THE SERVICEABILITY OF RESILIENT SEISMIC DESIGN IN NEW ZEALAND

Didier Pettinga
Holmes Consulting LP
Christchurch, New Zealand

Abstract

As with many seismic design codes around the world, the application of New Zealand seismic provisions has developed a focus on designing to Life-Safety, otherwise known as Ultimate Limit State (ULS) design. Typical New Zealand design practice sees the ULS lateral design completed first, with drift checks under applied Serviceability Limit State (SLS1) demands following. Recent New Zealand experience following the February 22nd 2011 Christchurch has demonstrated that the primary lateral system ULS performance is satisfactorily met by modern design, yet other smaller earthquake events have left some uncertainty as whether we are achieving consistently appropriate serviceability performance.

Japanese Building Standard Law and the California Performance-Based Seismic Design approaches dictate quite different applications of a two-tiered limit-state design, with serviceability design preceding Life-Safety or Collapse-Prevention checks respectively. Based on observations following recent earthquakes in Japan, there is evidence that this approach produces a more resilient building-stock that not only meets serviceability criteria, but also drives better performance under larger levels of ground motion.

A revision of the New Zealand SLS1 hazard definition, or conversely more rigorous or stringent allowances for applying reductions to the ULS design spectrum, would arguably provide a more resilient building-stock. While this might appear to be a brute-force approach to improving seismic resilience, the comparatively good performance of buildings in recent Japan earthquakes centred near major urban areas suggests this is a very reliable and penetrative means to improving seismic performance of buildings.

Introduction

Recent New Zealand earthquake event experiences from 2010 to 2016 have given our modern seismic design Standards their first real test over a range of demands from Serviceability Limit State (SLS) through to Life-Safety, or Ultimate Limit State (ULS). Observations following the Christchurch and Kaikoura earthquake events noted that the Life-Safety objectives of the New Zealand Building Code are generally being met by code consistent structural design, even if questions have been asked in the years following about the suitability and communication of these performance objectives. These questions have also tied into discussions around what our SLS or Low-Damage Design (LDD) performance targets are set at, and what they are intended to provide.

The Darfield earthquake of September 4th 2010 generated ground motions in central Christchurch that were higher than the code SLS design spectrum at the time, but significantly less that ULS. As described by Hare et al. (2012), there was a disconcerting amount of light-to-moderate structural damage in the central city buildings that could be attributed not only to the detailing of typical structural systems in use pre-earthquake, but also to the flexibility of the lateral-force resisting systems prevalent in the city. Non-structural damage was also extensive, although given the level of shaking to relative code, was perhaps not seen as such an issue at the time. It is reasonable to suggest that the concern was driven more by the knowledge that these construction forms are consistent across all of our major urban centres.

The vulnerability of our existing building stock was further highlighted in the Cook Strait earthquake sequence in 2013, which resulted in SLS level ground motions through much of the Wellington central city. Recorded non-structural damage provided a further demonstration of the mismatch between our design approach and performance expectations, along with further indications that flexibility of the structural systems was again a significant factor in the types and extent of damage, both structural and non-structural (Holden et al., 2013). Although some regions of Wellington suffered much larger ground motions as a result of the 2016 Kaikoura earthquake, there were many observations of structural and non-structural damage where ground motions were similar to the code SLS spectrum (Bradley et al., 2017).

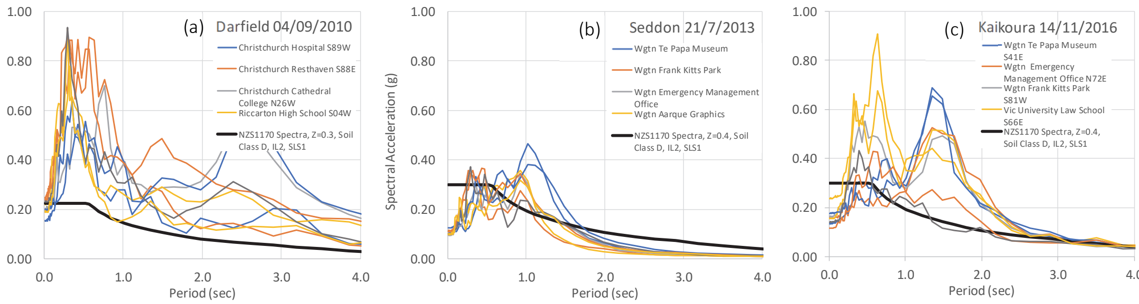

Figure 1. Acceleration spectra for (a) Darfield earthquake recorded in Christchurch (b) Cook Strait earthquake recorded in Wellington (c) Kaikoura earthquake recorded in Wellington.

