Directing Low-Damage Seismic Design with Building Functionality

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ABSTRACT

The upcoming New Zealand Low-Damage Seismic Design (LDSD) guidance information, currently in preparation, is widely anticipated to provide a reference point for aligning low-damage seismic performance targets. A key aspect of this guidance is neutrality in associating performance targets with different lateral force-resisting systems (LFRS) being considered for a given structure. Identified design targets are equally intended for differing structural systems, meaning that structural engineers can attempt to better align the LFRS with the building function or architecture, rather than be forced into particular structural formats. Not all lateral systems will be capable of providing efficient low-damage seismic performance as fixed-based structures, but they may well be suitable as the superstructure to a base isolated system, thereby making this an appealing option to open up more general coordination aspects i.e. better building utility. Designing for post-event functional recovery is a reference point that can be used to better inform LDSD via greater prioritization of building functionality preferences, rather than fitting function around a low-damage structure.

Using two base isolated buildings, designed and built over the past five years in central Christchurch, a brief discussion is presented around key inputs and outcomes that led to similar LDSD targets producing entirely different LFRS that best suited the day-to-day building functionality and assisted post-event functionality. While the designs were completed ahead of recently published NZ guidelines, performance targets closely aligned with both the LDSD guidance draft, and the design process reflected many of the key requirements in the NZSEE Base Isolation Design Guidelines.

1 INTRODUCTION

The past 10 years of the Christchurch rebuild have seen many positive signs that the building industry is prepared to invest in technology and methodologies that promote better seismic performance. The somewhat organic development of low-damage seismic design (LDSD) approaches to providing lateral force-resisting systems (LFRS) has meant that innovation from both universities and within the engineering industry has been fluid, and at times rapid in its uptake in practice.

Many authors (e.g. Hare et al, 2012) have noted the need for industry guidance on what a consistent definition of LDSD actually represents, and consistent method on how it can be achieved and promoted. Given the scrutiny that the structural engineering profession has been under over the past decade (Canterbury

Earthquake Royal Commission, 2012), the significance of being able to communicate consistent LDSD goals to the public whether presenting to owners, tenants, or otherwise, cannot be over-estimated.

The LDSD guidance information draft (in progress) has taken a form that is non-specific to what systems can achieve low-damage performance, and instead focuses on providing a framework and terminology to enable appropriate application and communication. Perhaps this is an unexpected benefit of having gathered the past 10 years' worth of industry experience on what we really need to be achieving. This framework will hopefully promote adaptability and continued low-damage technology, such that new ideas and means to tailoring the LFRS to building function can be brought into practice.

One particular benefit of utilizing an agreed framework for achieving LDSD targets is that it better facilitates development of structural solutions that can accommodate the needs or preferences of the client and/or architecture. While the explicit presence of LFRS (e.g. diagonal braces) has become more accepted in major New Zealand centres over the past decade, in the author's experience, architects and developers remain keen to explore opportunities to utilize a structural form that best suits their vision and anticipated function of the building.

The following discussion focusses on two case-study projects from Christchurch that exemplify the benefits of early LDSD discussions, that motivated opportunities to provide a building form that did not compromise the architecture. Arguably, through similar language as we see in the anticipated guidance, these projects were able to effectively explore LDSD options, and provide inputs to the decision-making process to facilitate better post-event functionality. Ultimately the projects achieve very similar LDSD performance while employing entirely different structural forms that closely align with traditional preferred systems for their utility.

2 LFRS TO SUIT BUILDING FUNCTION

In recent years, the idea of Functional Recovery has gathered momentum both here and in the United States (Haselton, 2019; EERI, 2019), with the current direction being one of equally weighting safety and recovery time in design, in-order to provide post-earthquake capacity such that pre-earthquake function is sufficiently maintained or restored. While the definitions and application of Functional Recovery are very much in their infancy, it can be expected that our understanding and successful implementation of Low Damage Design will inherently grow with moves to target Functional Recovery as a performance point during design. Observation of the early developments in the US for defining Functional Recovery, suggests that we will see objectives that inform how structural systems interact and align with the non-structural elements, and at the same time align with building function. Potentially the growing focus on what functionality is needed (and how soon) after a seismic event will be a catalyst for LDSD to permeate beyond the usual range of institutional and high-end commercial developments here in New Zealand.

