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A BRIEF INVESTIGATION INTO ESTIMATING FLOOR SPECTRA FOR BASE ISOLATED BUILDINGS

D PETTINGA[1]; A THOMPSON[2]

[1] Holmes Consulting LP, Christchurch, New Zealand
[2] Holmes Consulting LP, Auckland, New Zealand

SUMMARY

A summary of the performance of the current guideline provisions for generating floor acceleration spectra in base isolated buildings is presented, along with comparative results from a selection of representative buildings. A potential new approach is proposed utilising research previously published for non-linear MDOF systems. The small study presented here shows positive improvements in the prediction of floor acceleration spectra for base isolated buildings.

INTRODUCTION

Following the Christchurch earthquake sequence the New Zealand structural engineering community has seen significantly more focus drawn to design and detailing of non-structural elements and their seismic restraint. The method to determining design accelerations for these so-called Parts & Components, as presented in NZS1170.5:2004 has also been under some scrutiny, however to-date there has been no amendment to the method. The general approach presented in the loadings Standard has been adopted in the recent NZ base isolation design guidelines (NZSEE, 2019), with a simple change that the peak ULS upper-bound base-shear coefficient at the isolation level is substituted for the C(0) factor presented in the Standard for non-base isolated structures.

Application of this adapted approach in recent projects, and comparison against specific generated floor spectra suggests that it underestimates the spectral accelerations over a wide range of Parts periods. Given the natural alignment of base isolated building design with the non-structural performance outlines associated to low-damage seismic design philosophy, there is an obvious need to address the circumstances where the base isolation guidelines are unconservative. More-so, there is a need to identify alternative approaches that can capture the particular aspects of floor spectrum shape that result from base isolation.

Using non-linear time history models that represent four case-study base isolated buildings, a review of the current draft guideline method for creating Parts design spectra is presented. From this comparison, a proposed alternative is presented for updating the current base isolation Parts & Components guidance, in-order to demonstrate a direction for appropriate estimates of design accelerations.

BACKGROUND TO THE CURRENT FLOOR SPECTRA PROVISIONS

The release of the NZSEE seismic isolation design guidelines (NZSEE, 2019) is New Zealand's first coordinated design guidance document for seismically base isolated structures. Beyond the basic inputs and approaches to the isolation and primary structure design, the guidelines include information about generating design actions for Parts & Components based on the same input as presented in NZS1170.5:2004. For developing floor spectra without specific floor acceleration records from non-linear response history analyses (NLRHA), the guidelines have adopted the NZS1170.5 Section 8 approach, with the simple adjustment that the site hazard coefficient C(0) is replaced by the peak design acceleration developed at the target limit-state, immediately above the isolation plane (further clarified in the commentary to be the ADRS design base shear, rather than the peak recorded acceleration of the slab immediately above the isolation plane):

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p) \tag{1}$$

A further aspect identified in the guidelines, is that the floor response should be evaluated under two conditions, one being with the isolation response at peak ULS displacement, and the other just prior to isolation 'yield' when the superstructure is essentially in a fixed-base condition (reflecting a response the same as NZS1170.5:2004, Section 8). Presumably this is achieved by reducing the hazard factor R until the fixed base value of $C(T_{fixed})$ is the same as isolation 'yield', thereby reducing C(0) in the same proportion.

The above procedure described by equation (1) is conceptually simple and familiar to New Zealand engineers, recent studies (for example Sullivan et al., 2013) have identified aspects of building response that are not well captured by the codified equations. Under-prediction of floor spectra by the NZS1170.5 method has been noted for both short and long period responses of fixed-base buildings exhibiting both short and moderately long period contributions to dynamic response.

While the above adjustment provided in equation (1) makes sense in the NZ code context, the same issues identified by Sullivan et al. (2013) could be expected to present themselves for base isolated buildings. At the same time no background is given for changing a NZS 1170.5 design ground acceleration for an effective damped spectral base shear coefficient. Similarly, the NZS1170.5:2004 method equations make no account for the modal properties of the building structure, particularly where period elongation effects could be expected to extend the first-mode inelastic period out beyond 0.75 seconds. Aside from floor spectra generated in practice for projects, there has been limited study into developing generalised floor spectra for seismically isolated buildings, which when combined with the noted short-comings of the method, make sense for us to look for a better representation that might be adopted into the NZSEE base isolation guidelines.

