17th World Conference on Earthquake Engineering, 17WCEE
Sendai, Japan - September 13th to 18th 2020

JAPANESE AND NEW ZEALAND BUILDINGS – THE SIGNIFICANCE OF RESILIENT SERVICEABILITY GOVERNED DESIGN

D. Pettinga[(1)], Y. Hori[(2)], Y. Suzuki[(3)], M. Nakashima[(4)]

[(1)] *Technical Director, Holmes Consulting LP, didier.pettinga@holmesconsulting.co.nz*
[(2)] *Deputy Manager, Kobori Research Complex Inc., horiyus@kobori-takken.co.jp*
[(3)] *General Manager, Kobori Research Complex Inc., y-suzuki@kobori-takken.co.jp*
[(4)] *President, Kobori Research Complex Inc., nakashima-masayoshi@kobori-takken.co.jp*

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 dictates quite a different application of a two-tiered limit-state design, with serviceability design preceding Life-Safety checks. 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.

This contribution explores the key differences between Japanese and New Zealand seismic design performance expectations, and provides an evaluation of the alignment between design requirements and possible performance outcomes. Further discussion on the expected repairability from new building design is also provided, as this topic is becoming increasingly important in high-seismic hazard regions around the world.

Keywords: Performance based seismic design; Limit-State design; Japan; New Zealand; Repairability

1. 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 in [1], 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 the major urban centres in New Zealand.

The vulnerability of New Zealand's 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 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 [2]. Although some specific regions with soft soils in 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 [3].

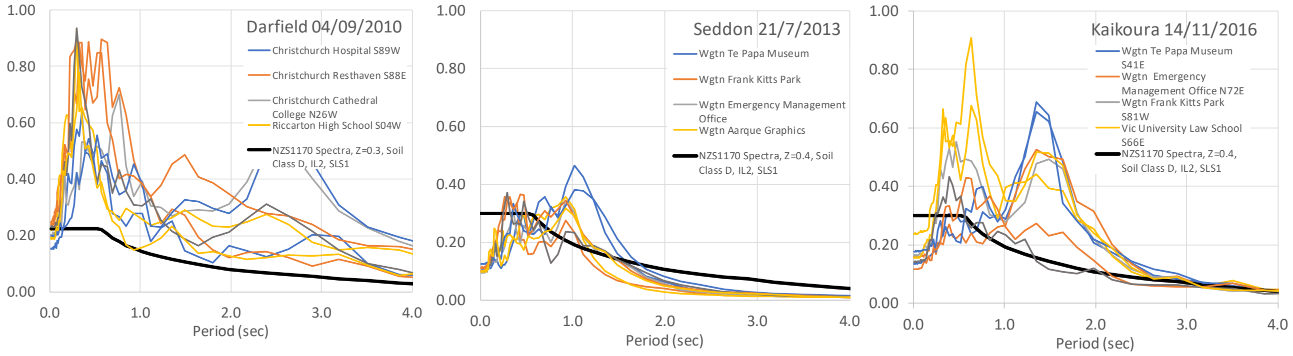

Fig. 1 Acceleration spectra for (a) Darfield earthquake recorded in Christchurch (b) Cook Strait earthquake recorded in Wellington (c) Kaikoura earthquake recorded in Wellington. The NZS1170.5:2004 design spectra are SLS1.

In contrast, 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 [4] provided an opportunity to compare the typical design approach of the New Zealand Standards [5] and Japanese Building Standard Law [6]. 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 New Zealand 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) [7], where we find that our designs using an evaluated ULS design ductility would inherently provide improved serviceability performance.

2. Comparing the Underlying Seismic Design Philosophies

2.1 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 that relates only to the inelastic capacity of the structural elements, rather than being calculated from yield deformations, in much the same way as the ASCE 7 "*R*" [8] or Japan 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 in design. It also noted that the values of μ used in design are not prescribed for given systems, as is the case in the Japan BSL and ASCE 7.

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.

2.2 Japanese BSL Design – A Reverse Approach to NZ

The typical seismic design approach of moment-frames (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 (when taller than 31m) 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 $C_0 = 1.0$).

For wall structures, the design is driven by simple strength capacity checks to Level 1 ASD demands, again with prescribed strength reduction factors that, when compared to NZ or ASCE7 values, result in significantly higher design base-shear values.