The NZSEE reconnaissance team that visited Kumamoto city in southern Japan following the pair of large earthquakes in April 2016, was left with a strong impression of what resilient seismic design outcomes could look like for New Zealand. The subsequent publication (Sarrafzadeh et al. 2017) provided an opportunity to compare the typical design approach of the New Zealand Standards (NZS1170.5:2004) and Japanese Building Standard Law (BSL, 2016). The contrast in resulting design strength, and therefore stiffness, was stark yet consistent with the observed limited extent of damage in Kumamoto, when compared to Christchurch and Wellington. More recently the 2018 Osaka earthquake has again highlighted the beneficial implications of modern Japanese design practice of creating stiff and strong buildings.

Reflecting on the recent NZ experience has drawn a few different voices forward, proposing the need to not only be more assertive and consistent in how we define acceptable performance, but that this could be better met by increasing the SLS demands from the 25 year return period to something higher. This paper provides some simple demonstrations using displacement-based considerations, to highlight why this makes sense. The logic for this change becomes more apparent if we acknowledge the outcomes from Direct Displacement-Based Design (DDBD) (Priestley et al 2007), where we find that our designs using an evaluated ULS design ductility would inherently provide improved serviceability performance.

The New Zealand Design Scenario

Seismic design in New Zealand, whether by equivalent static lateral force or multi-modal response spectrum analysis, follows a design process whereby the ULS design ground motion is first used to determine the elastic design base-shear along with application of an assumed design ductility, μ, that drives the reduction in design base-shear (being represented by the factor S_p/μ, assuming $T_1 > 0.7$ in all cases here). This design ductility has typically not been verified as part of the design process, and therefore is a rather arbitrary input in much the same way as the ASCE 7 "*R*" (ASCE7-16, 2017) or Japanese BSL "D_s" factors. For normal buildings designated Importance Level 2 (IL2) our design ULS Return Period is 500 years (with Return Period Factor = 1.0), and SLS Return Period is 25 years (Return Period Factor = 0.25). It is noted here that the 25 year spectrum closely follows the ULS spectrum when a total spectrum reduction of six (S_p/μ) is applied for design.

Using the ULS design base-shear the lateral-force resisting structure is sized and designed to meet these demands, with Capacity Design being applied in well-developed approaches provided in the relevant NZ design Standards. Once this design is complete, a final analysis of the structure subject to SLS lateral loads is carried out to confirm that drift limits are below acceptable targets, which are not singularly defined and therefore somewhat open to building specifics and interpretation. Assumed cracked stiffness values at SLS (and ULS) are applied for reinforced concrete structures, without guidance or requirement to verifying their appropriateness. More often than not SLS design is left to a simple check confirming that maximum assumed ductility (based on checking forces rather than yield deformations) and peak drift limits are not exceeded.

Japanese BSL Design and US Performance-Based Design — A Reverse Approach to NZ

Following from the NZSEE reconnaissance team report on the structural observations from the Kumamoto earthquake, our understanding of the typical seismic design approach (for buildings less than 60m high and relatively regular in form) required by the Japanese BSL sees buildings sized and strength designed using Allowable Stress Design (ASD), to not exceed a maximum serviceability (Level 1 $C_0 = 0.2$) storey drift of 0.5%. For steel and reinforced concrete structures, the allowable steel stress equals nominal yield. With this Level 1 design complete the frame structures are then reviewed using a pushover analysis to confirm that sufficient base-shear capacity develops at the point of 1.0% storey drift against code Life-Safety demands (Level 2 $C_0 = 1.0$), and wall structures have a simple strength capacity check. While the Japanese design levels are not probability-based, it is understood that Level 1 and Level 2 approximately correlate to 50 year and 500 year return periods respectively. It is also interesting to note that Capacity Design is not enforced within the BSL documents.

