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SIMPLIFIED RELATIONSHIPS BETWEEN INELASTIC AND ELASTIC SPECTRAL ACCELERATION DEMANDS FOR SEISMIC DESIGN IN NEW ZEALAND

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ABSTRACT

This paper investigates the suitability of the equal displacement approximation for short period structures, noting that historic definitions of what constitutes a "short period structure" are vague. The research identifies a new parameter, T_{peak}, defined as the period ordinate for the peak elastic spectral acceleration, to provide a lower-bound period for which the equal displacement approximation may be appropriate. The use of this parameter is supported by the results of inelastic spectra and reduction ratios generated for SDOF systems, with bilinear and Takeda hysteresis rules, subject to a large suite of ground-motion records. New equations for strength reduction factors are provided as a function of T_{peak} and agree well with strength reduction ratios obtained from inelastic spectra for the ground-motion set. It is found that a "short period structure" should be defined as those with periods greater than 0.2 s-0.3 s for most sites and locations in New Zealand, at a ground shaking intensity with annual probability of exceedance of 1 in 500. The study then goes on to conclude that use of the equal displacement approximation for all periods may be reasonable, at least for the immediate future, given that (i) many buildings in New Zealand are likely to have a period of vibration greater than the short period limit, particularly when foundation deformations are considered, and (ii) recent literature suggests that the design ductility currently adopted for ductile short period buildings may be conservative. Nevertheless, if future research prompts revisions to ductility and deformation capacity estimates for short period buildings, then the authors advocate for a return to period-based relationships between inelastic and elastic spectral demands.

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INTRODUCTION

The recent release of the New Zealand (NZ) National Seismic Hazard Model (NSHM) has prompted re-examination of the NZ seismic design provisions, NZS 1170.5 (Standards New Zealand, 2004a). A Seismic Risk Work Group (SRWG) has been established to propose updates to B1/VM1 of the NZ Building Code through a multi-year project that seeks to modernise several NZ standards, including NZS 1170.5 (2004a), which is to be superseded by the release of a proposed Technical Specification (TS) document, TS 1170.5. TS 1170.5 incorporates the latest NSHM and includes an updated method for computing the horizontal design action coefficient which permits the use of the equal displacement approximation for all structural periods to relate the inelastic and elastic spectral demands. This is despite a large amount of literature that suggests the use of the equal displacement approximation is not applicable for short period structures and can be somewhat non-conservative for medium to long periods. Justification for the use of the equal displacement approximation at all periods is examined and alternative approaches for implementing strength reduction factors in future code provisions are proposed.

Review of Research into Inelastic Spectral Demands

Relationships between inelastic and elastic spectral demands have been widely researched, including pioneering works of Veletsos & Newmark (1960) as well as more comprehensive studies such as those of Miranda & Ruiz-Garcia (2002) and Stafford et al. (2016). Early research indicated that the equal displacement approximation is applicable for medium and long-period structures whereas at short periods, alternative relationships should be adopted, such as the equal-energy rule (Newmark & Hall, 1982). Research conducted over the past two decades, considering a wider range of hysteretic models and improved modelling of elastic damping for structures responding non-linearly, has generally found the equal displacement approximation to underestimate the displacement demands at medium and long-periods of vibration (see, for example, Miranda and Ruiz-Garcia (2002) and Priestley et al. (2007)). Furthermore, the inelastic-to-elastic demand relationship is affected by earthquake magnitude and distance, as pointed out by Miranda (2000) and Chopra & Chintanapakdee (2004), with Tothong & Cornell (2006) and Stafford et al. (2016) advocating the use of inelastic-to-elastic demand relationships such that inelastic displacement demands are directly considered within the Probabilistic Seismic Hazard Analysis (PSHA) methodology.

However, there are a number of factors influencing the overall seismic risk of buildings and thus, in order to maintain consistency with previous loadings standards and for simplicity, the SRWG has recommended the use of the equal displacement approximation at all periods. This paper investigates and discusses the basis of this approach, as well as a simplified alternative.

Review of Strength Requirements from NZS 1170.5 (2004)

In NZS 1170.5 (Standards New Zealand, 2004a), the horizontal design action coefficient provides the required strength (via multiplication with the building weight) of potential plastic hinge regions in a structure to limit inelastic deformation demands to acceptable levels. The coefficient is set as a function of the structural ductility factor, μ, (representative of the ductility capacity), the structural performance factor, S_p, and the elastic spectral acceleration demand at the period of the building in the direction considered C(T_1). A minimum value for the design action coefficient is also prescribed. The design

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action coefficient is specified in NZS 1170.5 (Standards New Zealand, 2004a) as:

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu} \quad (1)$$

where k_μ is the structural ductility factor that is a function of the building period in the direction considered, T_1, structural ductility, and site class and is given for site classes A, B, C and D as:

$$k_\mu = \mu \qquad \text{for } T_1 \geq 0.7 \text{ s} \quad (2)$$

$$= \frac{(\mu - 1)T_1}{0.7} + 1 \qquad \text{for } T_1 < 0.7 \text{ s}$$

and for site class E as:

$$k_\mu = \mu \qquad \text{for } T_1 \geq 1 \text{ s or } \mu < 1.5 \quad (3)$$

$$= \frac{(\mu - 1)T_1}{0.7} + 1 \qquad \text{for } T_1 < 1 \text{ s and } \mu \geq 1.5$$

where T_1 need not be taken less than 0.4s according to amendment 1 of NZS1170.5 from 2016. It is generally considered that the equal energy rule better represents demands for short period structures, with the equal displacement approximation being more applicable for medium and long-periods. This is the reflected in the formulation of Equations (2) and (3). The commentary of NZS 1170.5 (Standards New Zealand, 2004b) notes that for periods between 0.35 s than 0.7 s the formulation provides a non-conservative estimate of the inelastic response but also goes on to say that as there are other inaccuracies in the derivation of inelastic response and to be consistent with other international standards, the approach was deemed suitable for the NZS 1170.5 (2004a) provisions.

Strength Requirements in the Proposed TS 1170.5

To further simplify the approach of NZS 1170.5 (Standards New Zealand, 2004a), the proposed TS recommends that the design action coefficient be reduced by the inelastic spectrum scaling factor, k_μ, which has been defined in the proposed TS as being equal to the structural ductility factor, μ, in line with the equal displacement approximation. The equal displacement approximation is prescribed at all periods of vibration, even though the expression is traditionally applied only for medium and long-period structures, such that the proposed structural ductility factor is given by:

$$k_\mu = \mu \quad (4)$$

The proposed TS also advocates for the continued use of Equation (1) to set horizontal design actions with reductions allowed through the use of the structural performance factor that is a function of the design ductility.

EXAMINATION OF STRENGTH REDUCTION FACTORS RELATIVE TO A PEAK PERIOD

Identification of a Peak Period, T_{peak}

Historically, the use of the equal displacement approximation has been justified for medium and long period structures with alternative approaches, such as equal energy, used at short periods. However, the definition of what is a "short period structure" is subjective and the current literature, including NZS 1170.5 commentary (Standards New Zealand, 2004b), suggests there is a transition zone between equal energy and equal displacement approximation which adds complication when seeking simplistic code implementation procedures.

To identify the period of vibration below which the equal displacement approximation might be deemed non-conservative, this paper develops inelastic spectra for sets of ground-motions recorded at sites with different V_{s30} values and then identified a peak-period, T_{peak}, corresponding to the period at which the maximum spectral acceleration, $S_{a,max}$, occurs. Figure 1 illustrates identification of the peak period from the elastic acceleration response spectrum of a single ground-motion. By generating inelastic spectra for a larger set of ground-motions and normalizing the period of vibration of each response spectrum by the peak period (to give T/T_{peak}), it is proposed that T_{peak} represents a reasonable lower-bound limit to the period from which the equal displacement approximation should be applied.

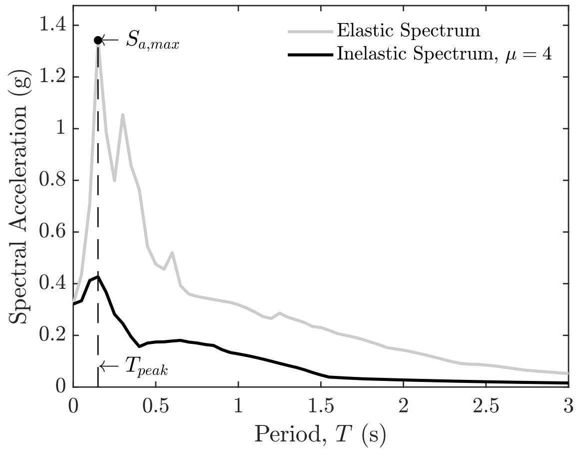

Figure 1: Illustrating the identification of the peak period of vibration for a ground-motion.

