangements of RC walls were designed and investigated. Designs are completed in accordance with NZ standards and the draft LDSD guidance material. Non-linear time history analyses are conducted in OpenSees and are used to predict building responses to ground motions at a range of intensity levels. These responses are then used to inform loss assessments for the various building designs. Loss assessment is 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.

## 2 SEISMIC DESIGN AND TIME HISTORY ANALYSES

### 2.1 Seismic design of the building

In order 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 NZS 1170.5 equivalent static method and NZS 3101 reinforced concrete seismic design provisions (Standards New Zealand 2004; Standards New Zealand 2006). Periods used for design were calculated by the Rayleigh method and are reported later in the results section of this paper. P-delta effects from shear and flexural deformations were considered in the design, along with accidental eccentricities. All designs had symmetrical wall arrangements to avoid any torsional effects. The criteria used for the two design scenarios consisted of strength and drift limit checks. LDSD included a damage control limit state (DCLS) where storey drift limits were checked, and an ultimate limit state (ULS) where strength requirements were satisfied. These limit states used ductility factors of 2.0 and 3.0 respectively. A ductility factor of 4.0 was used for the conventional design at ULS. Table 1 outlines the drift limits to which the two design scenarios adhered. Walls were designed with concrete having a characteristic compressive strength of 40 MPa and reinforcement having a characteristic yield strength of 500 MPa.

*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 |

## 2.2 Non-linear time history analysis

The walls were modelled with three dimensional lumped-plasticity frame elements in OpenSees (McKenna et al. 2006) for the time history analyses. Expected material strengths were used as per the recommendations of Priestley et al. (2007) (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 (see Otani (1981)).

The bilinear stiffness used to approximate the nonlinear behaviour of the RC walls in loading used a cracked section stiffness ($E_c I_{cr}$) as the initial stiffness, derived from the section strength (determined using expected material strengths), $M_N$, and yield curvature of the wall, $\phi_y$ (Priestley et al. 2007).

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

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

where $E_c$ = elastic modulus of concrete (determined in accordance with NZS 3101); $\varepsilon_y$ = 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 (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 assumed to act as rigid diaphragms in plane, fully flexible out of plane. The seismic mass of each floor was distributed to four nodes (placed symmetrically) on each floor. P-delta effects were incorporated into the model using dummy columns. These were corotational 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 based on experience from industry. This approach accepts 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 research.

A Rayleigh damping model was used, with 3% damping on the 1st and 2nd modes. Constants of proportionality were determined from the periods of the first and second modes. The sensitivity of building response to changes in the damping model (through changes to the periods, damping ratios, and mass matrix constant of proportionality) was tested, with sensitivity to the investigated inputs deemed acceptable.

Models were initially tested with static pushover analyses to confirm their force-displacement behaviour. The time history analyses used 180 bidirectional ground motion records selected by Yeow et al. (2018) to determine building responses for loss assessment. The ground motions (GMs) were divided into nine intensity levels, with each GM scaled to a specific spectral acceleration (at a conditioning period of 1.0 s) corresponding to the intensity level (Table 2). Intensity level 3 was the design intensity for DCLS, and intensity level 4 was the design intensity for ULS. The largest time steps 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.

*Table 2: Ground motion intensity levels with their corresponding return periods and spectral accelerations at the conditioning period of 1.0 s.*

| **Intensity Level** | **1** | **2** | **3** | **4** | **5** | **6** | **7** | **8** | **9** |
|---|---|---|---|---|---|---|---|---|---|
| **Return period (years)** | 63.5 | 100 | 250 | 500 | 1000 | 2500 | 5000 | 10,000 | 25,000 |
| **Spectral acceleration (g)** | 0.312 | 0.200 | 0.315 | 0.412 | 0.522 | 0.691 | 0.834 | 0.990 | 1.231 |

## 2.3 Results: Seismic design

Limitations with the NZS 3101 approach to determine the effective second moment of area ($I_e$) were identified during design. The approach (from Table C6.5 of the NZS 3101 commentary) only accounts for the geometric properties of the wall section, meaning a designer can only increase the wall thickness or length in order to increase wall stiffness. For LDSD, drift limits govern, and so a designer would aim to increase wall stiffness. To increase stiffness, NZS 3101 encourages the design of longer walls (most efficient way to increase area moment of inertia). This will likely be the method by which designers achieve the lower drift limits for LDSD when designing with RC walls.