A further key aspect of the standard BSL approach is the use of sectional area limit checks of the primary lateral system vertical elements (i.e. columns and walls), as discussed by Sarrafzadeh et al. (2017). This simple approach also reinforces the requirement for generally stiffer and stronger lateral force-resisting systems in typical buildings.

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, for the Tokyo region. It is also interesting to note that Capacity Design is not enforced within the BSL documents.

Of further interest is that the US Performance-Based Design movement, being largely been driven by the PEER Tall Buildings Guidelines [9] and Los Angeles Tall Buildings Structural Design Council document [10], bears many similarities to the Japanese BSL approach. Both these US documents specify a Serviceability Level Earthquake (SLE), which is otherwise not present in typical US seismic design [8]. In the case of the LA guidelines this represents the 43 year Return Period event, with damping being a function of building height. It is understood that the 43 year Return Period with 2.5% damping gives approximately the same demand as the 72 year event at 5% damping, depending on regional seismicity. 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 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.

3. How have we Arrived at these Different Approaches?

The differences between New Zealand and Japanese standard seismic design approaches are striking, and based on recent earthquake experience the building performance outcomes equally different. From the 1970s until 2010, New Zealand had not experienced a damaging earthquake near a large population centre. Through this period well-known developments, such as Capacity Design [11], came from New Zealand research. Similarly the general trends of in-practice seismic design that evolved over this 30-40 year period had gone untested, meaning that seismic design practice was largely anchored on academic research and experimental findings coupled with first-principles theory. The combined direction of this work had provided a demonstrable means to achieving Life-Safety performance, associated to the developing Ultimate Limit State design focus of the New Zealand codes.

By comparison, Japan experiences regular design-level seismic events that often affect major population centres. The regular opportunity for field observation of building performance under strong ground motions has enabled the research and practicing structural engineering communities in Japan to acknowledge unsatisfactory structural behaviour and adjust the building codes accordingly following post-event research and scrutiny. It is arguable that the un-intended result of this is that design regulations and design approaches have evolved with a heavier basis from experience and observation, rather than seismic design theory. This is a very different situation to New Zealand, and one in which potential behaviours not observed to occur (or be problematic) are not interrogated further. Much of the current Japan BSL approach stems from changes introduced after the 1923 Kanto earthquake, where C0 Level 1 design was defined using Allowable Stress Design with the target of elastic building response under this Rare Earthquake event.

From in-the-field experience the Japan BSL has arrived at a definition of hazard and design process that creates robust structures across a wide-range of the country's building-stock i.e. it is not just high-profile buildings that receive the design input necessary to achieve resilient seismic performance. Even 'typical' buildings are channelled towards conforming to a stronger building population that, whether by intention or not, in recent events has demonstrated many of the desired outcomes of Low Damage Seismic Design in New Zealand, or Functional Recovery, both of which are currently evolving in the NZ and the United State respectively.

It is the authors' consideration that the Japan BSL emphasis on SLS design coupled with elastic structural response, along with the intensity of the SLS ground motion relative to the Life-Safety, is a fundamental reason for the positive outcomes in recent major earthquake events in Japan.

The reader is referred to the design comparisons provided in [4], for demonstrations of the design base-shear differences following NZ and Japanese building codes.

4. What is the Implication of the 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.

As a simple demonstration of the outcomes from the Japan BSL, the following Fig. 2 provides the Level 1 and 2 storey drift response summary for a modern typical ductile 9-storey two-way steel moment-frame designed for Tokyo ground motions. The building was subjected to a representative suite of earthquake records, with the envelope of peak drift being recorded and shown here. The drift profiles can be compared against the results provided in Fig. 7(a & b), for the same building designed to current New Zealand Standards and typical methodology. In general, our studies have found that structural steel moment-frame buildings designed to the Japan BSL tend to have better Serviceability performance than typical NZ designs, but reasonably similar Life-Safety performance. Based on recent research by others [4] it felt that this is consistent with other lateral force-resisting forms.