Base Isolated Building Response is Different

One of the primary goals of seismic base isolation is to reduce the floor accelerations over the height of a building, thus reducing the imposed seismic demands on the lateral force-resisting system. It is well understood what the benefits of this approach to enhanced seismic performance are, and that fundamentally the building response is different to that of a traditional fixed-base structure. A recent study by Calvi and Ruggiero (2017) provides discussion around floor accelerations in base isolated buildings, and some interesting insights into why floor acceleration spectra are not well predicted by current codified non-structural (Parts and Components) response methods in Eurocode 8 (CEN EC8, 2004) and ASCE 7-10.

A key aspect of base isolated building response that affects the shape of the floor acceleration spectra, is the significant fundamental mode period lengthening, that occurs as the building moves from its elastic "fixed-base" state through to full isolated response. If considering the Ultimate Limit State (ULS) response of a typical base isolated building, the effective period could be 3 to 4x the elastic period. The NZS1170.5:2004 approach is independent of the primary structure (forcing) period, and this is the reason that it often underpredicts the floor acceleration spectrum shape, when compared against NLTHA results. The complexity of the isolated period response moving from its elastic phase through a continuum of significant period lengthening, perhaps makes such response more difficult to capture in a simple method.

THE BUILDINGS CONSIDERED AND THE ANALYSIS APPROACH

To investigate alternative approaches to developing floor acceleration spectra for design in base isolated buildings, four 3D building models developed over the past five years for projects in New Zealand where considered. These models were selected because they represent four very different superstructure LFRS scenarios, and were designed or evaluated for ground motions representing 500 and 2500 year return periods.

The 3D NLTHA models were created using ANSR (Mondkar and Powell, 1979). As seen in the following figures, the models incorporated the primary lateral and gravity structure frame and/or wall elements. All models included rigid diaphragm assumptions for the floor plates used to obtain floor acceleration records. Node tracking of accelerations was used at the diaphragm centre, and one extreme corner of the diaphragm, although for brevity results presented here will focus on the response at the diaphragm centre.

Probable material strengths were used for the isolation grillage and superstructure elements, while nominal isolator properties were applied to the models.

Ground Motion Records

Suites of 11 recorded ground motions selected to represent the Christchurch ground conditions and hazard. The suites used, reflected the design return period of the building, and were conditioned on 2 second period response, being generally representative of the building's isolated response at ULS. Scaling was carried out in accordance with ASCE 7-16 Chapter 16 (2016), except that the scaling period range was specified according to the NZSEE (2019) base isolation guidelines.

Table 1. Summary of earthquake record suites used in the NLRHA

Building 1			**Building 2**	**Building 3**	**Building 4**
EQ and Station	Scale Factor	EQ and Station	Scale Factor	Scale Factor	Scale Factor
Erzincan, Turkey. Erzincan	1.13	Chi-Chi, Taiwan. ILA044	1.87	1.72	2.46
Northridge-01. Administrative Building	0.79	Kocaeli, Turkey. Duzce	0.77	0.96	1.29
Imperial Valley-06. Meloland Geo Array	1.01	Chi-Chi (aftershock 2), Taiwan. CHY002	2.60	1.95	2.45
Kobe, Japan. Amagasaki	1.87	Northridge-01. Administrative Building	0.38	0.42	0.57
Denali, Alaska. TAPS Pump Station #10	1.00	Superstition Hills-02. Parachute Test Site	0.42	0.47	0.66
Kobe, Japan. Takarazuka	1.55	Christchurch, New Zealand. Resthaven	0.42	0.58	0.75
Northridge-01. Sylmar Converter Stn	0.70	Parkfield-02, CA. Cholame 2WA	1.04	1.70	2.54
Northridge-01. Pardee SCE	1.96	Kobe, Japan. Takarazuka	0.51	0.73	1.05
Montenegro, Yugo. Ulcinj Hotel Olimpic	2.18	Chi-Chi, Taiwan. TAP014	1.68	1.97	2.85
Chi-Chi, Taiwan. TAP003	2.13	Kobe, Japan. KJMA	0.47	0.71	1.00