The approach described above appears to ignore the effect of reinforcement on wall stiffness. Equation (1) demonstrates the relationship between section strength and stiffness. 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 hence properly consider the reinforcement in the section. Therefore, it was proposed that equation (1) should be used instead of the NZS 3101 approach for determining wall stiffness.

Consequently, four designs for the building were produced (Table 3 and Table 4), two for each design scenario (conventional and low-damage). 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)). 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 the ability to utilise reinforcement to stiffen walls, rather than being limited by modifying wall section geometry as encouraged by NZS 3101.

*Table 3: Wall designs for the four cases, in direction one.*

| Design case | Wall arrangement | Reinforcement ratio | Design period | Period of OpenSees model |
|---|---|---|---|---|
| Conventional | Ten 3 m walls | 0.601% | 0.87 s | 1.19 s |
| Low-damage | Eight 6 m walls | 0.359% | 0.37 s | 0.62 s |
| Conventional (Priestley stiffness) | Twelve 3 m walls | 1.84% | 0.78 s | 0.76 s |
| Low-damage (Priestley stiffness) | Six 6 m walls | 2.06% | 0.38 s | 0.39 s |

*Table 4: Wall designs for the four cases, in direction two.*

| Design case | Wall arrangement | Reinforcement ratio | Design period | Period of OpenSees model |
|---|---|---|---|---|
| Conventional | Two 6 m walls | 1.091% | 0.79 s | 0.87 s |
| Low-damage | Four 9 m walls | 0.473% | 0.40 s | 0.59 s |
| Conventional (Priestley stiffness) | Two 6 m walls | 1.92% | 0.83 s | 0.82 s |
| Low-damage (Priestley stiffness) | Two 9 m walls | 2.09% | 0.41 s | 0.43 s |

All wall designs were governed by the drift limits from Table 1. The designs produced with NZS 3101 appeared to give more walls of lower reinforcement ratio to achieve the required stiffness, compared to designs produced using Priestley's stiffness which allowed the designer to choose fewer walls of higher reinforcement quantity. 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.

Tables 3-4 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 designs assuming Priestley's stiffness are within 5% of the model period (as they use the same stiffness assumption). The design periods for those assuming the NZS 3101 stiffness were all lower than the model periods. The NZS 3101 stiffness overestimated the wall stiffness. 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%.

## 2.4 Results: Non-linear time history analysis

The results from the time history analyses of the two building designs (conventional and LDSD) were best summarised by the peak accelerations and drifts at the fourth floor of the building shown in Figure 2.The plots show the medians of the peak building responses for the 20 ground motions at each intensity.

Figure 2 shows that both conventional models drifted significantly less than the 2.5% limit in direction one at the ULS design intensity level. This is likely due to conservatism in the NZS code design methods, notably the consideration of P-delta effects and allowances for accidental eccentricity. The building designed with the Priestley stiffness assumption drifted less than the building designed as per NZS 3101. This is attributed to the NZS 3101 approach overestimating the stiffness of walls with lower reinforcement ratios.

Figure 2 shows that the LDSD models drifted significantly less than the conventional models in direction one as expected. The LDSD model designed with the NZS 3101 stiffness assumption drifted approximately 0.5% at the DCLS design level. The LDSD model designed with the Priestley stiffness assumption again drifted less than the model designed as per NZS 3101, likely due to the same reason as before. If it is assumed that the NZS code design methods should be conservative, and lead to the building models drifting significantly less than the drift limit (as with the conventional building models), the NZS 3101 stiffness approach is inadequate. Therefore, it is recommended that the Priestley stiffness approach is used for LDSD.