Figure 1 implies that the inelastic spectral acceleration demands remain relatively constant out to a period corresponding to T_{peak} and then reduce by a factor roughly in agreement with the equal displacement approximation (with caveats previously mentioned where research has shown the equal displacement approximation to be non-conservative even at medium and long periods). Given this observation, a new approach for defining strength reduction factors is proposed and investigated with the general equation characterised by a linearly increasing strength reduction factor for $T < T_{peak}$, and constant reduction factor in line with the equal displacement approximation for $T \geq T_{peak}$:

$$k_\mu = \mu \qquad \text{for } T_1 \geq T_{peak} \quad (5)$$

$$= \frac{(\mu - 1)T_1}{T_{peak}} + 1 \qquad \text{for } T_1 < T_{peak}$$

The ability of Equation (5) to represent observed reduction factors from a set of recorded ground-motions is discussed later in this paper along with the suggestions for implementation into future design provisions.

Numerical Modelling Approach

To evaluate the suitability of T_{peak} to represent a lower bound for which the equal displacement approximation is applicable, the inelastic spectra software *INSPECT* (Carr, 2016) was used to generate inelastic spectra for ductility ratios, μ, of 1.5, 2, and 4 for a suite of ground-motions that cover a range of ground-motion characteristics and provide evidence that the use of T_{peak}

is justified despite variations in site class, rupture distance (R_{rup}), and earthquake moment magnitude (M_w). Two single degree of freedom (SDOF) oscillators that are commonly used to represent a wide range of structural typologies were used. Figure 2(a) shows the bi-linear model that is generally considered to be representative of the hysteretic behaviour of steel or base isolated structures. Figure 2(b) shows the Takeda model (Otani, 1981) with Emori-Schonbrich (1978) unloading that is commonly used to represent reinforced concrete (RC) structures and bridges. *INSPECT* generates spectra for a specific ductility demand by undertaking consecutive nonlinear time-history analyses (NLTHA) and iterating the yield force, F_y, of the SDOF oscillator until the target displacement ductility is achieved within a prescribed tolerance.

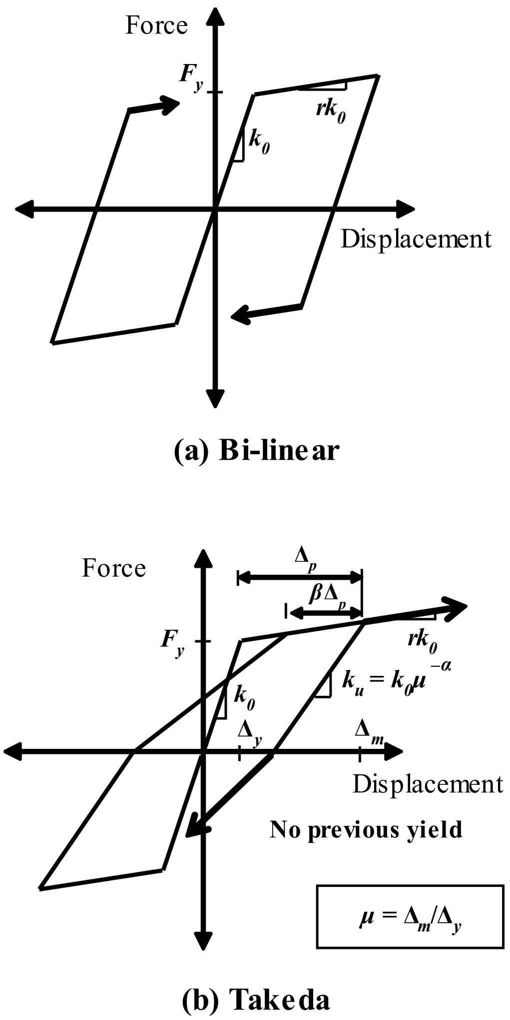

Figure 2: Hysteretic models adopted for the generation of inelastic response spectra.

The NLTHA integration time-step was set to be 1/10 times the time-step of the input ground-motion with 5% equivalent viscous damping modelled using the constant secant damping approach (ICTYPE = 6 in Ruaumoko, Carr 2023). No strength degradation was modelled. The hysteretic parameters required to fully define the hysteretic models are shown in Table 1 which align with the values proposed by Stafford et al. (2016) for the bi-linear and Takeda hysteresis rules. The post-yield stiffness ratio, r, was taken as 0.05 for both models with the unloading parameter, alpha, and reloading parameter, beta, the only additional parameters needed to fully define the Takeda hysteresis rule.

Table 1: Hysteretic parameters used to define the models in Figure 2.

Hysteretic Model	Parameters
Bi-linear	r = 0.05
Takeda	r = 0.05, α = 0.5, β = 0.0

Ground-Motion Selection

Ground-motions were selected from the Pacific Earthquake Engineering Research Center's (PEER) Next Generation Attenuation (NGA) West 2 database. Six sets of 81 to 100 ground-motions consistent with each of the proposed TS 1170.5 site classifications, I – VI, were considered while site class VII was omitted due to the rarity of such sites and the necessity for site-specific hazard assessments for extremely soft soil conditions. Where more than 100 applicable ground-motions were available in the data base, they were selected in ascending order of Record Sequence Number (RSN). Site class bins, as a function of $V_{s,30}$, are outlined in Table 2 and align with the values provided in the proposed TS (Lee et al., 2024). Ground-motions were taken as-recorded with no scaling or rotating and each orthogonal, horizontal component was analysed individually. Figure 4 outlines the rupture distance – moment magnitude relationships as a function of site class and faulting mechanism for all ground-motion records selected. A wide range of magnitudes and rupture distances, as well as several different faulting mechanisms, are represented for each site class bin. The intent of this research is therefore to examine the suitability of selecting T_{peak} as a lower bound for the equal displacement approximation for a broad range of ground-motions instead of critically examining how faulting mechanisms, rupture distance, magnitude, or hysteretic behaviour impact reduction ratios which has already been reported in the literature (Chopra & Chintanapakdee, 2004; Miranda, 2000).

Table 2: Proposed TS 1170.5 site classifications and corresponding $V_{s,30}$ range.

Site Classification	$V_{s,30}$ range (m/s)
I	750 – 2000
II	450 – 750
III	300 – 450
IV	250 – 300
V	200 – 250
VI	150 – 200