Chuetsu-oki. Kashiwazaki NPP Unit 1	0.83	Chi-Chi, Taiwan-03. CHY104	1.70	1.51	2.01

Building 1 – Nine-storey one-way steel MRF

This model represents a nine-storey building with one-way steel moment-resisting frames and composite floor system on steel gravity beams. The base isolation system uses a mix of lead-rubber bearing and flat PTFE friction sliders, that are connected with a steel beam transfer grillage. The superstructure comprises of three floors of podium on which three towers with seven additional storeys are supported. Figure 1 provides a summary of the building response properties at ULS.

Fixed-base T_{1x} & T_{1y}	1.24 s (X) & 1.26 s (Z)
Isolation T_i	0.89 s
ULS Isolation T_{eff}	3.3 s
ULS base-shear coeff	0.13g
ULS equiv visc. Damping ξ	32%
ULS centre-of-mass disp Δ	330 mm

Figure 1. 3D image of Building 1 showing general structural form.

Building 2 – Three storey perimeter reinforced concrete shear walls

This building model represents a three-storey reinforced concrete shear wall (punched wall) structure with internal reinforced concrete gravity structure. The isolation system is a mix of high-damping rubber bearings and high-damping rubber bearings with lead core (effectively an LRB). Of the buildings considered here, this is the only retrofitted isolation system, and it was originally design and installed in the mid-late 1990s.

Fixed-base T_{1x} & T_{1y}	0.16 s (X) & 0.14 s (Z)
Isolation T_i	0.76 s
ULS Isolation T_{eff}	1.9 s
ULS base-shear coeff	0.19g
ULS equiv visc. Damping ξ	23%
ULS centre-of-mass disp Δ	176 mm

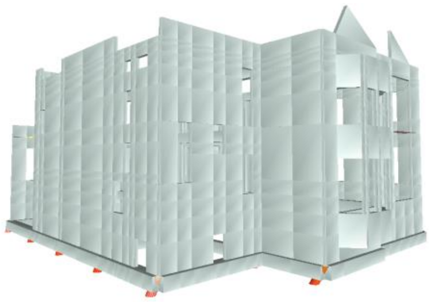

Figure 2. 3D image of Building 2 showing general structural form.

Building 3 – Four storey two-way steel MRF

Fixed-base T_{1x} & T_{1y}	1.14 s (X) & 1.06 s (Z)
Isolation T_i	0.87 s
ULS Isolation T_{eff}	2.5 s
ULS base-shear coeff	0.16g
ULS equiv visc. Damping ξ	27%
ULS centre-of-mass disp Δ	243 mm

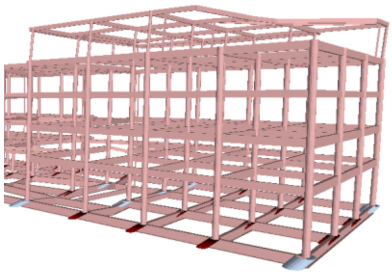

Figure 3. 3D image of Building 3 showing general structural form.

This model represents a four-storey two-way steel moment frame structure with lightweight penthouse. The base isolation system uses a mix of lead-rubber bearing and flat PTFE friction sliders, that are connected with a steel beam transfer grillage. The portion of the model shown in Figure 3 represents half of the building, which is actually two similar 'tower' structures (seismically separated) on a common isolation plan grillage. The image shown here clearly shows the increased flexibility of the penthouse structure with additional drift noticeable, compared to the main floors.

Building 4 – Nine storey reinforced concrete core-wall building

The final building model is a nine-storey reinforced concrete shear-wall structure with steel gravity frame. The isolation grillage is formed by reinforced concrete beams, which tie a hybrid LRB + flat PTFE friction slider system together.