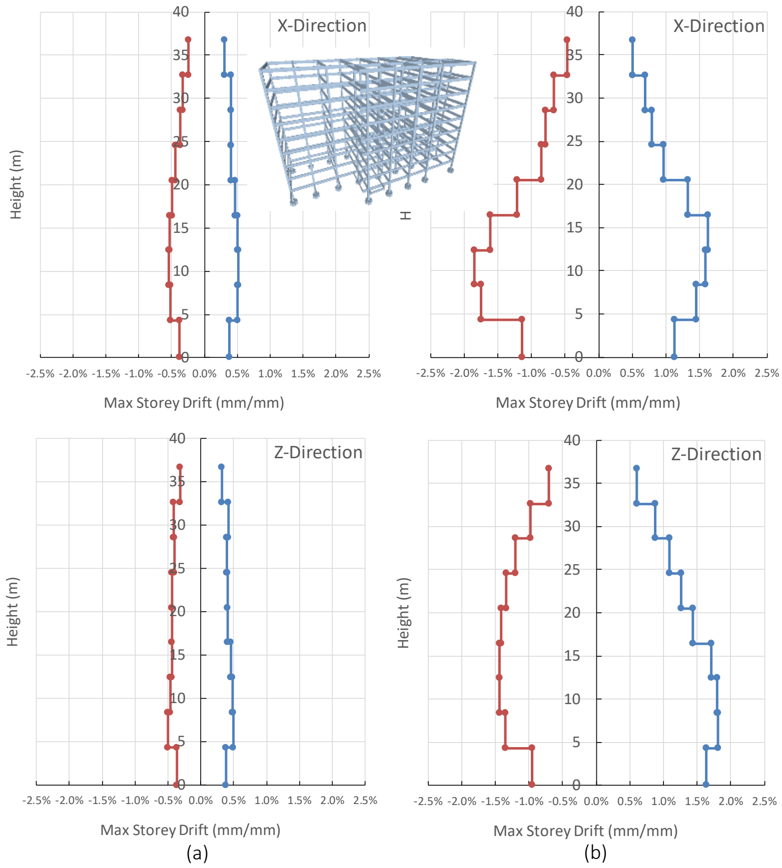

Fig. 2 Example Japan BSL designed two-way steel moment-frame building envelope of storey drift response under (a) Level 1 and (b) Level 2 ground motion demands. The design did not utilise Performance-Based Design drift limits.

If strength and stiffness are driven by SLS design to a target maximum drift that is less than 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 by a peak ULS drift limit. Conversely, in the Japan BSL scenario where base-shear reduction factors (Ds that are effectively equal to Sp/μ or 1/R) are stipulated for vfarious systems, having a known ductility means that the maximum allowed ULS drift could be back-calculated and used to confirm suitable or consistent Life-Safety performance. What should be noted, and is demonstrated in the sections following, 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 in [4], and also noted in [7], the Japanese D_s factors represent a significantly lower spectrum reduction than the New Zealand and US allowances. As will be shown below, these Ds values have a much closer alignment to the design ductility values that result from Direct Displacement-Based Design.

4.1 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 ULS 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 μ = 4.0 (S_p/μ = 0.175), as shown in Fig. 3a. 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 (Fig. 3b). 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 Fig. 3c.

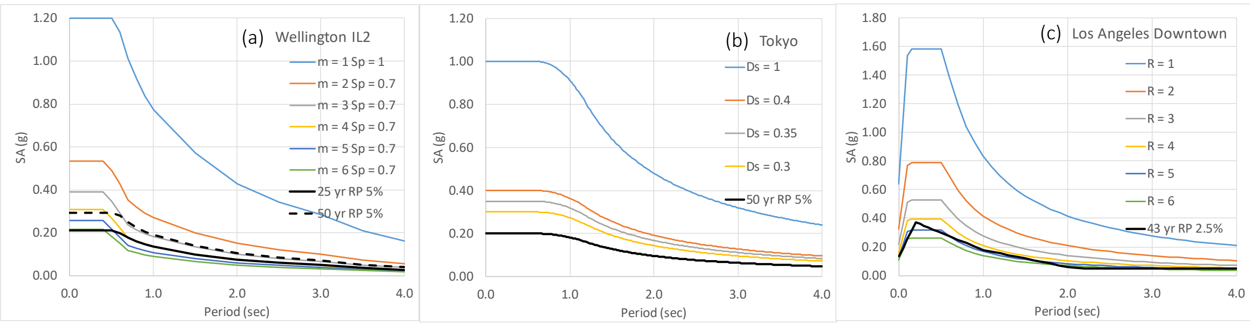

Fig. 3 Comparison of design acceleration spectra for (a) Wellington ULS for range of ductility and with SLS1 25 year and 50 year spectra (b) Tokyo Level 2 for a range of RC frame Ds factors and Level 1 (c) Los Angeles DBE for a range of R-values and approximate SLE 43 year RP spectrum

Clearly if New Zealand was to target improved seismic performance, lifting the SLS return period from 25 year to 50 years (or more) is the most direct approach to achieving improvement. However at what cost has not been comprehensively evaluated, and would need cost-benefit analyses to initiate such a step-change. Since the Canterbury earthquakes, there have been a few high-level contributions to demonstrate the effect on cost [12], 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.