2.1 Building Function Influences Low-Damage Needs

The typical open-plan nature of office fitouts means that the focus for seismic performance targets associated to LDSD or Functional Recovery can differ from other building usages, such as institutional or residential. Where large open-plan spaces are provided, there are often relatively few aspects of the fitout affected by lateral displacements, yet a significant amount of suspended items and fixed plant present that are acceleration sensitive, and necessary for building functionality.

At the other end of the spectrum are apartment developments where uninterrupted external views (i.e. no diagonal bracing), prevention of noise transmission and well-defined tenancy spaces are key functionality requirements. Typically, a LFRS of reinforced concrete walls forming shear cores around stair/lifts and lobby spaces, are an efficient means to meeting these requirements. With relatively limited suspended items, often with ceilings battened to the underside of floor slabs, the amount of acceleration sensitive fit-out is low compared to office or institutional spaces. Provided there is opportunity for temporary ventilation (opening windows etc), it is arguable that mitigation of high accelerations, often perceived as a negative outcome of shear-wall buildings, is not so important for post-earthquake functionality. In contrast, the large number of partition walls, particularly affecting exit pathways suggests that drift control becomes a more prominent driver for LDSD to align with Functional Recovery.

With the LDSD guidance information written in a framework that does not dictate what LFRS system type can be considered to potentially achieve low-damage performance, the above discussion might suggest that designers can focus on acceleration or drift, based on building function. However there are inherently cross-overs with most buildings that lead to acceleration and drift control being important, such that one can't solely focus on delivering LDSD via acceleration or drift considerations independently. Notably institutional buildings are generally a true mix of acceleration and drift sensitive fitout.

The opportunity offered by the LDSD framework is that a LFRS suited to acceleration or drift control and, as highlight above, well aligned with building utility can become a truly low-damage structure through being non system specific. The introduction of seismic base isolation is one approach to promoting standard superstructure forms to being low-damage, yet there are a growing range of systems and devices that a designer could consider. Hopefully we see continue to see more options coming into practice over the coming decade.

3 CASE STUDIES WITH LDSD TARGETS ⇔ FUNCTIONAL RECOVERY

The following two case-studies, both recently completed buildings in Christchurch, provide comparison to the potential for significantly different construction forms that were largely dictated by building use, to evolve from low-damage seismic design criteria that remain independent from the form of lateral force-resisting system. In these examples both buildings developed into base isolated systems, while key drivers for the final LFRS in each case were associated to accommodating architectural and general function preferences for building use. In both cases, without necessarily having the terminology of Functional Recovery, early set-out of the seismic performance revolved around rapid re-occupancy and improved levels of post-disaster functionality.

While both case studies were designed prior to the LDSD draft taking form, the decision-making process and key discussions were closely aligned to the intent that the guideline draft has taken. In both instances a balance of restricted drift limits and limiting ductility development were key inputs, and are key considerations whether on seismic isolation or not. The ability for the developments to remain open to architectural and functional preferences is seen as a key benefit to the direction that the LDSD guidance information has adopted.

Similarly, the NZSEE base isolation design guidelines (NZSEE, 2019) were not complete at the time of design, although for the second example, the draft was largely written and understood.

3.1 Base Isolated RC Shear Wall Apartment Building

3.1.1 Building Overview

The building is an eight storey (plus penthouse) central city apartment development, with a 395m2 typical floor plate, designed in 2015-2016. The gravity system uses 210mm Comflor on steel beams and columns, with the lateral force-resisting system being reinforced concrete internal shear walls. The isolation plane is located at ground level, with a 2.0m deep unoccupied basement. The isolation transfer grillage consists of a series of in-situ reinforced concrete beams, with the isolator bearings themselves sitting on short plinths above the raft slab foundation.

Steel braced-frame options were also investigated during Concept Design, however a concrete shear wall option was preferred in order to satisfy aspects of coordination with architectural, and improve acoustic transmission ratings. LDSD became a part of the design criteria following discussion around potential for a market to develop where post-earthquake functionality became a premium. The preference to use shear walls came from anticipating that these offered a better opportunity to set and maintain acoustic performance.