Fixed-base T_{1x} & T_{1y}	0.81 s (X) & 0.70 s (Z)
Isolation T_i	0.77 s
ULS Isolation T_{eff}	2.1 s
ULS base-shear coeff	0.17
ULS equiv visc. Damping ξ	35%
ULS centre-of-mass disp Δ	186 mm

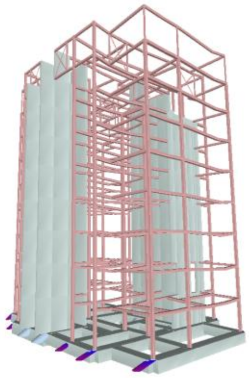

Figure 4. 3D image of Building 4 showing general structural form.

RESULTS

With acceleration response histories recorded at the diaphragm centre-of-mass at each level in the building models, the specific floor response spectra from each ground motion were processed for the two horizontal components (X and Z). The record suite average was calculated, and for a selection of floors over the height of each building model, the resulting floor acceleration spectra ("recorded spectra") are presented in Figure 5. The results considered here exclude lightweight penthouse levels, with the investigation focussing on the concrete slab diaphragms only.

Also shown in Figure 5 are the calculated Parts and Components spectra for the corresponding floor levels, using the NZSEE (2019) base isolation guideline (BIG) approach. The most noticeable aspect of this comparison with the recorded spectra, is that the current prediction equations consistently underestimate the spectral amplification associated to the base isolation period lengthening, that acts as the forcing function across all floor levels. The expanded period range of amplified response extends from the initial elastic phase response, through to the average effect period associated with the peak isolation plane response. This is consistent with the discussion of Calvi and Ruggerio (2017). Beyond that effective isolated period, the spectra gradually reduce in a manner consistent with traditional dynamic amplification considerations (Biggs, 1971; Sullivan et al., 2013).

Generally the prediction of short period response is more reasonable, although still missing the peak accelerations, particularly of the top floor. Although not shown, the lightweight penthouse response from Building 3 develops roof accelerations around 3g. This would not be picked up by the current guideline method, and highlights the influence of changing lateral system stiffness and the challenge this represents for estimate approaches that do not incorporate structural dynamic properties