4.2 Improved Performance and Repairability between SLS and ULS

Recent seismic events in New Zealand have led to increasing demand for performance beyond Life Safety in strong (design level) earthquakes. After both the 2011 Christchurch and 2016 Kaikoura Earthquakes engineers were faced with challenging assessment and little technical guidance on the residual capacity and reparability of many moderately damaged ductile RC structures. This culminated in drawn-out decision-making processes, with unoccupied buildings incurring ongoing costs to building owners and often leading to the demolition of these structures. Such cases have led to discussions on the performance objectives of New Zealand's seismic design philosophy, and the potential role for a performance level that bridges the divide between SLS and ULS design. Marder et al. [13] discuss the possibility of an intermediate design level in the form of a "reparability limit state" (RLS). The consideration of reparability in the design phase would largely be aimed at increasing the probability of repair following damaging events and hence the likelihood that decisions could be expedited, and a structure could be re-occupied shortly after an earthquake.

An increase in the SLS return period in New Zealand would result in a shift towards stiffer structures with reduced displacement demands. The effect of such a change would not only improve the seismic performance of our building stock to achieve SLS performance objectives, but inherently work to increase the probability that a structure is in a reparable state following a damaging event beyond SLS. Investigations into the repairability of RC structures have thus far been able to demonstrate the feasibility of repair of RC beam plastic hinges when damage is deemed to be low to moderate [13]. The impact of repair on such components has yet to be investigated on global building performance. Research is ongoing at the University of Auckland (NZ) to further investigate this topic, with further tests being done on repair of plastic hinge regions via epoxy injection. The results of these tests are intended to be carried forward into analyses on global building behaviour with the aim of using these results to define a RLS in terms of engineering demand parameters

5. 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. Fig. 4 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 building effective height will often be 0.7-1%.

If we assume that the maximum drift is set at 2.5% then for the range of yield drifts in Fig. 4(a & b), the design ductility values can be calculated, as presented in Fig. 5. Note that Fig. 5a shows the frequency of design ductility values calculated from the range of beam-bay spans and beam depths, while Fig. 5b 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 $\mu = 1.5$ to 2.5 for frames, with values tending to be $\mu = 2.0$ to 3.5 for wall structures. Aspects of these outcomes were touched on in a displacement-focussed force-based design approach described in [14].

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 Fig. 6 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:

$$T = 1.0 k_1 {h_n}^{0.75}\, x\, 1.33 \tag{1}$$

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 Fig. 6 represent these factored period estimates (the blue dots are described below).