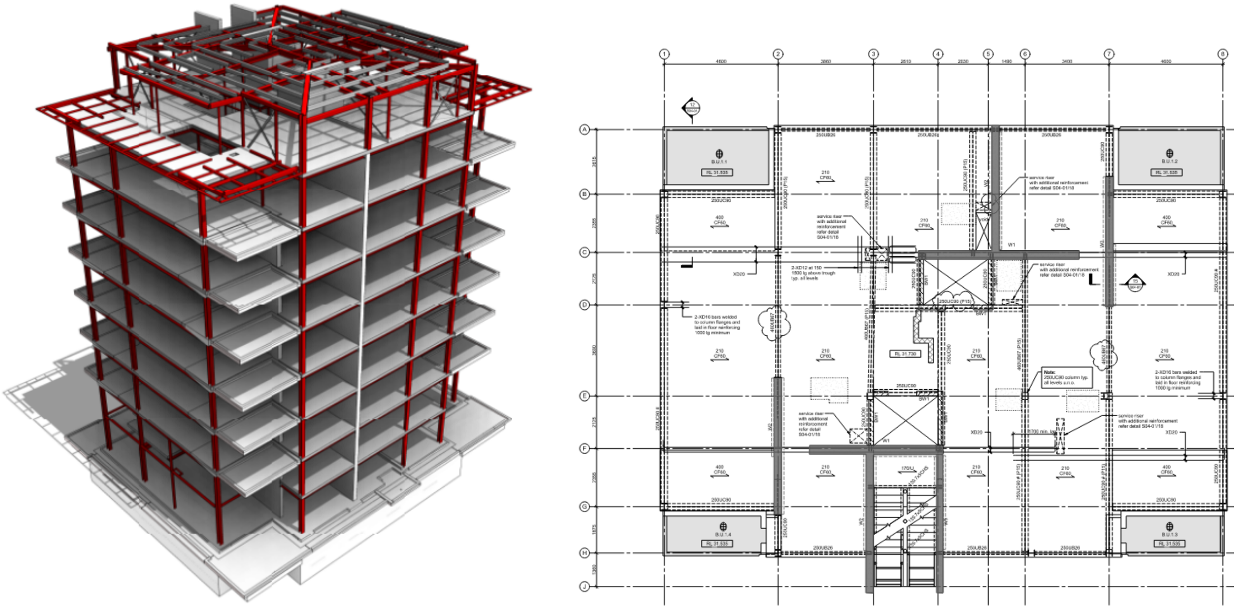

Figure 1. (a) 3D rendering of the apartment structure (b) typical structural framing plan.

3.1.2 Seismic Performance Targets and the Approach to Evaluation

Early discussion with the client identified a pathway to separating and demonstrating NZ Building Code (NZBC, 1992) minimum performance, from LDSD performance. The approach to demonstrating NZBC compliant performance (SLS1 and ULS superstructure design to $\mu = 1.25$, Importance Level 2) for Building Consent purposes, was based on an Alternative Solution via non-linear time history analysis, using minimum requirements per the New Zealand loading Standard (NZS1170.5:2004) with an envelope of three amplitude-scaled records, applied in two orientations (i.e. six analyses at each of four accidental eccentricities).

For this project the accepted approach evaluated low-damage performance targets used the mean response from a suite seven records (Figure 2, applied in two orientations for a total of 14 analysis at each of four accidental eccentricities) at the Damage Control Limit-State, being defined at the 500 year Return Period. The definition of the DCLS initially looked at both the 250 year and 500 year Return Period during Concept and Preliminary Design. The decision to proceed with these targets set at a 500 year Return Period was based on the early cost estimates, which indicated marginal additional cost to build to this higher performance.

While explicit evaluation of the recovery period was not possible in this project, it is notable that the client directive for design was essentially driven by their desire to promote a seismically resilient inner-city residential development.

With Code minimum performance evaluation capturing the aspects of strength and basic drift performance for SLS1 and ULS, the LDSD verification primarily targeted a maximum DCLS 0.5% storey drift. However, a further control was introduced to have the walls remain elastic for the DCLS earthquake (from NLTHA verification). No particular floor acceleration criteria were set, however the potential benefits of base isolation were demonstrated and accepted from early optioneering.