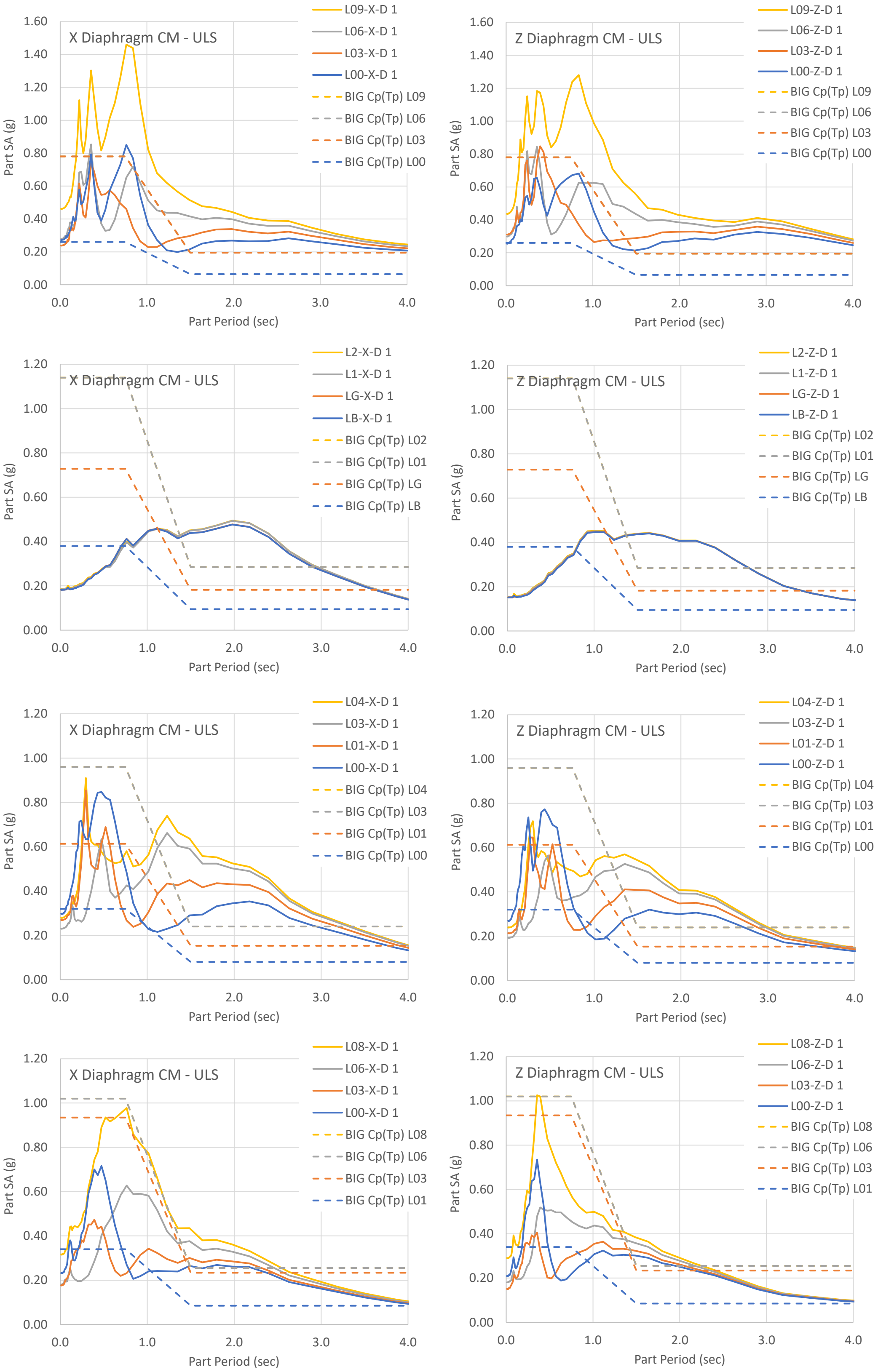

Figure 5. Processed floor acceleration spectra from NLTHA recorded acceleration traces at the diaphram centre-of-mass, including predicted spectra following the NZSEE seismic isolation guideline approach (a) Building 1 (b) Building 2 (c) Building 3 (d) Building 4.

OPPORTUNITY IN AN ALTERNATIVE APPROACH

The method of estimating floor spectra in non-linear MDOF systems by Welch and Sullivan (2017) has been applied to the same set of results. While this approach has been derived from a significant body of analytical results, and therefore is not strictly first-principles based, it does stem from prior studies (Sullivan et al., 2013) that directly draw from the work of Biggs (1971).

The reader is referred to the Welch & Sullivan (2017) publication for the full description of the method and the relevant equation numbers (denoted by "WS Eq.(x)") as referenced here. For brevity only the key assumptions and approach to generating the necessary inputs are discussed here. The method is fairly involved, although uses information that for base isolated buildings is readily at-hand during the design process. Given that this study is exploratory only, it seems reasonable to expect that a more in-depth study would identify reasonable and robust simplifications.

As discussed in more detail below, the main assumption in applying this method here is that the base isolation scheme reduces the total building response to a simple two degree of freedom scenario. This enables the response to be represented by the primary mode of movement on the isolation plane with essentially rigid-body superstructure, and a secondary mode of the superstructure responding in its fundamental fixed-based mode, of which the floor response is assumed reasonably representative of all floors above the 1st isolated floor.

The first key input required for WS Eq.(1) is the primary and non-structural damping. For this study the primary structure damping is the equivalent viscous damping of the base isolation response at the chosen limit state (ξ_{pb}). The non-structural damping (ξ_{NS}) is assumed to be 5%, though could be adjusted to suit recommendations for the Parts under consideration.