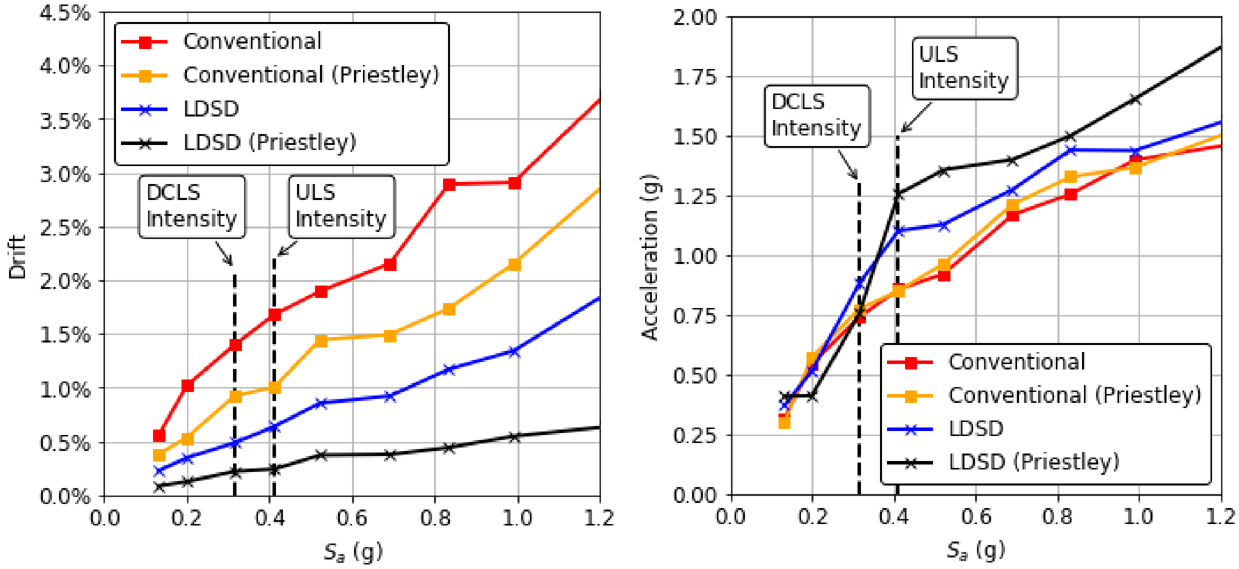


*Figure 2: Median values of the peak fourth storey drifts in direction one (left) and peak roof accelerations in direction two (right) across the nine intensity levels, recorded from time history analyses.*

The roof accelerations shown in Figure 2 suggested that the stiffer buildings experienced higher accelerations. The stiffer LDSD models experienced higher accelerations than the conventional models, and the models using Priestley's stiffness also experienced higher accelerations than their 'NZS compliant' counterparts. It should be noted that the drifts of the four models in direction two demonstrated similar behaviour to that of direction one, and the accelerations of the four models in direction 1 demonstrated similar behaviour to that of direction two.

Some may consider the intensity-based comparison of engineering design parameters for different buildings in Figure 2 to be inappropriate, given that ground motions were selected with reference to a conditioning period and all the buildings have different periods. 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, Figure 2 does 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.

## 3 LOSS ASSESSMENT

Loss assessments were completed using Performance Assessment Calculation Tool (PACT) software (FEMA 2018a) in accordance with the performance-based earthquake engineering method developed by the Pacific Earthquake Engineering Research centre and outlined in FEMA P-58 (FEMA 2018b). The governing equation behind the loss estimation was:

$$EAL = \iiint P(EDP|IM) \cdot P(DS|EDP) \cdot P(C|DS) \cdot d(IM) \tag{4}$$

where EAL = expected annual loss due to building repair or replacement; P(EDP|IM) = probability of obtaining engineering design parameters (peak drifts or accelerations) at a given intensity measure (IM); P(DS|EDP) = probability of building components reaching damage states (certain levels of damage) at a given engineering design parameter; P(C|DS) = probability of incurring costs at given damage states; and d(IM) = annual rates of exceeding a given value of IM (Moehle and Deierlein 2004). The software estimated losses using equation (4), with the independent variables/inputs being:

- Peak inter-storey drifts and floor accelerations from the non-linear time history analyses using the 180 ground motions, over the nine hazard levels/intensity measures;
- A hazard curve for the specified intensities (from Yeow et al. (2018)); and
- A building inventory of damageable components, with their fragility and consequence functions.

Fragility functions specify the peak drift or acceleration at which a damage state was reached. Consequence functions describe the expected repair cost associated with reaching a given damage state. The case-study building inventory (and fragility and consequence functions) from Yeow et al. (2018) 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, partition walls (full and partial height), stairs, suspended ceilings, ceiling bracings, lighting, a traction elevator, water pipe distribution, sanitary pipe distribution, fire sprinklers, HVAC components, and transformers (100 to 350 kVA).

Available loss information for RC walls did not recognise the effect of wall length on their fragility functions. Damage states for RC walls are 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 lengths of wall 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 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 5).