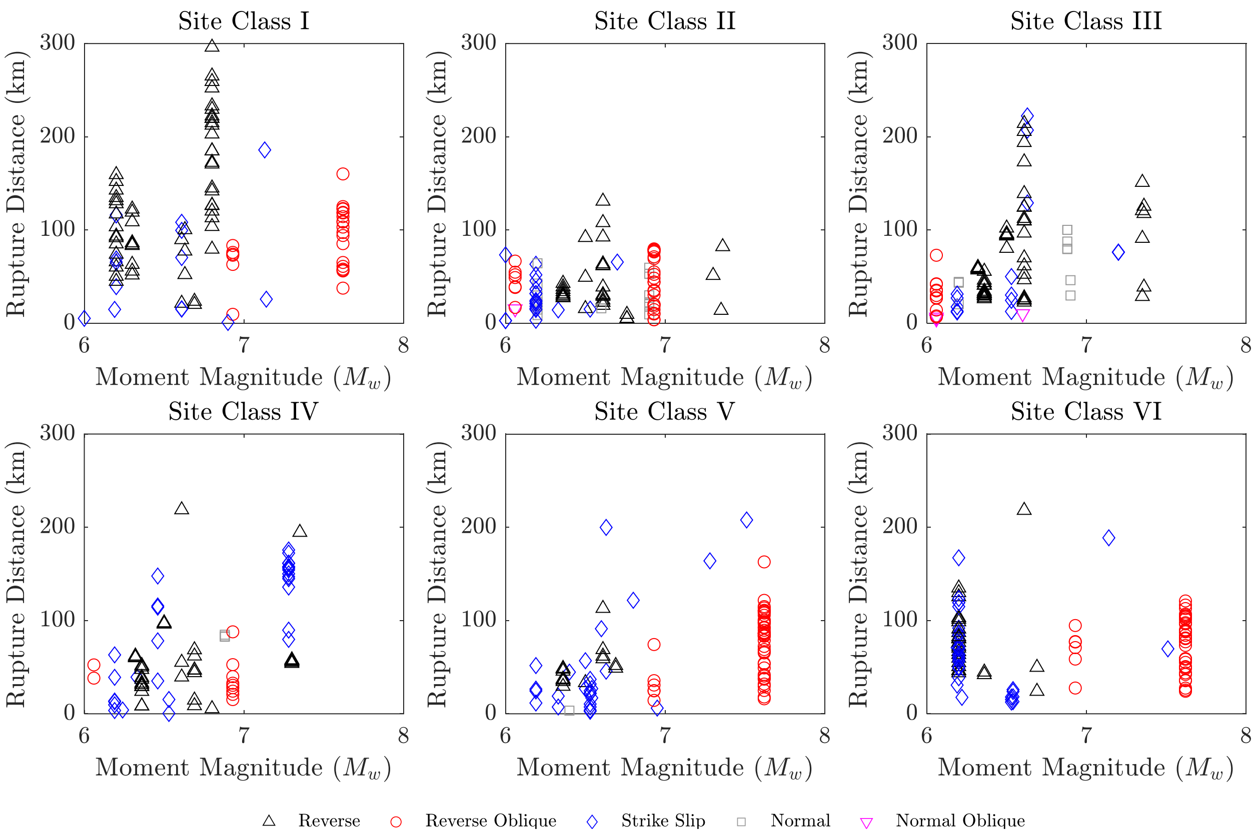

Figure 3: Rupture distance and moment magnitude as a function of site class and faulting mechanism for the selected ground-motions.

INELASTIC SPECTRA RESULTS

The elastic and inelastic spectra generated using *INSPECT* are shown in Figure 4 and Figure 5 for the bilinear and Takeda hysteresis models respectively, for a ductility ratio of $\mu = 4$. The spectra were normalised by *PGA* to highlight the spectral shape and allow comparison between records. Moreover, median spectra were computed for both the elastic and inelastic set of results. The median elastic spectrum was then divided by the ductility ratio to represent the inelastic demands that would be expected when applying the equal displacement approximation at all periods, as considered by the SRWG in the proposed TS. Comparison of the predicted inelastic demand using the equal displacement approximation with the actual median of the inelastic spectra appears to indicate that for longer periods (i.e., T greater than one second) the equal displacement approximation is reasonable regardless of site classification. This result agrees well with the literature. However, for shorter periods, the equal displacement approximation results in an underrepresentation of the demands with increasingly non-conservative results as the period tends towards zero. At a period ordinate of $T = 0$ s, the proposed TS appears to indicate that the *PGA* is reduced by the ductility factor, but such structures should be designed using the unfactored *PGA* directly from the NSHM. Normal buildings are, however, not expected to have very short periods, as discussed later.

The results in Figures 4 and 5 indicate that site classification has a limited impact on the appropriateness of the equal displacement assumption. However, it can generally be seen that for the soft soil classes of V and VI the rule is less suitable compared to firmer sites, I – IV. Similar trends were also observed for ductility ratios of 1.5 and 2. However, to maintain conciseness the figures are not reproduced in this paper.

To examine the spectral shape more closely, Figure 4 is reproduced in Figure 6 with the x-axis normalised with respect to $T_{peak.}$ Again, the median elastic and inelastic spectra are plotted for a ductility ratio $\mu = 4$. Furthermore, the predicted inelastic demands considering the equal displacement approximation and Equation (5) are compared. It can be seen that the equal displacement approximation appears to provide a non-conservative strength estimate compared to the median inelastic spectra. In comparison, Equation (5) reasonably predicts the demands and, importantly, reflects that as the period tends towards zero, the inelastic demands tend towards the *PGA* directly obtained from the NSHM uniform hazard spectra (UHS).

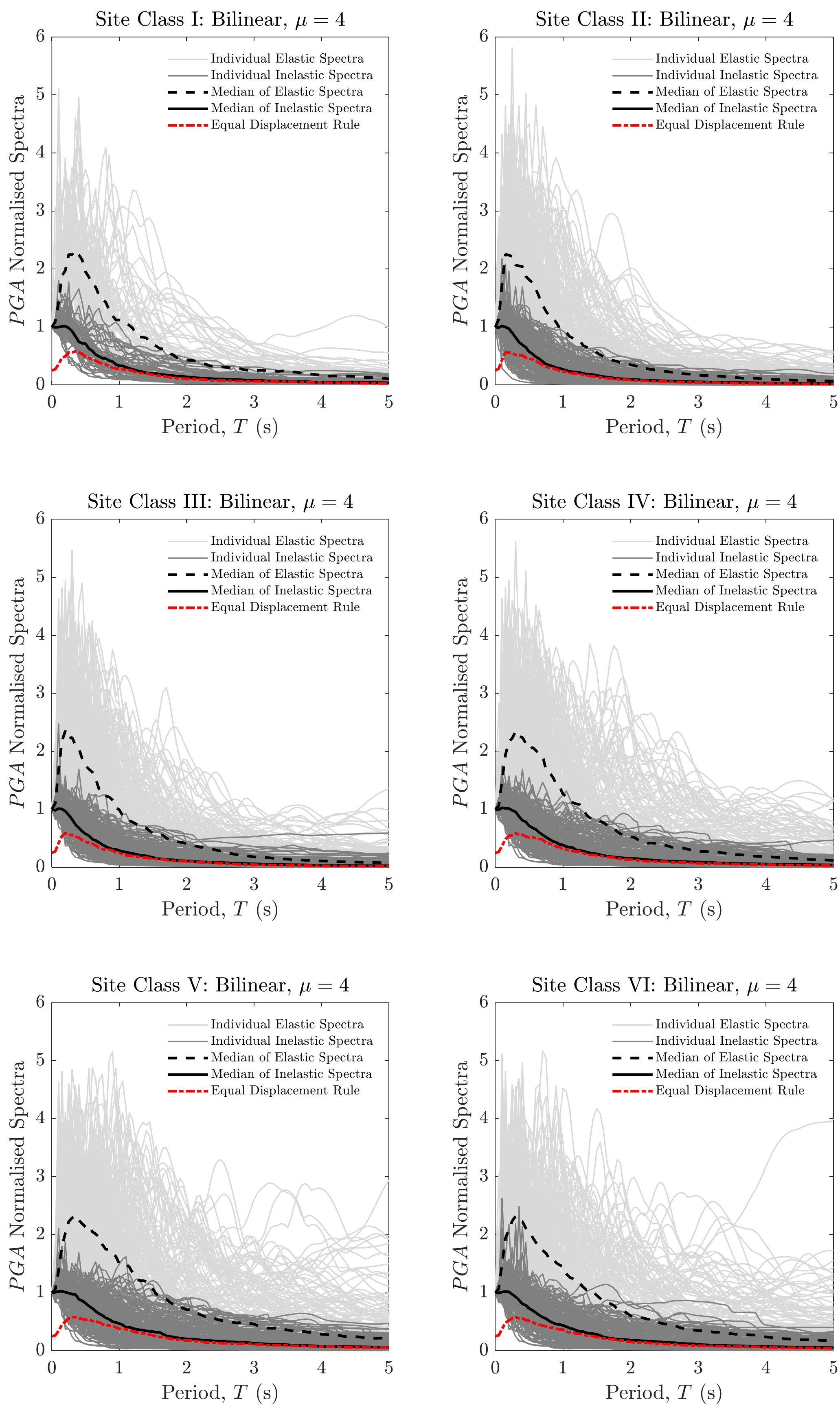

Figure 4: Elastic and inelastic spectra generated from the ground-motion set for all site classes considering the bilinear hysteresis rule and ductility demand of μ = 4.