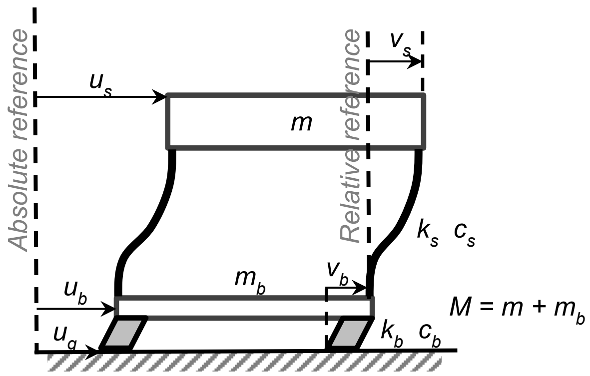

Figure 6. Two-degree-of-freedom representation of superstructure + isolation response

The next input required is an estimate of a Floor Response Spectra reduction factor (R_I), that accounts for the reduction in response from elastic to non-linear behaviour in the fundamental mode (i.e. the isolated mode). This has been provided in WS Eq.(3) as a function of ductility demand. For this investigation it seems reasonable to consider that typical base isolation systems will respond with an apparent "ductility" well in excess of the upper limit $\mu = 5$ presented in the method.

For this study it was considered appropriate to assume that the superstructure response is also reduced as a result of the isolation damping (consistent with standard base isolation design philosophy). Because the building system has been reduced to a two-mode response, the so-called higher mode response associated to WS Eq.(4) is in-fact the superstructure fixed-base fundamental mode. In this case, given it is a mode significantly affected by the isolation response it seems reasonable to assume that the response reduction R_{HM} (superstructure mode) = R_I (base isolation mode), using the RC wall form of the equation. Note however that the ductility associated to the superstructure (that affects period lengthening of the floor spectra) should reflect the superstructure response, which is often linear for base isolated buildings. In summary the response amplitude of the superstructure mode is reduced by the isolation reduction factor, but its period shift is associated to the superstructure ductility.

These reduction factors are applied in the calculation of peak modal floor acceleration ($a_{max,j,i}$), being WS Eq.(6). In this case because of the simplification to a two-degree-of-freedom system (Figure 6) can be carried out using the derivation presented by Christopoulos and Filiatrault (2006), which provides:

First mode shape $\{\emptyset^1\} = \begin{Bmatrix} 1 \\ \epsilon \end{Bmatrix}$ (2)

Second mode shape $\{\emptyset^2\} = \begin{Bmatrix} 1 \\ -\left(\frac{1-(1-\gamma)\epsilon}{\gamma}\right) \end{Bmatrix}$ (3)

requiring the ratio of base isolation frequency (ω_b) and superstructure frequency (ω_s): $\epsilon = \left(\frac{\omega_b}{\omega_s}\right)^2$ and the superstructure mass (m) to total system mass (M) ratio: $\gamma = \frac{m}{M}$. The participating mass in each mode can also be calculated from these simple parameters, although not shown here.

The above simplification enables the method of Welch and Sullivan (2017) to be applied through hand-calculation alone, using simple outputs from the base isolation design process (i.e. a 3D computer model is not necessarily required). The rest of the process as applied here follows from the outlined procedure in the referenced paper, including the recommendation in WS Eq.(9) to envelope the predicted floor spectra for the first isolated floor with the maximum of the calculated spectral floor acceleration and the ground motion elastic (5% damped) spectral acceleration.

When this method is applied against the recorded floor spectra from the four model buildings, the outcomes are generally improved from the current guidance method, although not necessarily showing consistent trends for over- or under-prediction (Figure 7). For direct comparison with the NLRHA ground motion intensity, the prediction curve (Figure 7, black solid) from WS Eq.(9), uses the average elastic spectrum ordinates of the scaled records (instead of the target design spectrum).

Generally the envelope from WS Eq.(9) captures the peak response of all floors well, however it tends to overpredict the first isolated floor response in the short period range. It is interesting to note that the dashed curve representing the superstructure effective first-mode mass consistently underpredicts the short period peaks for buildings 1, 3 and 4 (having similar fixed-base superstructure and isolated elastic periods). This suggests that the simplification represented in Figure 6 misses a key contribution of response from the fixed-base second mode. The extended range of amplification due to the shifting isolated mode effective period is well captured by the method, and a definite improvement from the current guideline approach. Although not shown in Figure 7, the penthouse roof acceleration spectrum for Building 3 and Building 4 exceed the prediction envelope. This aspect of response will need separate consideration and research.