The decision to set the DCLS 500 year Return Period also brought mitigation of liquefaction effects into the design. The geotechnical assessment identified liquefaction induced settlements at the 500 year RP of around 150mm, and up to 50mm at SLS1. Due to a shallow gravel layer at 6m depth this was not expected to cause significant surface effects, however with the DCLS requirements in-place rammed aggregate pier (RAP) ground improvement to 6m depth was used to remove the potential for differential settlements in the shallow liquefaction region, and effectively create a 10m 'raft' over the deeper liquefiable layers.

Record #	Description	Scaling Ave of 7 k1 only
1	Chichi Taiwan CHY016 1999	2.81
2	Imperial Valley El Centro Array #9 5/19/1940	1.89
3	Landers Joshua Tree 06/2/92	1.43
4	729 Superstition Hills 02 Wildlife Liquef Array	0.65
5	Loma Prieta Woodside 1989	2.89
6	Chichi Taiwan TAP010 1999	1.77
7	Chichi Taiwan ILA055 1999	2.30

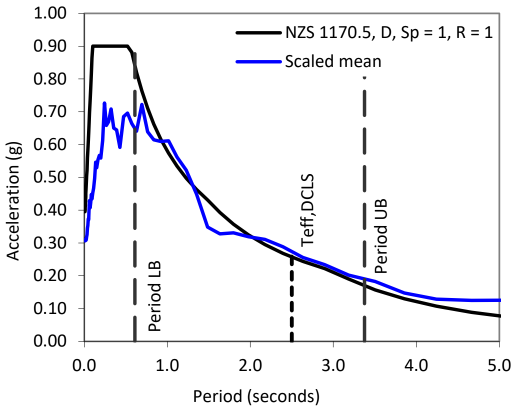

Figure 2. (a) Summary of 7 record suite used for DCLS evaluation (b) Mean scaled spectrum for DCLS 500 year RP including effective period $T_{eff,DCLS}$ = 2.5 sec and period scaling boundaries.

3.1.3 DCLS Performance

Figure 3 provides a summary of the building performance in each principal horizontal direction (at the centre-of-mass for clarity). Nominal isolation properties were used to evaluate the displacement and floor acceleration performance, while upper-bound properties were applied for the wall yield checks.

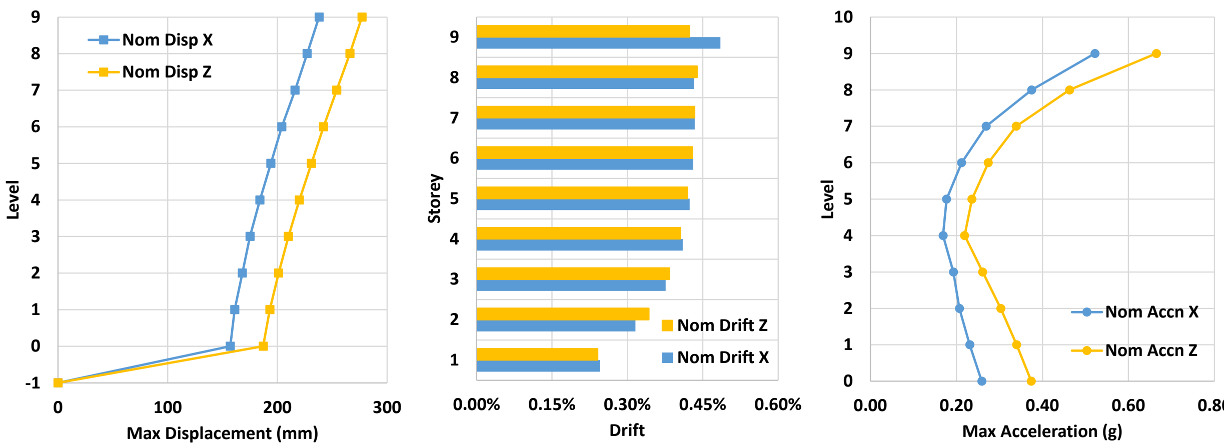

Figure 3. centre-of-mass average maximum response profiles (a) peak floor displacement (b) storey drift (c) peak floor acceleration

The key performance target of storey drift less than 0.5% was comfortably met (and similarly at the floor-plate corners this was the case), and the control of floor accelerations was notable, with only the penthouse level suffering accelerations significantly in excess of the isolation plane peak acceleration.