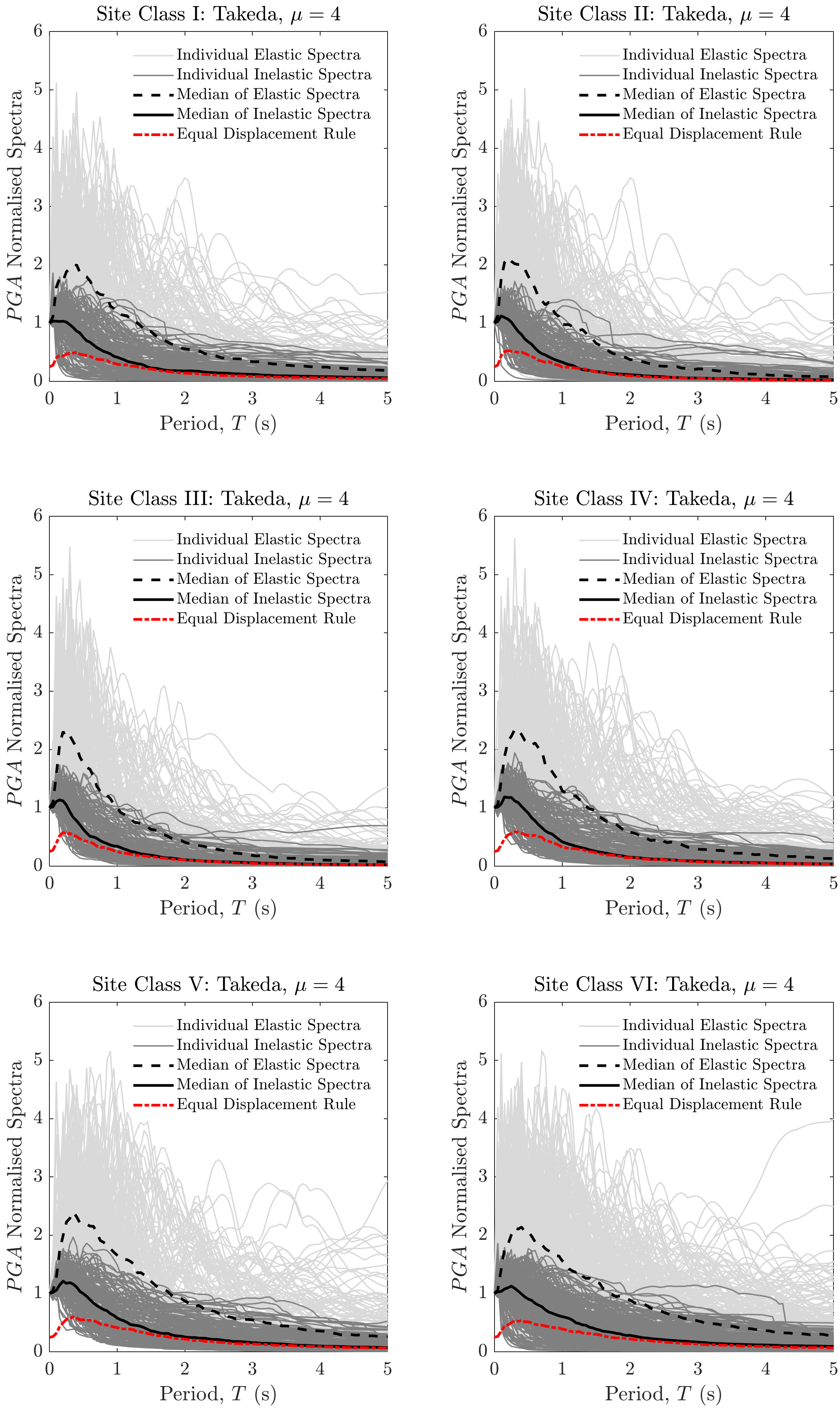

Figure 5: Elastic and inelastic spectra generated from the ground-motion set for all site classes considering the Takeda hysteresis rule and ductility demand of μ = 4.

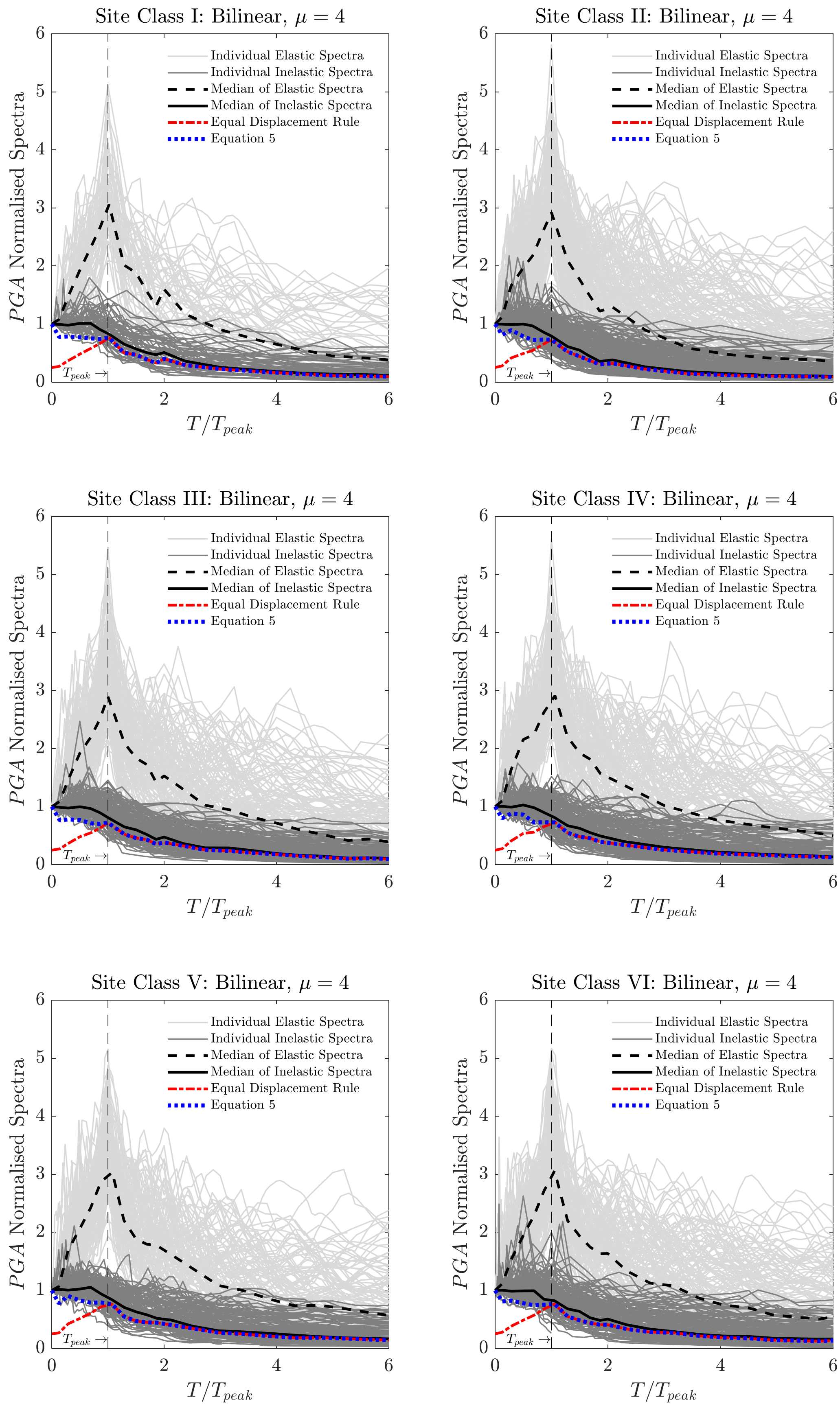

Figure 6: Elastic and inelastic spectra generated from the ground-motion set for all site classes with period normalised by T_{peak} considering the bilienar hysteresis rule and ductility demand of μ = 4.

FORCE REDUCTION FACTORS

Median Reduction Factors

Another approach for considering the inelastic spectra data is to compute force reduction factors that represent the elastic to inelastic spectra ratio for each ground-motion. The resulting force reduction factors for individual records are shown in Figure 7 to Figure 9. Additionally, for a given site class, ductility ratio, and hysteretic model, the median force reduction factor, θ, was computed by obtaining the median value of data bins with a bin width corresponding to the square root of the total number of T/T_{peak} ratios considered for all ground motions in the site class set. The bin width was fixed to 0.2 for $T/T_{peak} < 2$ so that a consistent bin spacing was enforced in the period range of interest around T_{peak}. The median reduction factor is compared to the proposed TS reduction factor (i.e., $k_{\mu} = \mu$) in Figure 7 and Figure 8, for $\mu = 4$ for all TS site classes and the bilinear and Takeda hysteresis rules respectively. The proposed TS reduction factor appears to be reasonable for periods greater than T_{peak} but is non-conservative for short periods. The median reduction factor appears to align well with Equation (5) which suggests T_{peak} is a suitable parameter for defining what constitutes a short period as evidenced by the abrupt change in slope of the median inelastic spectra at the T_{peak} ordinate. Furthermore, Figure 9 evaluates reduction factors as a function of ductility for the bilinear and Takeda hysteretic models. This highlights that Equation (5) provides a reasonable representation of inelastic demands for a range of site classes, hysteretic models and ductility demands.