Typical seismic design in the United States to ASCE 7-16 (2017) does not set-out a serviceability design level, however the PEER Tall Buildings Guidelines (PEER, 2017) and Los Angeles Tall Buildings Structural Design Council document (LATBSDC, 2017) do specify a Serviceability Level Earthquake (SLE) that in the case of the LA guidelines represents the 43 year Return Period event with assumed damping of 2.5%. This corresponds to approximately the same demand as the 72 year event at 5% damping. Similar to the Japan BSL approach, the PBD guidelines have engineers designing first to the SLE to meet a 0.5% drift limit with little or no structural damage. Following this, performance checks under MCE ground motions are carried out, although no explicit design is required. Bearing in-mind that these guidelines are the result of Tall Buildings initiatives, this definite focus on serviceability design rather than serviceability checks is a significantly different approach to New Zealand. The proposed Functional Recovery limit-state proposed in California, while still in its early stages of definition, will presumably be the first step in seeing SLE-type requirements imposed across standard building designs.

What is the Implication of Design Sequence?

By targeting SLS drift there is an inherent focus on stiffness and on limiting deformations to being less than yield. By restricting the structure to elastic response with SLS demands, the principles and intent of displacement-based approaches such as Direct Displacement-Based Design (i.e. aligning deformation, stiffness and ductility demand being $\mu = 1.0$) are being quasi applied to the structural design. Using elastic analyses in which assumptions of effective/cracked stiffness are likely to be better correlated at the SLS level and therefore the resulting internal force demands, the limit state design maintains better alignment between analysis and design assumptions. Whether the member design itself uses ASD or Load Reduction Factor Design (LRFD) to maintain elastic response at the SLS limit state, the outcomes appear better conditioned than standard NZ practice where stiffness, ductility and deformation are loosely contained by observed trends in analysis and (at times) tenuous assumptions for inelastic ULS behaviour.

Assuming the seismic hazard is appropriately represented, it seems reasonable to expect that the Japan BSL and US Performance-Based Design approaches should provide better performing buildings under serviceability level events than we could expect from the current NZ approach. How this extends to ULS performance needs further consideration and would certainly require a more comprehensive study than presented here.

If strength and stiffness are driven by SLS design to a target maximum drift that is less or equal to yield, then restrictions are inherently placed on the maximum ULS ductility demand that can be expected for a given peak ULS storey drift. For example, if we assume that nominal yield is just initiated at the SLS drift limit then we know the absolute maximum design ductility that could be applied to our ULS design, based on also being restricted to a peak ULS drift limit. Conversely, in the Japan BSL scenario where base-shear reduction factors (D_s that are effectively equal to S_p/μ or 1/R) are stipulated for various systems, having a known ductility means that the maximum ULS drift could be back-calculated and used to confirm suitable or consistent Life-Safety performance. What should be noted, and is demonstrated below, is that for most typical structural geometries the nominal yield drift of frames or walls will exceed likely SLS drift limits.

As discussed in detail by Sarrafzadeh et al. (2017) and also noted by Priestley et al. (2007), the Japanese D_s factors represent a significantly lower spectrum reduction than the New Zealand and ASCE 7 allowances. As will be shown below, these D_s values have a much closer alignment to the design ductility values that result from Direct Displacement-Based Design.

Matching up SLS to ULS Demands

In the NZ context if it is assumed that the SLS limit is achieved at the point of nominal yield, the ratio of elastic ULS:SLS demand defines the maximum spectrum reduction using a combination of μ and S_p. Following current NZS1170.5 spectrum definitions (ignoring the minimum base shear limits), this would have the maximum design ductility $\mu = 4.0$ ($S_p/\mu = 0.175$), as shown in Figure 2a. Ductility values larger than this will result in SLS base shears governing the design. By comparison the maximum reinforced concrete moment-frame reduction that can be applied to the Japanese spectrum is $D_s = 0.3$ (and 0.25 for steel moment-frames), which is still 150% of the Level 1 serviceability demand (Figure 2b). It is assumed that in-order for the base shear capacity to be sufficient during the Level 2 Life-Safety check, the system overstrength (due to Allowable Stress Design and some inelastic action overstrength) makes up some of the difference here, where typical MRF system overstrength could be 30-40%. In the case of the LA spectrum, the maximum value of R = 5 follows the assumed serviceability spectrum (derived from https://seismicmaps.org/) in Figure 2c