A subsequent study by Yang (2020) confirmed the ability of the building to meet the LDSD targets via a separate model development, along with more rigorous investigation of the potential annual loss, which was less than $NZ1000 (2020).

3.2 Base Isolated Steel MRF Office Building

3.2.1 Building Overview

A four-storey office building, the typical floor-plate is split into two seismically separated structures (west building and east building, floorplates 930m^2 and 980m^2 respectively, Figure 4) that project off a common isolation grillage at ground level. The west building has a lightweight penthouse level that is separate to the office tenancy and not subject to the LDSD performance specification for the building. The superstructure uses a limited-ductility (Category 3 per NZS3404:1997) two-way structural steel moment-frame, with flange-hung Double-T precast floor units and 100mm topping slab. The isolation grillage is also structural steel beams, connecting to fabricated steel nodes. The isolation bearings sit on round reinforced concrete basement columns, that cantilever from the 800 thick reinforced concrete raft slab. The basement is one storey deep and used for parking and housing of some services.

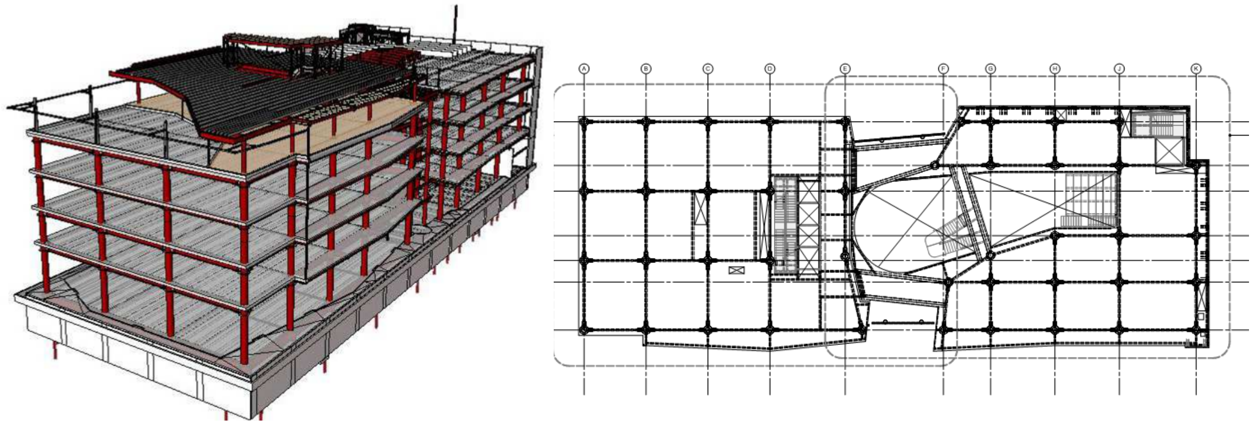

Figure 4. (a) 3D rendering of the office building (b) typical structural framing plan

3.2.2 Seismic Performance Targets and the Approach to Evaluation

The seismic design performance targets were primarily driven by a tenant issued building performance specification, that defined both a DCLS (250 year return-period), and a requirement for the building to meet 130% NBS. No aspect of the building usage formally defined it as needing to be Importance Level 3, therefore from the perspective of NZBC compliance and submission for Building Consent, minimum requirements for meeting IL2 performance were agreed to still be appropriate (SLS1 and ULS superstructure design to $\mu = 1.25$). In a similar manner to the apartment building case-study, compliance was based on an Alternative Solution via non-linear time history analysis, using minimum requirements per NZS1170.5:2004 with an envelope of three amplitude-scaled records, applied in two orientations (i.e. six analyses at each of four accidental eccentricities).

The client accepted approach to evaluating low-damage performance targets used the mean response from a suite seven records (Figure 5, applied in two orientations for a total of 14 analysis at each of four accidental eccentricities) at the Damage Control Limit-State, being defined at the 250 year Return Period. The requirement to meet 130% NBS was treated as a post-design assessment target, and as such while the isolation plane rattle-space was defined using ground motions scaled to a hazard factor R = 1.3, the superstructure and isolation grillage were themselves designed for base shears resulting from R = 1.0. Using verification via the NZSEE asssessment guidelines (NZSEE, 2017), it was possible to demonstrate acceptable performance of the superstructure when subjected to ground motions scaled R = 1.3.