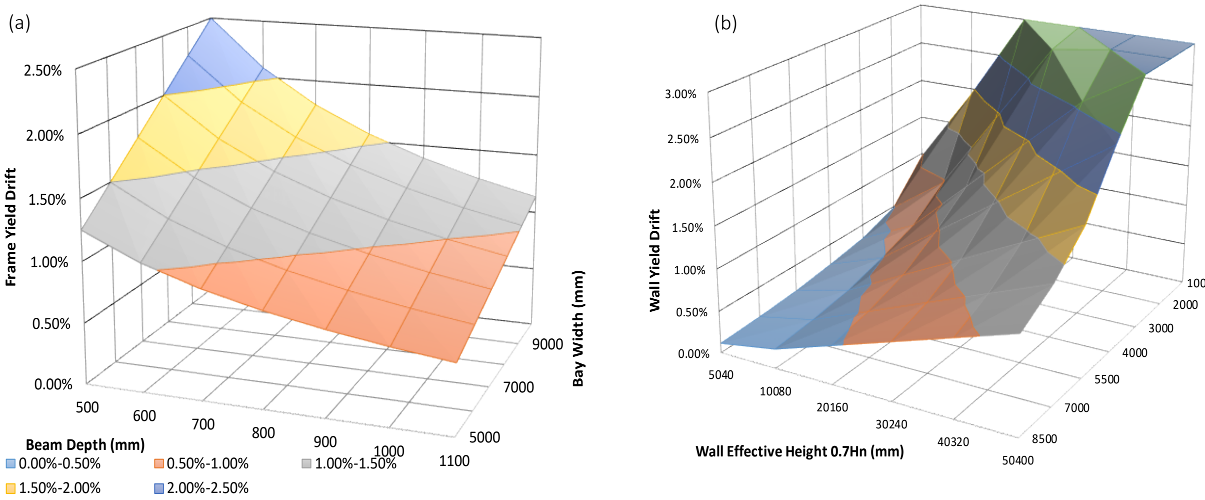

Fig. 4 (a) RC moment-frame nominal yield drift as a function of beam-bay span and depth (b) RC shear-wall nominal yield drift as a function of wall effective height (0.7Hn) and wall length. First yield is assumed to typically occur at 65-75% of nominal yield

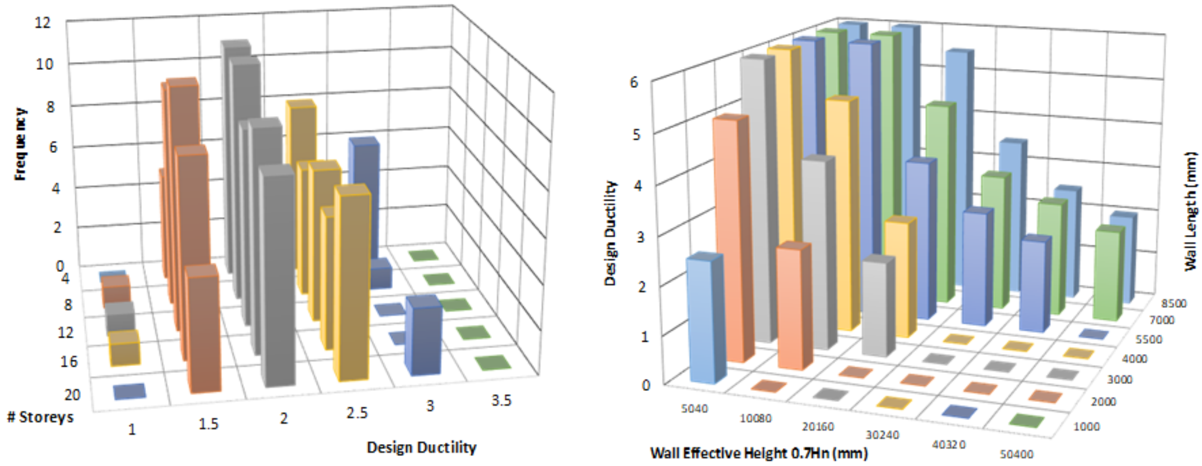

Fig. 5 (a) RC moment-frame design ductility for the range of frame geometries in Fig. 4a assuming peak storey drift is 2.5% (b) RC shear wall design ductility as a function of wall effective height (0.7Hn) and wall length assuming peak storey drift of 2.5%.

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 Fig. 6 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).

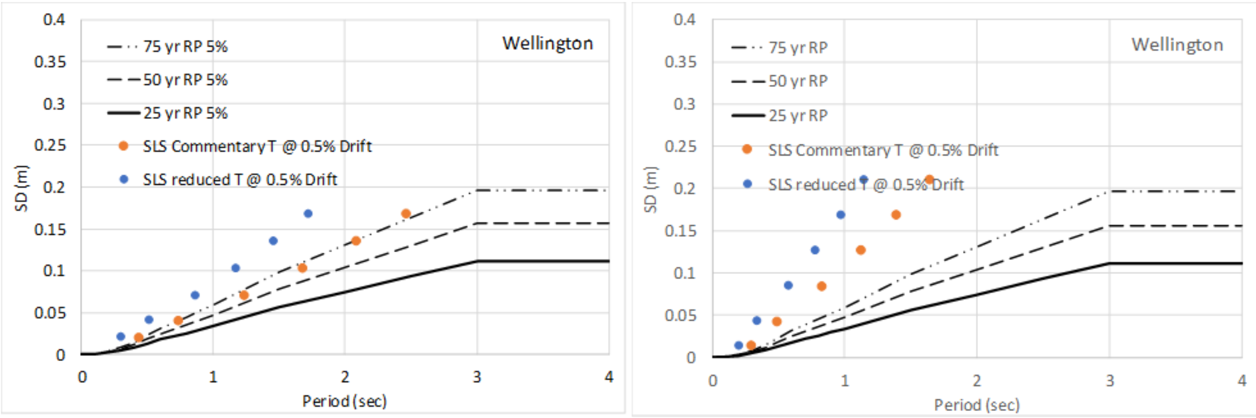

Fig. 6 Centre-of-mass response building heights n = 2 to 20. Orange dots represent design to typical ductility (likely μ= 4 or more), blue dots represent the approximate the reduction of design ductility to 1.5-2.5 (a) RC frame approx. elastic period-design displacement points (b) RC shear wall approx. elastic period-design displacement. Allowance for torsion amplification of 20% has been made in determining a reduced centre-of-mass displacement.