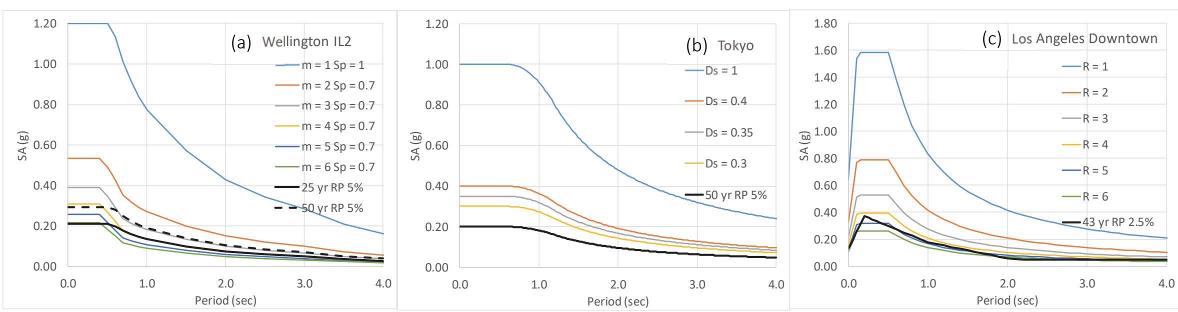

Figure 2. Comparison of design acceleration spectra for (a) Wellington ULS for range of ductility and with SLS1 25 year and 50 year spectra (b) Tokyo Level 2 for a range of RC frame D_s factors and Level 1 (c) Los Angeles DBE for a range of R-values and approximate SLE 43 year RP spectrum.

Clearly if New Zealand was target improved seismic performance, lifting the SLS return period from 25 year to 50 years (or more) is the most direct method approach to achieving improvement. However at what cost, has not been comprehensively evaluated and would need comprehensive cost-benefit analyses to initiate such a step-change. Since the Canterbury earthquakes, there have been a few high-level contributions to demonstrate what this effect on cost is (e.g. Moore, 2018) with indications being that it is not particularly significant to the total project cost if following traditional seismic design approaches.

The following section will however demonstrate that the perceived additional construction cost of targeting improved SLS performance becomes irrelevant if we acknowledge that in most instances our assumptions on ULS design force reduction do not capture the likely performance under design-level ground motions. In-fact, if we either adopt realistic reduction factors or force their explicit evaluation, we find that the SLS performance will inherently improve towards the 50 year return period.

Displacement-Based Design – the Alternative Pathway to the same Destination?

An alternative approach to improving the seismic performance of New Zealand buildings could come from adopting Displacement-Based Design into standard practice, or at least acknowledging the implications of knowing the yield deformation of the seismic-resisting structure. As has been demonstrated by various authors over the past 20 years, both theoretically and experimentally, yield drifts or displacements tend to be much larger than is acknowledged when applying arbitrary spectral reductions for ductility. Figure 3 provides a summary of (a) reinforced concrete moment-frame yield drift and (b) shear wall yield drift, where it is clear that with likely geometries of each system, the yield drift at the effective height will often be 0.7-1%.

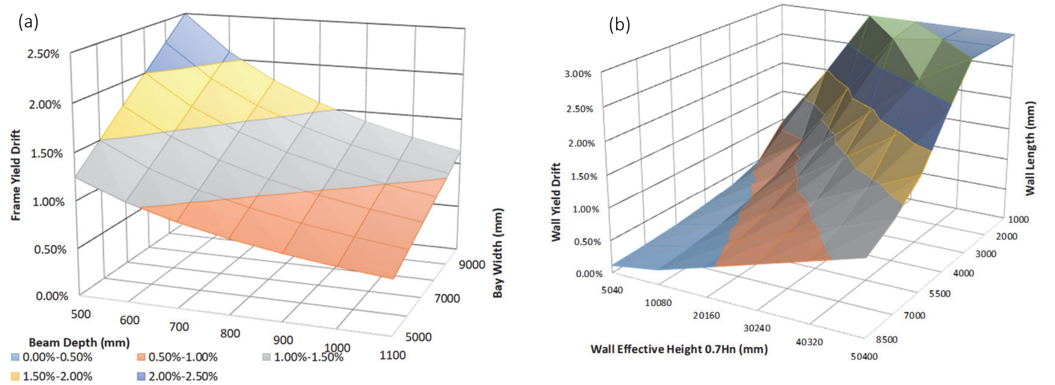

Figure 3. (a) RC moment-frame nominal yield drift as a function of beam-bay span and depth (b) RC shear-wall nominal yield drift as a function of wall effective height (0.7H_n) and wall length. First yield is assumed to typically occur at 65-75% of nominal yield.