A storey drift target of 0.5% was specified for the DCLS, however again no explicit floor acceleration values were specified.

With the ULS design based on μ = 1.25 Category 3 performance, the MRF performance at DCLS was implied to be elastic.

The definition of the DCLS for this project did not produce significant outcomes that affected the chosen foundation system. Liquefaction is expected at the site, however the presence of existing ~15m long driven reinforced concrete piles over the western half of the site, and the need to balance the vertical stiffness of these by adding screw-piles over the eastern half, meant that liquefaction was not expected to result in significant settlements under DCLS ground motions.

Associated to the structural performance specification for the building, there were post-disaster back-up systems also required such as water and power supply. The inclusion of these, while not defining a recovery period in themselves, were strong indicators as to what level of seismic demand the building itself was expected to support on-going post-event functionality.

Record #	Description	Scaling Ave SRSS 7 k1&k2
1	Superstition Hills Parachute Test Site 1987	0.49
2	Kobe KJMA 1995	0.53
3	Chichi TAP014 1999	1.74
4	Northridge Jensen Filter Plant 1994	0.40
5	Chichi Taiwan CHY002 1999	2.53
6	Chichi Taiwan CHY104 1999	1.71
7	Kobe Takarazuka 1995	0.53

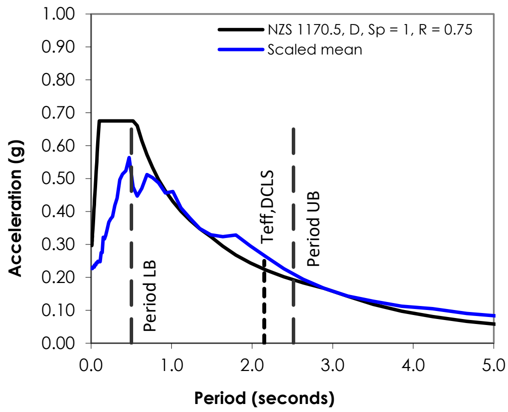

Figure 5. (a) Summary of 7 record suite used for DCLS evaluation (b) Mean scaled spectrum for DCLS 250 year RP including effective period $T_{eff,DCLS}$ = 2.15 sec and period scaling boundaries.

3.2.3 DCLS Performance

The results for the DCLS verification shown in Figure 6 demonstrate that the base isolation was effective in activating at this level of demand, and produced peak storey drifts less than 0.5% over the four office floors that were required to meet this target. It is notable that the drift profile shows the typical MRF form, with the largest deflection over the Ground Floor, and significantly lower response up the height of the building. Similarly the acceleration profiles show a much softer response when compared to the shear-wall apartment building, with essentially uniform maxima over the main floors. Again the penthouse roof has markedly amplified accelerations, which is due to changes in the LFRS that resulted in notable reductions in stiffness.

To assist with appropriately capturing Parts & Portions design accelerations, specific floor spectra were produced for DCLS and ULS. Initial tests of applying the guidance in the base isolation guidelines suggest that these need clarification based on further study, in-order to ensure that the resulting design parameters are capturing the likely parts response to the changing isolated building characteristics. Figure 7 provides an example of the difference in floor specific spectra, with those resulting from the guideline method.