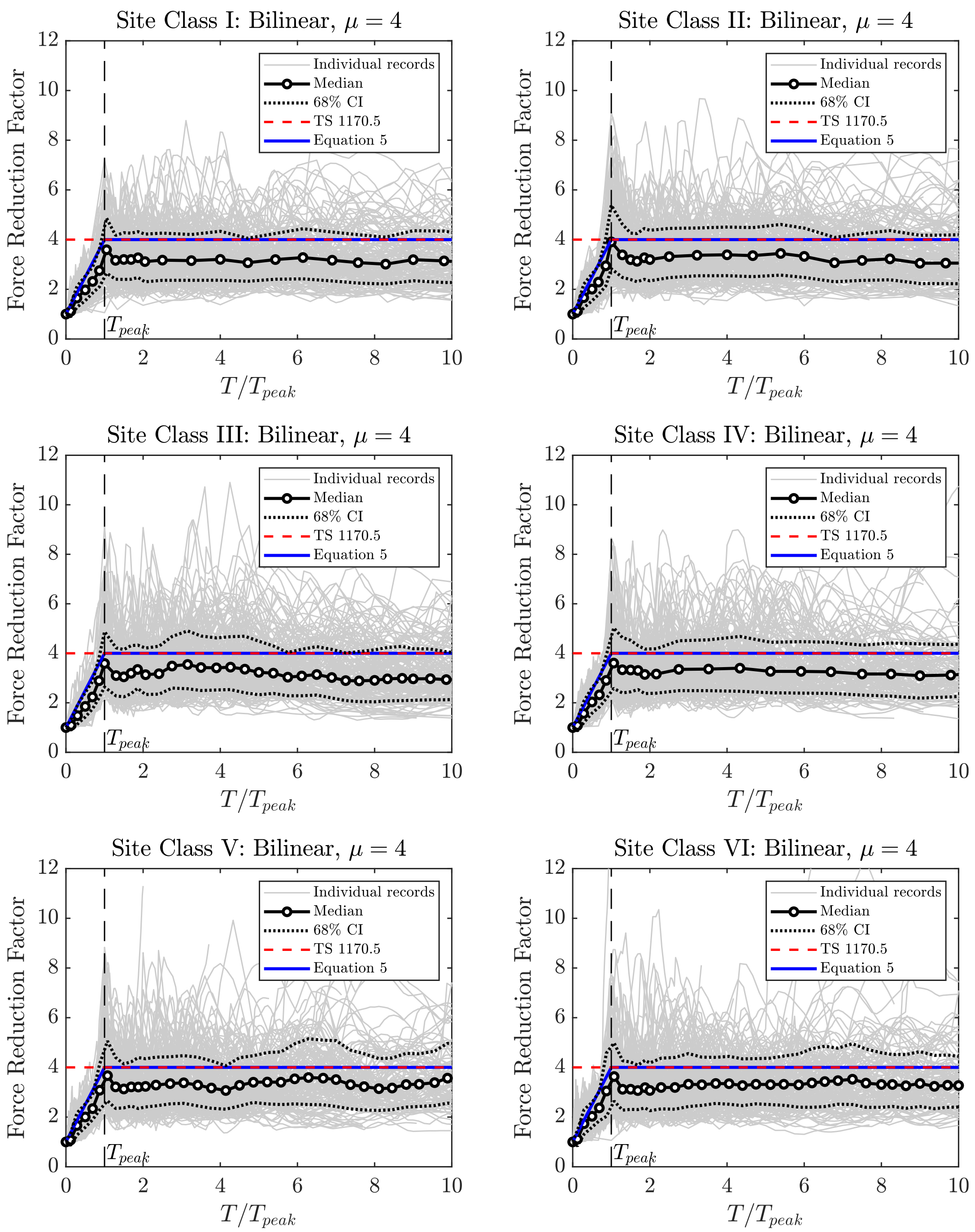

Figure 7: Reduction factors generated for the bilinear hysteresis rule and ductility demand of μ = 4 for all site classes.

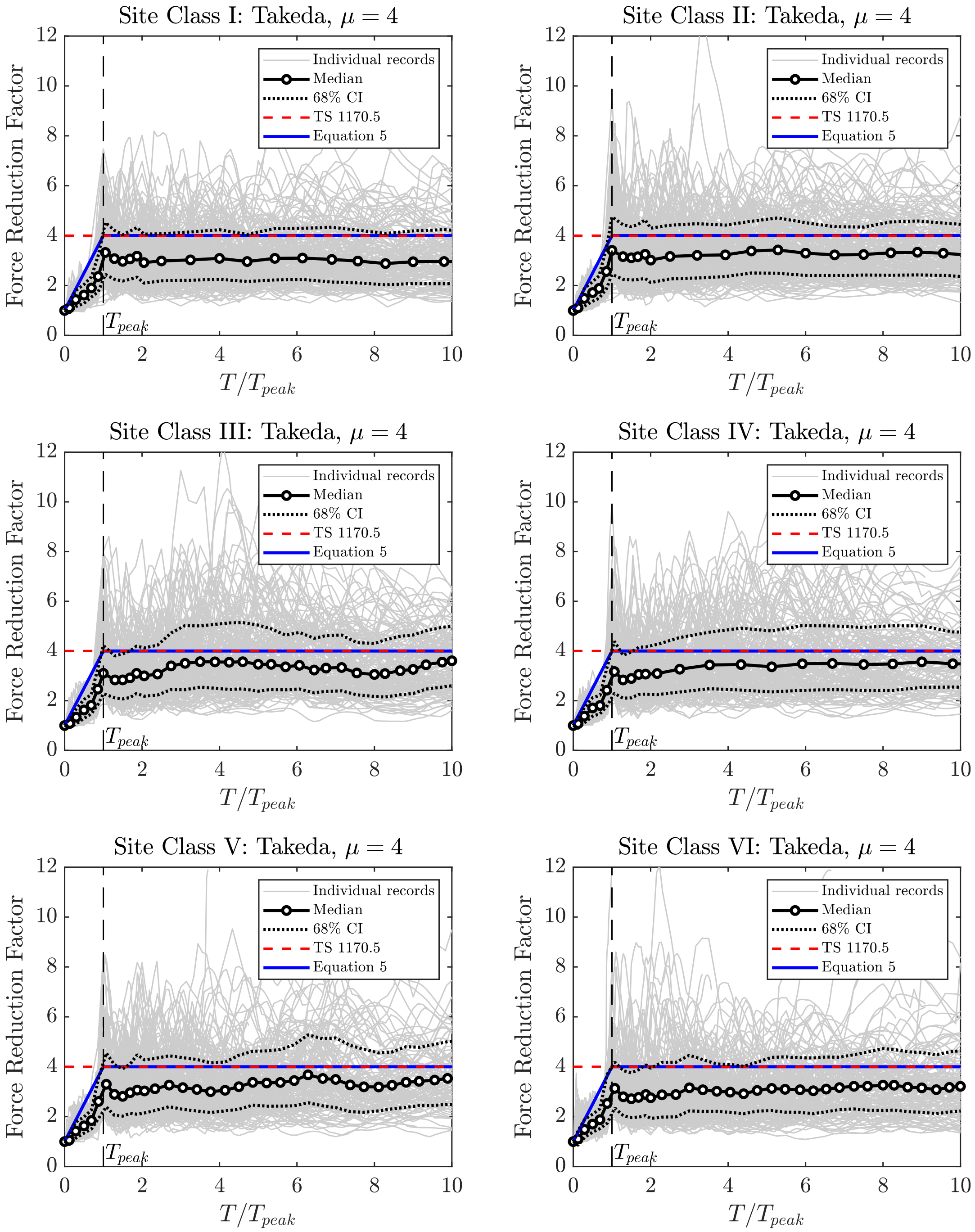

Figure 8: Reduction factors generated for the Takeda hysteresis rule and ductility demand of μ = 4 for all site classes.