If we assume that the maximum drift is set at 2.5% then for the range of yield drifts in Figure 3, the design ductility values can be calculated, as presented in Figure 4. Note that Figure 4a shows the frequency of design ductility values calculated from the range of beam-bay spans and beam depths in Figure 3a, while Figure 3b shows the design ductility for given wall length and effective height combinations. Between each of this plots, it is apparent that for typical frame or wall geometries the design displacement ductility that matches the likely inelastic response for given a peak displacement (or drift) is μ = 1.5 to 2.5 for frames, with values tending to be μ = 2.0 to 3.5 for wall structures. Aspects of these outcomes were touched on in a displacement-focussed force-based design approach described by Deam (2005).

A further means to investigating the margin between targeted performance and demand is to consider the design displacement spectrum for frames and walls in Wellington, which in Figure 5 has curves for 25, 50 and 75 year return periods presented. The dots provided over these spectra represent 0.5% drift at the effective height for six equivalent SDOF structures for which the SLS elastic period has been derived from the simple period estimation equations in the Commentary to NZS1170.5:

$$T = 1.0k_1 {h_n}^{0.75}\ x\ 1.33 \qquad (1)$$

Where h_n is the total structural height (m), k_1 = 0.075 and 0.05 for RC frames and walls respectively, and the 1.33 multiplier is introduced to amplify the conservatively low estimate resulting from the simplified Commentary equation to something closer to typical dynamic analysis period evaluations. The orange dots in Figure 5 represent these factored period estimates (the blue dots are described below).

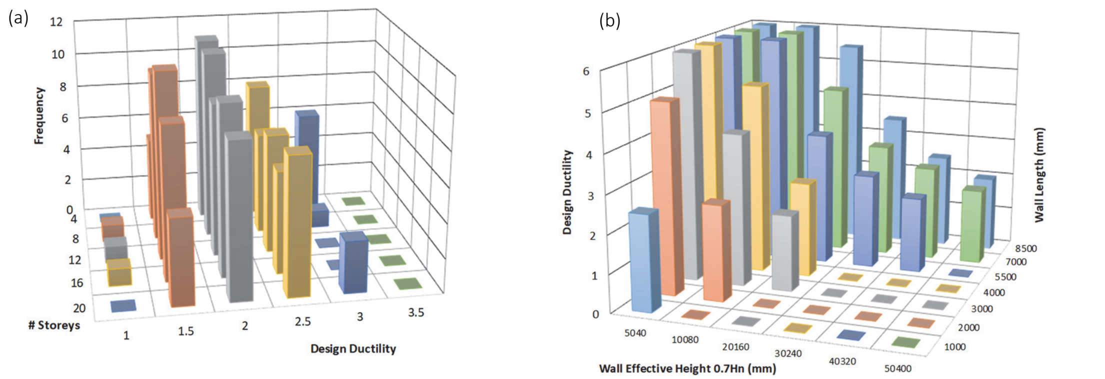

Figure 4 (a) RC moment-frame design ductility for the range of frame geometries in Figure 3a assuming peak storey drift is 2.5% (b) RC shear wall design ductility as a function of wall effective height (0.7Hn) and wall length assuming peak storey drift of 2.5%.

The SLS target drift is a value that has been in discussion in New Zealand in recent years. The use of 0.3% correlates to a conservative limit for typically detailed plasterboard wall linings, however 0.5% has been under consideration as a more general target for low damage design. This equals the value set in the Japan BSL for Level 1 design, and similarly for SLE design in the US Performance-Based Design guidelines. The key outcome from the plots in Figure 5 is that the 50 year return period displacement demand is consistently lower than the period-displacement point associated to this assumed limit state drift (noting that in-plan torsion and higher-mode effects might well cover some of this difference).