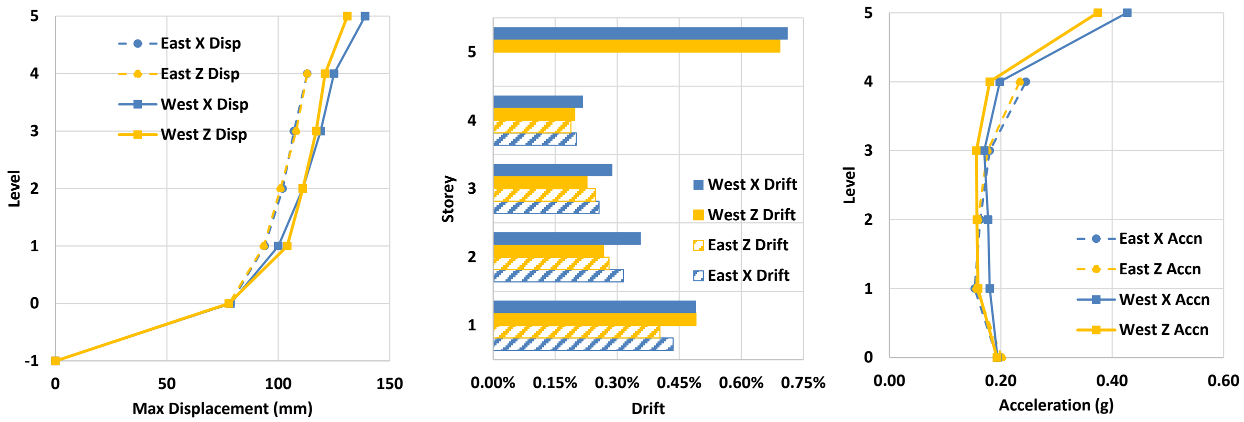

Figure 6. centre-of-mass average maximum response profiles (a) peak floor displacement (b) storey drift (c) peak floor acceleration

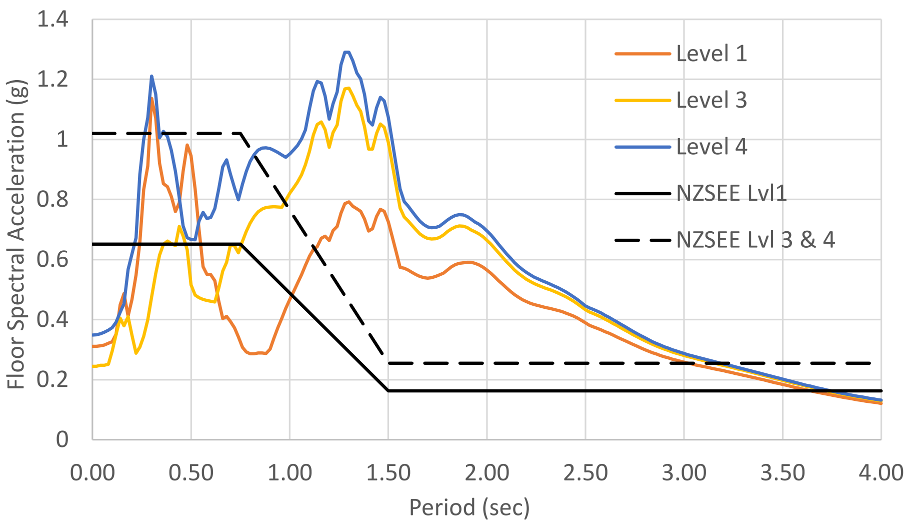

Figure 7. Comparison of ULS floor spectra produced from the NLTHA diaphragm centre-of-mass accelerations with design spectra produced following the NZSEE base isolation guidelines.

4 LEADING BY FUNCTIONAL RECOVERY

The two case-studies presented here provide qualitative examples of the value that clients, familiar with seismic performance concepts and an understanding of their desired post-earthquake scenarios, are placing on including low-damage seismic design in the project brief. While the apartment building discussed here arrived at a LDSD approach through discussion and optioneering during Concept Design, the office building was driven by a tenant supplied specification that clearly outlined their reasons and need for rapid return to using the building and its facilities.

The structural systems used in each case were chosen to suit the perceived governing building utility requirements, and then tailored towards low-damage seismic performance – in both cases with base isolation. Within the early design discussions on structural system, the connections between best suited structure for day-to-day function and the significance for this to be maintained following a moderate earthquake were made, and therefore lead the LDSD process.

What we see in these examples is that the introduction of Functional Recovery, or some equivalent terminology, to early design discussions greatly improves the ability of clients and/or tenants to identify their preferred seismic performance targets. It is logical therefore that this will be a key connection for New

Zealand to continue fostering as we look to motivate more low-damage developments both in research and practical application. Of equal interest will be case-studies where such considerations inform a decision to utilise non-seismically isolated building forms.

ACKNOWLEDGEMENT

The author would like to thank the anonymous reviewer, whose comments provided useful and meaningful feedback that improved the consistency and clarity of this paper.

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