Across the four buildings, the results suggest that the first isolated floor and top floor response could be adequately represented using the approach described here. Possibly the envelope provided by WS Eq.(9) could be considered applicable for the superstructure, particularly once diaphragm corners, accidental eccentricities and upper bound isolation properties are incorporated. Alternatively potential simple adjustment to capture the superstructure second-mode is to extend a horizontal line from the superstructure first-mode peak (dashed line) back to T = 0. Indicatively this would cap the short-period floor response on Buildings 1, 3 and 4.

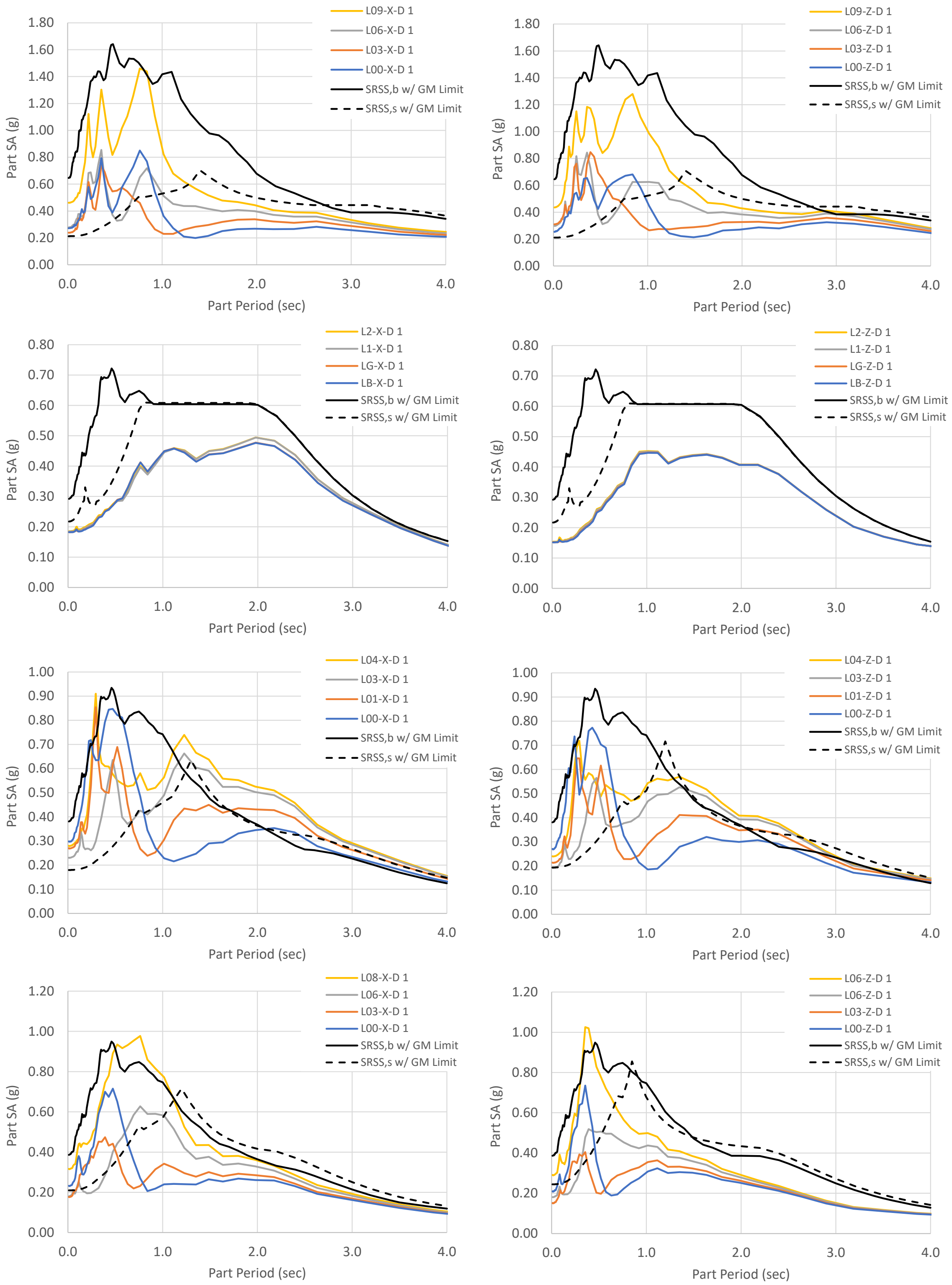

Figure 7. Processed floor acceleration spectra from NLTHA recorded acceleration traces at the diaphram centre-of-mass including prediction using the Welch & Sullivan method (2017) (a) Building 1 (b) Building 2 (c) Building 3 (d) Building 4. Solid black curve represents m_b response, and dashed black curve m response.

IN SUMMARY

A review of the current method presented in the NZSEE seismic isolation guidelines has been carried out using four case-study buildings. Using a suite of 11 records appropriately selected for each building and ULS design return period, the floor acceleration spectra at various levels over the height of the building have been developed from non-linear response history analyses. When comparing these against the method in the guidelines, it is clear that the amplification of the floor acceleration spectrum that occurs as the isolation system moves through a range of effective periods during the ground motion, is not captured.

A simple approach to adopting the method of generating floor acceleration spectra presented by Welch and Sullivan (2017) has been proposed here. Although more involved, the methodology is more intuitive as it works with the modal characteristics of the isolation system and the superstructure. It also accounts for non-linear response and associated equivalent viscous damping. Comparison of this proposed adaptation to the results from the non-linear response history analyses, show improved predictions over a wide range of Parts periods. Indicatively there is still an opportunity to improve the approach as applied to base isolated structures, particularly where superstructure fixed-base periods are close to isolated elastic periods, or penthouse or flexible roof structures are present.