*Table 5: Damage states (DS) assumed for RC walls in this study.*

| DS | Damage description | Strain |
|---|---|---|
| 1 | Approximately 10 m of visible cracking* (less than 0.5 mm width) | 0.005 (steel in tension) (twice the yield strain) |

| | | |
|---|---|---|
| 2 | Approximately 50 m of visible cracking* (greater than 0.5 mm width) with minor spalling | 0.004 (concrete in compression) |
| 3 | Approximately 100 m of visible cracking* (greater than 0.5 mm width) with major spalling (exposed rebars) | 0.006 (concrete in compression) |
| 4 | Extensive cracking, spalling, bar buckling and/or concrete crushing | 0.06 (steel in tension) (Priestley and Kowalsky 1998) |

*It is noted that the length of cracking for the different damage states is independent of the wall length.

These 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), the following process was utilised to determine the drifts triggering each damage state:

1. Use moment-curvature analysis to convert the damage state strain to a section curvature, $\phi$, at the base of the wall.
2. Determine the wall displacement profile using a mechanics-based relationship between wall curvature and storey displacements, similar to direct displacement-based design methods from Sullivan et al. (2012).
3. Convert the wall displacement profile into inter-storey drifts for the fragility function.

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), as estimated by a UC-led investigation in collaboration with Beca and Naylor Love.

A building value of $9,250,000 NZD (2020, inflation adjusted) was assumed in accordance with the research by Yeow et al. (2018). 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 (FEMA 2018b).

## 3.1 Results and discussion

Loss assessments were completed for the four building designs. The key results from the two designs produced using the NZS 3101 stiffness approach (the 'code-compliant' designs) are shown in Table 6.

*Table 6: Loss results for conventional and low-damage designs using the NZS 3101 stiffness method.*

| | **LDSD** | **Conventional** |
|---|---|---|
| Total Expected Annual Loss* | 0.10% | 0.13% |
| Expected Annual Loss from drift-sensitive components | 0.02% | 0.07% |
| Chance of a repair cost greater than 5% of building replacement cost over 10 years** | 1/27 | 1/20 |

*Should be 0.1% according to draft LDSD guidance material
**Should be a 1/25 chance, or less likely, according to draft LDSD guidance material

LDSD results shown in Table 6 are close to the performance objectives stated by the guidance material, so the research suggests that the low-damage design criteria can lead to the desired loss performance. From non-linear time history analysis (Figure 2), the LDSD building was shown to drift close to the 0.5% drift limit at the design intensity. Thus, the results for this design can provide a reference for the losses that could be expected if the building responds to the limit of the proposed LDSD criteria.

The results show a similar total EAL for the low-damage design and the conventional design (0.10% compared to 0.13%). However, the LDSD building gave an EAL of 0.02% for drift-sensitive components, compared to 0.07% for the conventional design. The conventional building drifted much less than the 2.5% limit in the non-linear time history analyses (Fig. 2), so a building that drifts closer to the 2.5% limit would be expected to have much higher losses. Therefore, there is likely to be a significant reduction in drift-sensitive damage from conventionally designed buildings to LDSD buildings.

Key loss assessment results for the building designs using Priestley's stiffness method are shown in Table 7. These again support the observations in the relative performance between conventionally designed and low-damage designed buildings. However, a lower overall magnitude of losses was estimated for buildings designed using Priestley's stiffness compared to those designed with the NZS 3101 method. This was because the buildings designed with Priestley's stiffness drifted less (Fig. 2), as the design stiffness was lower than the NZS 3101 design stiffness (which overestimated the wall stiffness).

*Table 7: Loss results for conventional and low-damage design using Priestley's stiffness method.*

| | **LDSD** | **Conventional** |
|---|---|---|
| Total Expected Annual Loss | 0.08% | 0.10% |
| Expected Annual Loss from drift-sensitive components | 0.01% | 0.04% |
| Chance of a repair cost greater than 5% of building replacement cost over 10 years | 1/33 | 1/26 |

A summary of the total expected annual losses for all four designs is shown in Figure 3. Notably, acceleration-sensitive losses appear to form the majority of the EAL for low-damage designs. Whilst the building drift and its associated losses decrease with a lower drift limit, corresponding floor accelerations and their associated losses appear to increase. This highlights the limitation of limiting drift in a low damage design. A decrease in building drift through stiffening the lateral system likely causes increases in floor accelerations and a significant corresponding cost to the repair of acceleration-sensitive components. Limiting damage to these components, in addition to setting lower drift limits, could further decrease overall losses to a building. This may be achieved by strengthening acceleration-sensitive components. There are significant uncertainties in loss estimation and the values reported in this work could be assumed to correspond to a 50% confidence level. Whilst the actual values may vary therefore, the study has shown that low-damage seismic design criteria could be an effective way of limiting damage and losses.