Figure 9 shows that as ductility demands increase, the equal displacement rule becomes increasingly non-conservative, but the rule is simple and has been in use for some time. As such, Equation (5) is considered to provide a convenient approach for code implementation as only one equation is required to define the strength reduction factor for a range of building types and soil conditions.

Reduction Factor Variability

It is important to note the significant variability of design forces obtained using the equal displacement approximation. To assess this variability, the lognormal dispersion of the force reduction factors in each bin was computed using:

$$\beta = \sigma_{\ln X} \tag{6}$$

where ln$X$ represents the logarithm of the set of reduction ratios for a given bin, and σ is the standard deviation. The lognormal dispersion is shown in Figure 10 for (a) the bilinear and (b) Takeda hysteresis rules for all site classes as a function of the T/T_{peak} ratio. No obvious trends were observed for dispersion as a function of site class and so grouping was done by ductility ratio and hysteretic model only. In general, dispersion is greater for higher ductility ratios but remains relatively constant for periods greater than T_{peak}.

Considering the previously obtained dispersion, β, the 16th and 84th percentile reduction factors are given by $\theta e^{-\beta}$ and θe^{β} respectively, where θ represents the median force reduction factor for the bin. Figure 7 to Figure 9 plot the 16th and 84th percentile reduction factors (i.e., the 68% confidence interval (CI)). In general, the equal displacement approximation is reasonable with the 68% CI reduction factors enveloping the TS 1170.5 equation. However, consider reduction ratios associated with a ductility factor μ = 4 for the Takeda hysteresis rule at a location characterised by site class VI (i.e., bottom-right subplot of Figure 8). In this case, the 16th percentile force reduction factor is around two, meaning the equal displacement approximation can result in an error up to 100% for the 68% CI.

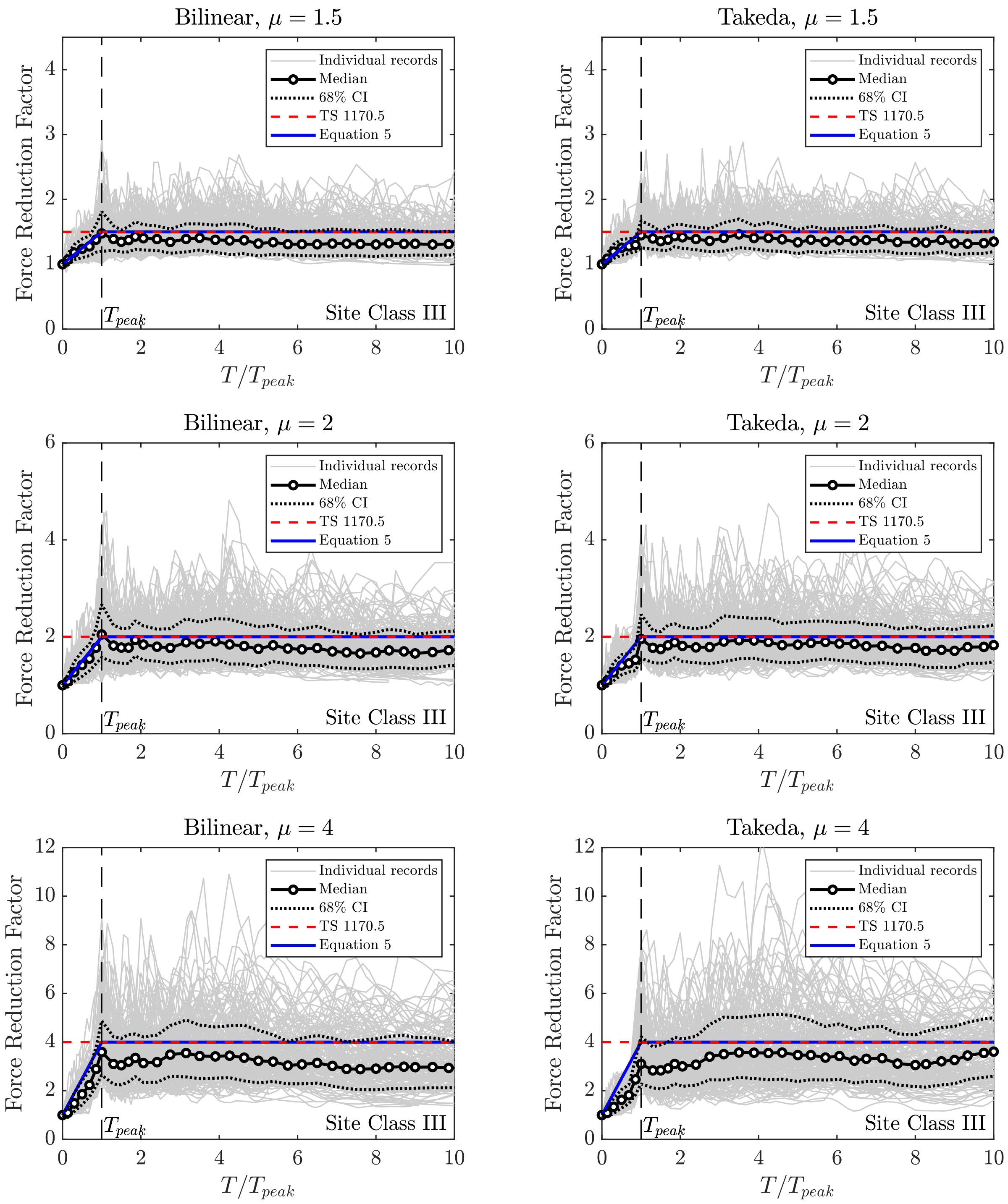

Figure 9: Reduction factors generated for site class III as a function of ductility demand.

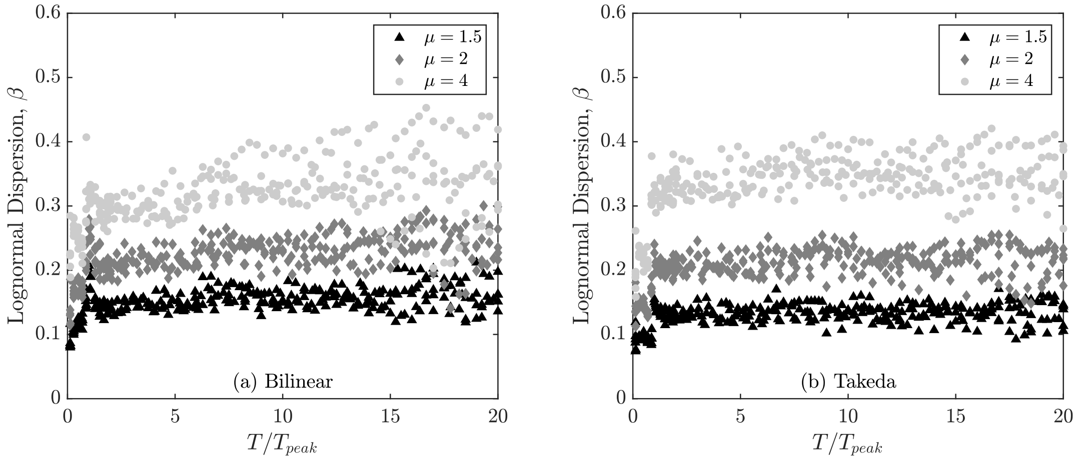

Figure 10: Lognormal dispersion of the force reduction factors for the (a) bilinear and (b) Takeda hysteresis rules.

VARIATION OF PEAK PERIOD VALUES FROM THE NATIONAL SEISMIC HAZARD MODEL

Figure 11 outlines the T_{peak} values from the NSHM obtained by evaluating the period at which the UHS is a maximum for the 1 in 500 annual probability of exceedance (APoE) and 1 in 2500 APoE. Based on the prior definition of T_{peak}, the values shown in Figure 11 can be considered a lower bound for which the equal displacement approximation is appropriate and would appear to indicate that short period structures should be defined as those with periods less than 0.2 s to 0.3 s for most sites and most common site classes, regardless of APoE. Sites with hazard strongly affected by high magnitude ruptures may exhibit T_{peak} values up to 1 s for the 1 in 2500 APoE for soft soil sites but these locations are typically sparsely populated South Island centres (i.e., Franz Josef, Haast, and Otira).