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 Fig. 6 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 align with target drift limits potentially more stringent than 0.5%. If design was driven by SLS performance, then plots such as in Fig. 6 quickly demonstrate that wall systems are a better option to meeting lower drift limits when required, an outcome that is intuitive.

One further point regarding wall systems and yield deformation that is worth noting, and would need to be considered in future studies around repairability and cost-benefit, is that stiffer walls will tend towards longer walls. In-step with Fig. 4(b) this will reduce the yield curvature (and therefore yield drift), which will likely lead to early onset of cracking in the lower levels of the wall. It might well be that this earlier development of cracking and stiffness reduction counters some of the benefits from the initially stiffer structural system.

6. Demonstrating Design to Serviceability in a New Zealand Context

A case-study nine-storey 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 [15] 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 [15] 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. Fig. 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.

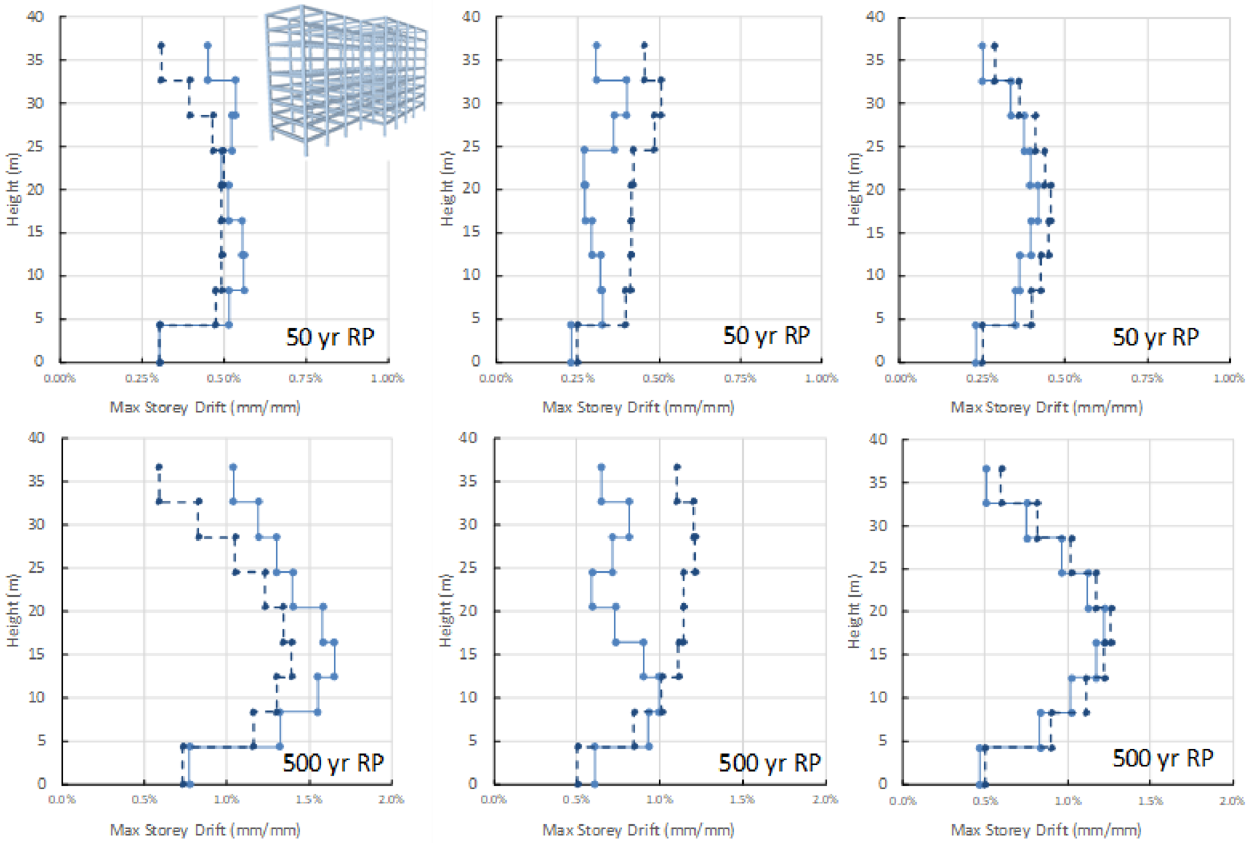

Fig. 7 Average diaphragm centre-of-mass maximum storey drift profiles (a & b) Building 1 ULS design with μ = 4 typical NZ design (c & d) Building 2 ULS design with μ = 2 displacement-based ductility evaluated design (e & f) Building 3 SLS 50 year RP design with μ = 2.

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 in [12], 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.

7. 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. Although Life Safety objections have largely been met, an increasing demand for functional and reparable structures following seismic events has been highlighted. 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.

8. Acknowledgements

The authors would like to thank Dion Marriott and Bill Tremayne for their advice and clarifications to the US design codes and practice.

9. References

[1] Hare, J., Oliver, S., Galloway, B., 2012, Performance Objectives for Low Damage Seismic Design of Buildings, *Proc. NZ Soc. Earthquake Engineering Conference*, Christchurch, New Zealand, #35, 2012.