A further point of note is that lower design ductilities, either specified by code or evaluated via DDBD, result in increased building strength (say by a factor of two). The elastic period will therefore reduce by ~30% as a result of the strength and stiffness dependency. If this reduction is applied to the estimate from Eq.(1) then the period-displacement points in Figure 5 will translate back as shown in the two plots (blue dots). This approximate estimate shows that the 50 year return period design spectrum demands can still be aligned with target drift limits, potentially even more stringent than 0.5%. If design was driven by SLS performance, then plots such as in Figure 5 quickly demonstrate that wall systems are a better option to meeting lower drift limits when required, an outcome that is intuitive.

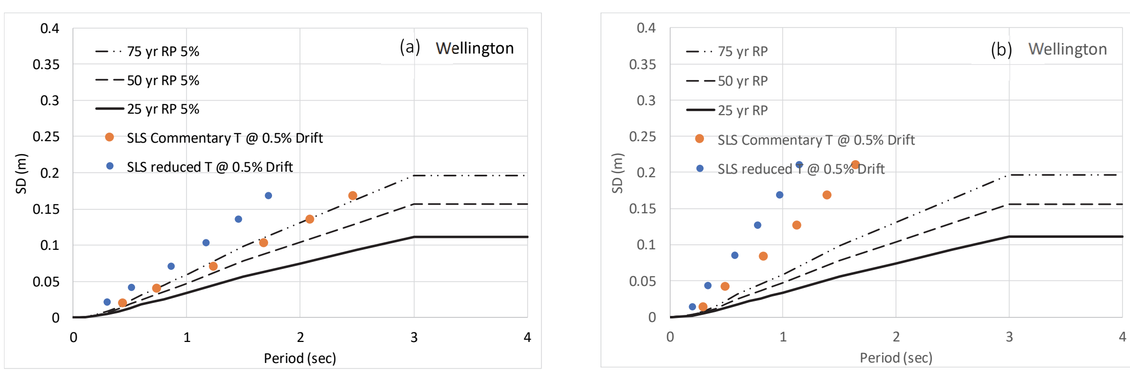

Figure 5 Centre-of-mass response building heights n = 2 to 20. Orange dots represent design to typical ductility (likely μ = 4 or more), blue dots represent the approximate the reduction of design ductility to 1.5-2.5 (a) RC frame approx. elastic period-design displacement points (b) RC shear wall approx. elastic period-design displacement. Allowance for torsion amplification of 20% has been made in determining a reduced centre-of-mass displacement.

Demonstrating Design to Serviceability in the NZ Context

A nine-storey case-study two-way (concrete filled tube columns) steel moment-frame building was designed to the Wellington IL2 ULS (500 year, R = 1.0) and then to SLS (50 year, R = 0.35) spectra. Building 1 uses a typical design approach to ULS with an assumed μ = 4.0 and Capacity Design (NZS3404:2009 Category 1) applied, with drift checks at SLS only. Building 2 targeted ULS design with ductility μ = 2.0 (Category 2) which was determined by evaluating the nominal yield drift in each direction and assuming a target maximum storey drift of 1.75%. It is noted that the transverse frames have 12m long spans which suggest design ductility demands less than μ = 1.5. Building 3 used the proposed SLS design with a ductility of μ = 1.0, however the structure was assumed to be Category 3, and Capacity Design was applied in-line with NZS3404:2009 requirements.

All three structures were subjected to a suite of seven record-pairs, scaled to match the 500 year and 50 year return period design spectra. Figure 6 shows the average ULS and SLS drift profiles for each building. As expected, compared to Building 1 the storey drifts are better controlled by the design for Buildings 2 and 3 at both SLS and ULS demands. It is interesting to note that although the drift profile shape differs between Building 2 and 3 (a function of the beam size distribution), the peak storey drifts are very similar and typically below the target SLS 0.5% drift. This is likely due to the influence of Capacity Design on the columns.