Some justification for the temporary use of the equal displacement approximation at all periods is provided in the following section, but in the future, the use of different force-reduction expressions for structures characterised by periods less than T_{peak} is recommended.

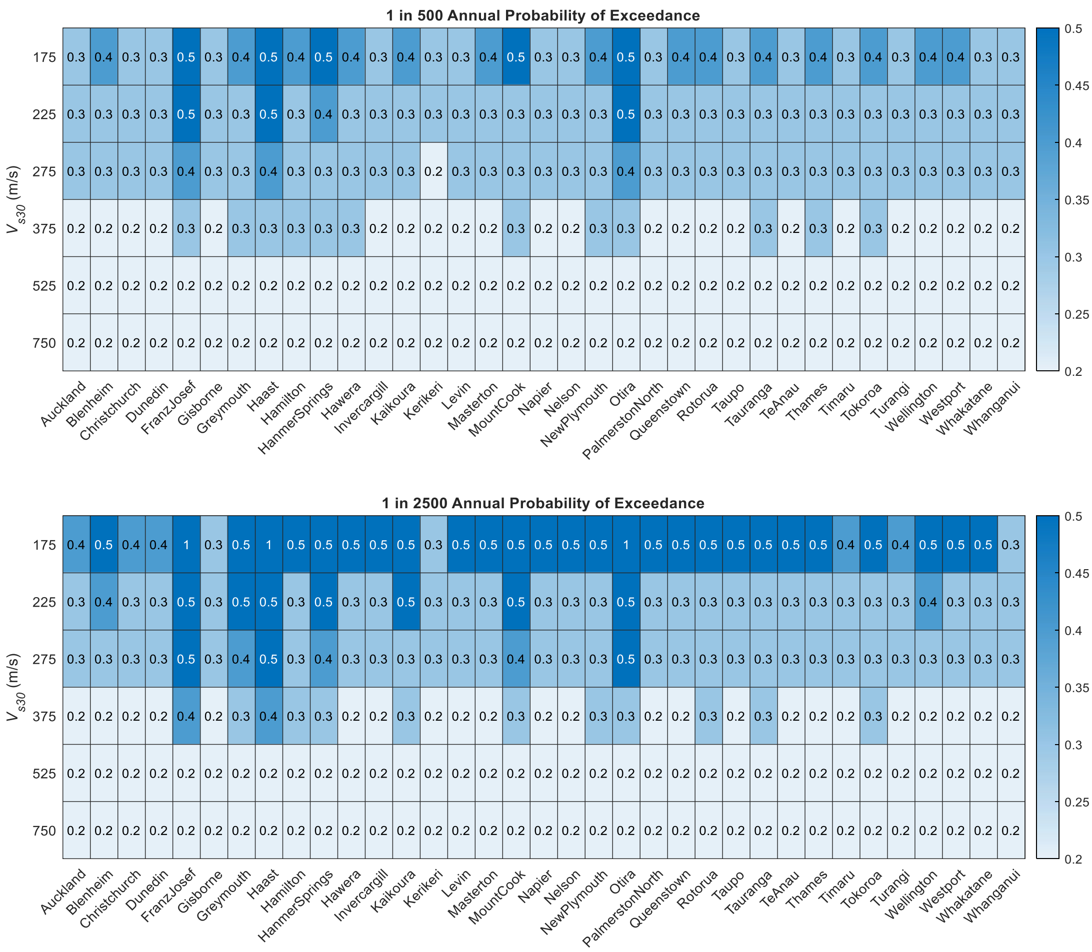

Figure 11: 1 in 500 and 1 in 2500 APoE T_{peak} values obtained from the NSHM for selected cities across all site classes.

RECOMMENDATIONS FOR DESIGN

Short-Term Recommendations

The results in Figures 7 to Figure 9 suggest that the use of the equal displacement approximation is non-conservative for periods greater than T_{peak} and is clearly invalid for periods less than T_{peak}. However, there are several reasons why the TS has adopted the equal displacement approximation for very short period structures. Firstly, recent studies in the United States (Applied Technology Council, 2020) report that short period buildings did not suffer high collapse rates in past earthquakes, even if numerical models suggested they should. One of the key observations from the Applied Technology Council (2020), which examined timber-framed buildings, steel concentrically braced frame buildings and reinforced masonry buildings, was that design ductility values and the associated displacement capacities, are too conservative for short period buildings. Furthermore, it is noted that a small amount of lateral deformation of foundations (that is often not accounted for in structural analyses) could significantly increase the period and deformation capacity of a short period structure.

In the short-term, the SRWG advocates the use of equal displacement approximation at all periods, owing to conservative definition of system deformation capacity and likely impact of foundation deformations. The proposed constant reduction factor for all periods also represents a simplistic approach that is consistent with the previous provisions of NZS 1170.5 (Standards New Zealand, 2004a).

Long-Term Recommendations

It is undesirable that an error in inelastic demand estimation (i.e., extending the use of equal displacement approximation to short periods) be used to treat under-estimations in the capacity of short period structures. Considering this, the SRWG is currently undertaking a broader review of the seismic design process. As part of that, more realistic assessment of building ductility and deformation capacity is advocated, especially for short period structures for which there is historic evidence for higher performance than modelling suggests (Applied Technology Council, 2020). Improved numerical approaches that better capture the displacement behaviour and collapse capacity of theses short period buildings could be used to inform inelastic spectra and would likely result in reduced strength requirements. Following this, the authors recommend a return to period-based elastic-to-inelastic demand ratios, potentially using the alternative approach proposed with Equation (5).

If more realistic assessment of deformation capacity becomes possible, T_{peak} could be incorporated into future versions of the strength reduction factor k_μ. T_{peak} should ideally be defined per locality, soil class, and APoE which could easily be achieved as part of the design tables in the proposed TS. However, another approach may be to define appropriate T_{peak} values as a function of corner period, T_c, which is a parameter already reported in the proposed TS design tables. Figure 12 outlines the ratio of T_{peak}/T_c for all 212 listed sites in the TS for each APoE and site class considered in the NSHM. The ratio centres around 0.5 and there appears to be little dependence on site class. Given this information, the general form of Equation (5) can be re-written fully in terms of variables already provided within the proposed TS:

$$k_\mu = \mu \quad \text{for } T_1 \geq 0.5T_c \qquad (7)$$

$$= \frac{(\mu - 1)T_1}{0.5T_c} + 1 \quad \text{for } T_1 < 0.5T_c$$

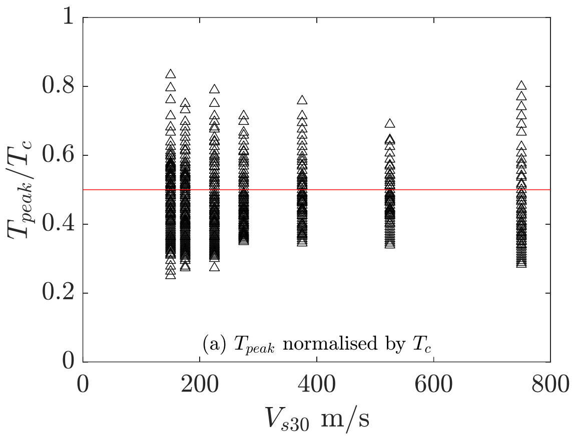

Figure 12: Relationship between T_{peak}/T_c ratio for each TS 1170.5 site class considering spectra from the NSHM.

CONCLUSIONS

This work investigated the suitability of the equal displacement approximation for short period structures and considered how to define a “short period structure” when relating inelastic and elastic spectral demands. Inelastic spectra, developed for a broad range of ground-motion characteristics and two different hysteretic models, indicate that a new parameter, T_{peak}, defined as the period ordinate for which the elastic acceleration response spectrum is a maximum, represents a lower-bound period for which the equal displacement approximation may be appropriate. Moreover, it is found that a “short period structure” should be defined as those with periods greater than 0.2-0.3s for most sites and locations in NZ, at a ground shaking intensity with an APoE of 1 in 500. Future research should aim to identify, through careful ground-motion selection, the impact of magnitude, fault rupture distance, and faulting characteristics on T_{peak} to check for variability with respect to these parameters.