REFERENCES

ASCE Standard ASCE/SEI 7-10 (2011) *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Virginia, USA, 2011

ASCE Standard ASCE/SEI 7-16 (2017) *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Reston VA, 2017.

Biggs, J.M. (1971) Seismic response spectra for equipment design in nuclear power plants, *Proc. of 1st International Conf. Structural Mechanics in Reactor Technology*, Paper K4/7, Germany, 1971

Calvi, P.M., Ruggiero, D.M. (2017) Earthquake-Induced Floor Accelerations in Base Isolated Structures, *Proc. 16th World Conf. on Earthquake Engineering*, Paper No. 105, Chile, 2017.

CEN EC8 (2004) *Eurocode 8 – Design provisions for earthquake resistant structures*, EN-1998-1:2004: E, Comite Europeen de Normalization, Belgium.

Christopoulos C., Filiatrault A. (2006) *Principles of Passive Supplemental Damping and Seismic Isolation*, IUSS Press, Pavia, Italy, 2006; 480.

Mondkar D.P, Powell G.H. (1979) *ANSR II Analysis of Non-linear Structural Response User's Manual*, EERC 79/17, University of California, Berkeley, CA, USA, 1979.

New Zealand Standard NZS1170.5:2004 (2004) *Structural Design Actions Part 5: Earthquake Actions – New Zealand incl. Amendments 1-3*, Standards New Zealand, Wellington

NZSEE (2019) *Guideline for the Design of Seismic Isolation Systems for Buildings – Draft for Trial Use*, pp 162, New Zealand Society for Earthquake Engineering, New Zealand.

Sullivan, T.J., Calvi, P.M., Nascimbene, R. (2013) Towards improved floor spectra estimates for seismic design, *Earthquakes and Structures*, Vol. 4, No. 1, 2013, pp 109-132.

Welch, D.P., Sullivan, T.J. (2017) Illustrating a new possibility for the estimation of floor spectra in nonlinear multi-degree of freedom systems, *Proc. 16th World Conf. on Earthquake Engineering*, Paper No. 2632, Chile, 2017.