[2] Holden, C., Kaiser, A., Van Dissen, R., Jury, R., 2013, Sources, Ground Motion and Structural Response Characteristics in Wellington of the 2013 Cook Strait earthquakes, *Bull. NZ Soc. For Earthquake Engineering*, Vol. 46(4), pp188-195.

[3] Bradley, B., Razafindrakoto, H., Nazer, A., 2017, Strong Ground Motion Observations of Engineering Interest from the 14 November 2016 Mw7.8 Kaikoura, New Zealand Earthquake, *Bull. NZ Soc. For Earthquake Engineering*, Vol. 50(2) pp. 85-93.

[4] Sarrafzadeh, M., Elwood, K.J., Dhakal, R.P., Ferner, H., Pettinga, D. Stannard, M., Maeda, M., Nakano, Y., Mukai, T., Koike, T., 2017, Performance of Reinforced Concrete Buildings in the 2016 Kumamoto Earthquakes and Seismic Design in Japan, *Bull. NZ Soc. For Earthquake Engineering*, Vol. 50(3), pp. 394-435.

[5] New Zealand Standard (NZS1170.5:2004), 2004, *Structural Design Actions Part 5: Earthquake Actions – New Zealand*, Standards New Zealand, Wellington.

[6] The Building Standard Law of Japan (2016), The Building Center of Japan (BCJ), Tokyo, Japan.

[7] Priestley, M.J.N., Calvi, G.M. and Kowalsky, M.J., 2007, *Displacement-Based Seismic Design*, IUSS Press, Pavia, Italy, pp 720.

[8] ASCE Standard (ASCE/SEI 7-16), 2017, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers (ASCE), 2017.

[9] PEER, 2017, *Guidelines for Performance-Based Seismic Design of Tall Buildings*, Pacific Earthquake Engineering Research Center, Report No. 2017/06, May 2017, pp. 147.

[10] LATBSDC, 2018, *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region*, Los Angeles Tall Buildings Structural Design Council, Los Angeles, USA., pp 74.

[11] Paulay, T., Priestley, M.J.N., *Seismic Design of Reinforced Concrete and Masonry Buildings*, Wiley, 1992, pp 744.

[12] Moore, T., 2018, Repairing SLS Anomalies in NZ Seismic Code to Reduce Earthquake Losses, *Bull. NZ Soc. For Earthquake Engineering,* Vol. 51(1), pp34-46.

[13] Marder, K., Sarrafzadeh, M. and Elwood, K.J., 2018, *Effectiveness of repair via epoxy injection of earthquake damaged reinforced concrete beam elements,* Proceedings of the 17th U.S.-Japan-New Zealand Workshop on the Improvement of Structural Engineering and Resilience, Queenstown, New Zealand.

[14] Deam, B.L., 2005, *A Displacement-Focused, Force-Based Structural Design Procedure*, Proc. NZ Soc. Earthquake Engineering Conference, Taupo, New Zealand, #34, 2005

[15] New Zealand Standard (NZS3404:1997), 1997, *Steel Structures Standard*, Standards New Zealand, Wellington.