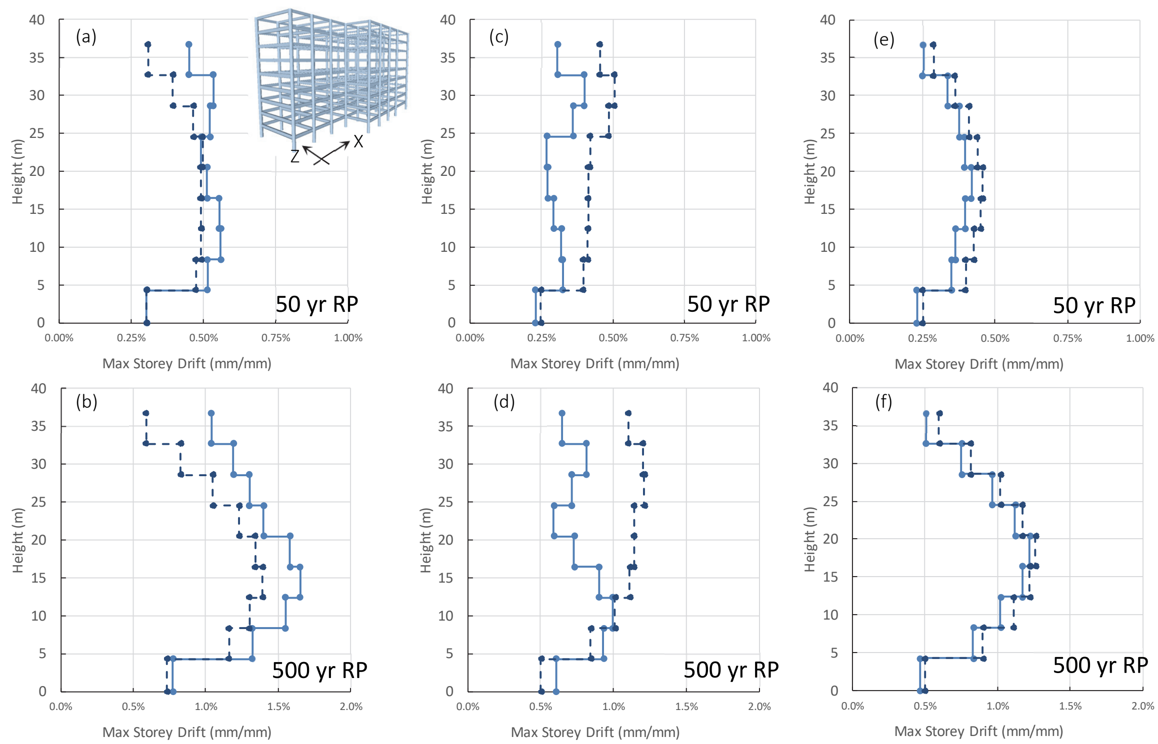

Figure 6. Average diaphragm centre-of-mass maximum storey drift profiles (a & b) Building 1 ULS design with μ = 4 typical NZ design (c & d) Building 2 ULS design with μ = 2 displacement-based ductility evaluated design (e & f) Building 3 SLS 50 year RP design with μ = 1.

The demonstrations in the sections above are very much exploratory to gain an understanding of what the influence of changing the NZ design philosophy might represent. To advance either the SLS-based design or the displacement-based ductility evaluated ULS design (or otherwise code restricted ductility) approaches, a more rigorous study is needed to better understand the cost-benefit associated to the increased design base shear coming from either method. As discussed by Moore (2018), the point of diminishing return with increasing SLS design Return Period needs to also be considered in this process. A further issue not discussed here is the level of design acceleration used for non-structural restraint in our buildings. This is arguably as important as considerations of storey drift, and would play as bigger part in in determining an improved level of serviceability driven seismic design.

Conclusions

Following recent seismic events in New Zealand there has been a significant amount of discussion around acceptable performance for ground motions less than Ultimate Limit State. This paper has provided a brief demonstration that the current New Zealand approach to seismic design, which is driven by ULS design using arbitrary spectrum reductions followed by SLS performance checks, could be significantly improved if it were to adopt an approach in-line with Japanese Building Standard Law or Californian Performance Based Design. Both these modern overseas approaches design elastically to SLS and confirm acceptable performance checks for Life-Safety (and beyond). The result of this approach is that ductility demands are inherently determined by the hazard and the design process, rather than arbitrarily assumed by the engineer.

An alternative, that may find further traction in New Zealand is to adopt displacement-based design aspects that require explicit evaluation of yield deformation and therefore calculation of expected design ductility. This approach generally produces lower values of design ductility than have been assumed in New Zealand design to-date, bringing a more consistent relationship between ULS and SLS design.

Further investigation is required to better understand the associated costs and benefits of the suggested changes to our seismic design approach. The evidence from recent earthquakes in Japan is that the increased emphasis on serviceability design, rather than performance checking, is effective and of considerable benefit.

Acknowledgements

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