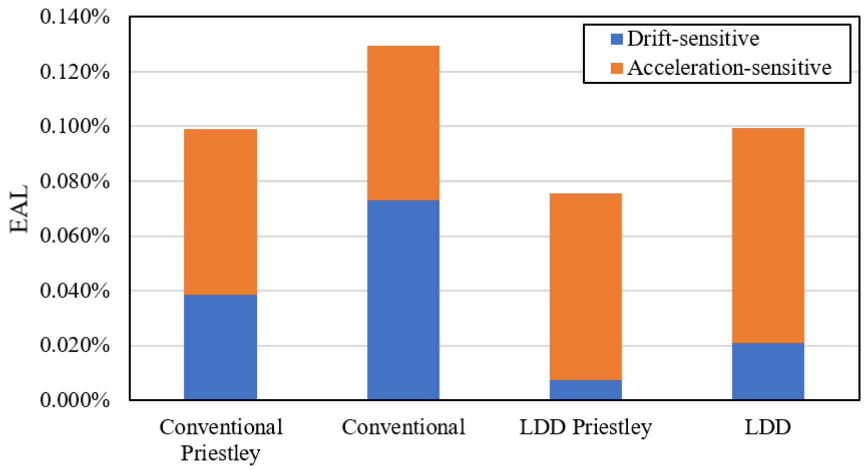


*Figure 3: Summary of expected annual losses for the four designs, showing contributions from drift and acceleration sensitive components.*

## 4 CONCLUSIONS

The effectiveness of draft LDSD guidance material for reducing losses for the RC wall building was investigated by design and assessment of a four storey RC wall building. An expected annual loss of 0.10% for an LDSD-complying building was estimated. The chance that a repair cost exceeding 5% of the building replacement cost is incurred was 1/27 (less than 1/25) over a 10-year period for the LDSD-complying building. These results supported the performance objectives of the guidance material. The research also indicates that the low-damage design criteria would lead to a notable reduction in expected losses, particularly those due to drift-sensitive components. Acceleration-sensitive components were found to contribute significantly to the total EAL for the RC wall LDSD buildings considered in this work.

In light of the limitations of the NZS 3101 approach to RC wall stiffness, Priestley's method of determining stiffness (which recognised the relationship between strength and stiffness) was investigated as an alternative for design. The walls designed using this alternative approach had higher stiffness and higher reinforcement ratios. It appeared that the NZS 3101 approach overestimated wall stiffness for walls with lower reinforcement ratios. Consequently, in the interest of conservative design in which drift limits govern, Priestley's stiffness is recommended for the design of low-damage buildings.

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## REFERENCES

Dhakal, R.P. 2010. Damage to non-structural components and contents in 2010 Darfield Earthquake, *Bulletin of the New Zealand Society for Earthquake Engineering,* Vol 43 (4): 404-411.

FEMA. 2018a. Appendix C: PACT User Manual, *Seismic Performance Assessment of Buildings Volume 2 - Implementation Guide.* Washington, D.C.: Federal Emergency Management Agency.

FEMA. 2018b. *Seismic Performance Assessment of Buildings Volume 1 - Methodology*. Washington, D.C.: Federal Emergency Management Agency.

McKenna, F., Fenves, G.L., Scott, M.H., and Mazzoni, S. 2006. *OpenSees Command Language Manual*. Berkeley, USA: University of California, Berkeley.

Moehle, J., and Deierlein, G. 2004. A framework methodology for performance-based earthquake engineering. *Proc., 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, 01/01.

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

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

Priestley, M.J.N., and Kowalsky, M. 1998. Aspects of drift and ductility capacity of rectangular structural walls, *Bulletin of the New Zealand National Society for Earthquake Engineering,* Vol 31 (2): 73-85.

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.

Sullivan, T.J., Priestley, M.J.N., and Calvi, G.M. 2012. *A Model Code for the Displacement-Based Seismic Design of Structures (DBD12)*. Pavia, Italy: IUSS Press.

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.