The inelastic spectra results also confirm that the equal displacement approximation is not strictly applicable for short period structures and so alternative equations for force-reduction factors, as a function of T_{peak}, are proposed. Results suggest that the new proposal provides an effective and simple means of setting force reduction factors. However, it is also concluded that presently, use of the equal displacement approximation for all periods can be considered reasonable given that: (i) many buildings in NZ are likely to have a period of vibration greater than the short period limit, particularly when foundation deformations are accounted for, and (ii) recent literature suggests that the design ductility currently adopted to limit the fatality (or collapse) risk of ductile short period buildings may be conservative. Nevertheless, if future research prompts revisions to ductility and deformation capacity estimates for short period buildings, then it may be necessary to reconsider the use of the equal displacement approximation at all periods.

REFERENCES

1 Applied Technology Council (2020). "*Short-Period Building Collapse Performance and Recommendations for Improving Seismic Design; Volume 1 – Overarching Findings, Conclusions, and Recommendations*". FEMA P-2139-1, Federal Emergency Management Agency, USA. https://www.nibs.org/files/pdfs/FEMA_P-2139-1.pdf

2 Carr A (2016). *INSPECT - Inelastic Response Spectra Computation*. Carr Research Ltd.

3 Chopra AK and Chintanapakdee C (2004). "Inelastic deformation ratios for design and evaluation of structures: Single-degree-of-freedom bilinear systems". *Journal of Structural Engineering*, **130**(9): 1309–1319. https://doi.org/10.1061/(ASCE)0733-9445(2004)130:9(1309)

4 Emori K and Schnobrich WC (1978). "*Analysis of reinforced concrete frame-wall structures for strong motion earthquakes*". University of Illinois at Urbana-Champaign.

5 Lee RL, Cubrinovski M and Bradley BA (2025). "Site classification methodology for TS 1170.5 design spectra". *Bulletin of the New Zealand Society for Earthquake Engineering*, **58**(1): 11-39. https://doi.org/10.5459/bnzsee.1686

6 Miranda E (2000). "Inelastic displacement ratios for structures on firm sites". *Journal of Structural Engineering*, **126**(10): 1150–1159. https://doi.org/10.1061/(ASCE)0733-9445(2000)126:10(1150)

7 Miranda E and Ruiz-García J (2002). "Evaluation of approximate methods to estimate maximum inelastic displacement demands". *Earthquake Engineering and Structural Dynamics*, **31**(3): 539–560. https://doi.org/10.1002/eqe.143

8 Newmark NM and Hall WJ (1982). "*Earthquake Spectra and Design*". Engineering Monographs on Earthquake Criteria.

9 Otani S (1981). "Hysteresis model of reinforced concrete for earthquake response analysis". *Journal of Faculty of Engineering, The University of Tokyo (B)*, **36**(2): 551–558.

10 Priestley MJN, Calvi GM and Kowalsky MJ (2007). *Displacement-based Seismic Design of Structures.* IUSS PRESS.

11 Stafford PJ, Sullivan TJ and Pennucci D (2016). "Empirical correlation between inelastic and elastic spectral displacement demands". *Earthquake Spectra*, **32**(3): 1419–1448. https://doi.org/10.1193/020515EQS021M

12 Standards New Zealand (2004a). *NZS 1170.5—Part 5: Earthquake Actions—New Zealand*. Standards New Zealand.

13 Standards New Zealand (2004b). *NZS 1170.5—Part 5: Earthquake Actions—New Zealand Commentary*. Standards New Zealand.

14 Tothong P and Cornell CA (2006). "An empirical ground-motion attenuation relation for inelastic spectral displacement". *Bulletin of the Seismological Society of America*, **96**(6): 2146–2164. https://doi.org/10.1785/0120060018

15 Veletsos AS and Newmark NM (1960). "Effect of inelastic behavior on the response of simple systems to earthquake motions". *Proceedings of 2nd World Conference of Earthquake Engineering*, 895–912. https://www.iitk.ac.in/nicee/wcee/article/vol.2_session2_895.pdf

APPENDIX A – LIST OF GROUND-MOTIONS USED IN STUDY

Table 3. List of Record Sequence Numbers (RSN) from the PEER West NGA-2 ground-motion database.

	Site Class					
Record Number	**I**	**II**	**III**	**IV**	**V**	**VI**
1	59	1	12	30	6	60
2	80	2	13	31	7	174
3	455	14	15	51	8	175
4	765	17	28	61	9	192
5	788	32	37	93	20	326
6	789	33	39	95	22	334
7	795	57	40	126	36	462
8	797	58	52	141	38	469
9	804	63	54	158	53	718
10	1011	67	55	167	56	721
11	1091	71	62	268	69	726
12	1108	72	64	322	82	728
13	1245	73	65	329	122	729
14	1256	78	66	332	160	738
15	1257	81	68	333	162	758
16	1307	85	70	337	163	777
17	1319	87	74	341	165	780
18	1347	89	75	344	169	799
19	1352	91	76	345	172	808
20	1366	94	79	350	176	962
21	1371	121	83	360	183	1049
22	1378	125	84	367	186	1147
23	1432	139	86	368	191	1185
24	1440	164	88	428	266	1189
25	1442	231	90	430	269	1199
26	1445	265	92	432	328	1200
27	1446	283	123	436	331	1207
28	1452	284	124	437	338	1209
29	1518	286	137	439	342	1212
30	1571	288	138	449	343	1216
31	1587	289	140	456	348	1228
32	1613	291	142	461	355	1229
33	2508	293	166	468	446	1233
34	2514	294	187	505	447	1237
35	2633	295	188	507	458	1239
36	2687	296	190	509	464	1247
37	2753	297	230	515	719	1250
38	2759	299	232	542	720	1317
39	2805	300	280	573	737	1328
40	2929	302	281	575	757	1331
41	2989	304	282	577	768	1334
42	2995	325	287	578	778	1336
43	2996	336	290	579	783	1337
44	3042	351	298	580	786	1357
45	3053	352	301	733	873	1364
46	3094	356	303	752	967	1365
47	3135	357	313	754	1059	1390
48	3138	358	323	761	1141	1411
49	3139	369	324	772	1146	1412
50	3145	420	327	776	1180	1414
51	3194	427	330	790	1183	1418
52	3251	448	335	800	1187	1421
53	3318	450	339	806	1188	1422
54	3324	454	340	833	1195	1423
55	3325	471	346	839	1196	1538
56	3374	472	347	841	1203	1542
57	3390	476	349	843	1204	1599
58	3429	495	353	847	1213	2454
59	3430	496	354	851	1217	2463
60	3479	497	359	853	1220	2464
61	3542	511	362	856	1221	2471
62	3799	512	363	861	1222	2473
63	3893	525	364	866	1223	2476
64	3895	528	366	871	1224	2478
65	3920	536	425	878	1226	2492
66	3925	537	426	883	1240	2493
67	3954	541	429	888	1242	2497
68	4083	550	431	895	1243	2501
69	4167	552	433	909	1246	2502
70	4247	553	434	920	1304	2510
71	4248	554	435	931	1308	2513
72	4926	555	438	933	1311	2559
73	4969	557	440	936	1323	2561
74	5006	572	441	944	1324	2562
75	5013	587	453	949	1329	2647
76	5043	734	457	951	1330	2651
77	5048	739	460	956	1332	2696
78	5052	740	470	959	1343	2706
79	5053	741	503	964	1344	2707
80	5054	747	504	968	1353	2713
81	5055	748	506	970	1356	2715
82	5073	749	508	-	1359	2718
83	5084	750	510	-	1362	2720
84	5110	751	513	-	1373	2723
85	5138	753	514	-	1374	2736
86	5168	755	516	-	1382	2737
87	5184	763	517	-	1383	2741
88	5185	769	518	-	1384	2745

Record Number	Site Class					
	I	II	III	IV	V	VI
89	5191	771	519	_	1385	2747
90	5205	774	520	_	1386	2755
91	5232	775	521	_	1388	2758
92	5313	779	522	_	1389	2815
93	5360	781	523	_	1395	2818
94	5363	782	524	_	1397	2831

Record Number	Site Class					
	I	II	III	IV	V	VI
95	5388	791	526	_	1399	2839
96	5406	793	527	_	1409	2840
97	_	794	_	_	_	2885
98	_	796	_	_	_	2889
99	_	798	_	_	_	2939
100	_	801	_	_	_	2948