Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 52, No. 3, September 2019

GROUND MOTION INPUT FOR NONLINEAR RESPONSE HISTORY ANALYSIS: PRACTICAL LIMITATIONS OF NZS 1170.5 AND COMPARISON TO US STANDARDS

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(Submitted *January 2019*; Reviewed *April 2019*; Accepted *April 2019*)

ABSTRACT

Nonlinear response history analysis (NLRHA), or so-called "nonlinear time history analysis", is adopted by practicing structural engineers who implement performance-based seismic design and/or assessment procedures. One important aspect in obtaining reliable output from the NLRHA procedure is the input ground motion records. The underlying intention of ground motion selection and amplitude-scaling procedures is to ensure the input for NLRHA is representative of the ground shaking hazard level, for a given site and structure.

The purpose of this paper is to highlight the salient limitations of the ground motion selection and scaling requirements in Sections 5.5 and 6.4 of the New Zealand (NZ) loading standard NZS 1170.5 (2004). From a NZ regulatory perspective; there is no specific framework for seismic hazard analysis and ground motion selection (thus self-regulation is the current norm). In contrast, NZS 1170.5 contains many prescriptive requirements for scaling and applying records which are challenging to satisfy in practice. Also discussed within, there are implications for more modern guidance documents in NZ, such as the 2017 *"Assessment Guidelines"* for existing buildings, which cite NZS 1170.5, a standard which is at least 16 years old (draft issued in 2002). To emphasize the above issues with NZS 1170.5, this paper presents a summary of the more contemporary approaches in the US standards ASCE 7-16 (new buildings) and ASCE 41-17 (existing buildings), along with some examples of the more stringent US requirements for Tall Buildings.

INTRODUCTION

Nonlinear response history analysis (NLRHA), or so-called "nonlinear time history analysis" (NLTHA), is an increasingly common analysis approach used by consulting engineers for performance-based seismic assessment and design. It is the most sophisticated analysis method available. Actual ground accelerations are applied to the base of a computation model that explicitly models the non-linear force and deformation response of structural components. The resulting damage and time-varying properties of the non-linear components are continually updated within this process.

The greater level of sophistication associated with NLRHA requires considerable technical knowledge of structural behaviour and earthquake engineering, processes within the analysis software itself, as well as skills to critically review resulting behaviour and output. This is required to define the proper inputs for NLRHA—and confidence in the quality of resulting design decisions or advice relies on the inputs.

One critical input "ingredient", and the focus of this paper, is the ground motion (i.e. acceleration time series) selection, scaling and application procedures. Ideally, when paired with appropriate and compatible methods of response measurement, evaluation procedures should provide a sufficiently accurate representation of seismic demands at the selected hazard level—neither overly conservative, nor unconservative—and which are specific to the site and structure which is being evaluated.

Acknowledging their exclusion from the NZ Building code verification method B1/VM1, specific requirements for ground motion selection and scaling are given in Sections 5.5 and 6.4 of the New Zealand Standard (NZS) for seismic loading, NZS 1170.5 [1]. From a regulatory perspective therefore, these provisions have been generally straightforward to apply as part of an alternative solution using NLRHA. However, it is now over 16 years since these Sections 5.5 and 6.4 were drafted. The purpose of this paper is to highlight some frequently encountered issues and limitations of NZS 1170.5, given more developments in the literature since its publication, and in the context of their application in practitioner environment.

US standards ASCE 7-16 [2] "Minimum Design Loads for New Buildings", and ASCE 41-17 "Seismic Evaluation and Retrofit of Existing buildings [5] have adopted more contemporary methods than NZS 1170.5, which makes them attractive to practitioners as alternative solutions (for reasons which will be discussed within). Although similar requirements exist in other international standards (such as Eurocode 8), a global review is beyond the scope of this paper.

This paper is particularly relevant for consulting engineers in NZ in both new design work and assessment of existing buildings following the NZSEE (2017) Engineering Assessment Guidelines [4]. The research community should also find the content relevant, on the basis that "code-based" ground motion selection and scaling procedures are widely adopted in research applications of NLRHA.

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This paper is organised into the following sections:

1. Common applications of NLRHA in NZ;
2. General overview of the three main "tasks" for determining suitable ground motion input for NLHRA. The tasks are:
   i) ground motion selection;
   ii) ground motion scaling;
   iii) application to the computational model.
3. Summary of the relevant NZS 1170.5 requirements.
4. Requirements of other guidelines, including the NZSEE *Assessment Guidelines* (2017), and the draft base-isolation guidelines (currently in development).
5. Limitations of the current NZS 1170.5 procedure are presented, with emphasis on the prescribed period ranges, goodness of fit rules, and genuine reasons for large amplitude scale factors.
6. US Standards ASCE 7-16 and ASCE 41-17 are summarised, along with brief mention of some practical issues.
7. "Above-minimum" requirements for exceptional, or tall buildings in the US are summarised.

PRACTICAL APPLICATIONS OF NLRHA IN NZ

NLRHA in a practitioner environment is generally reserved for situations where suitable technical goals cannot be achieved using simpler analysis approaches. This may arise where simpler approaches do not reliably handle complex configurations or behaviours, or it could be due to a particular type of output information sought. Common instances are listed below:

1. New buildings with specific components (e.g. viscous dampers, base-isolators), materials, or configurations not adequately provided for within the NZ Building Code (NZBC) verification methods. Verification of seismic performance by NLRHA is often applied as part of an alternative solution per 2.2.16 of the NZBC (B1 Structure), possibly in combination with a rational design procedure based on simpler methods and/or general principles.

2. Performance verification of existing buildings with seismic retrofit/strengthening. The overall compliance strategy with the NZBC is similar as in 1 (above).

3. Detailed Seismic Assessment (DSA) of existing buildings completed under [4]. NLRHA may be adopted for complex behaviours and/or poorly configured buildings which may inappropriately extend the capabilities of simpler tools outside generally accepted uncertainty. Examples include structures with significant material or geometric non-linearity, significant irregularities (e.g. mass, stiffness or geometric), mixed/transitioning lateral load resisting systems, unrestrained inelastic torsion, undesirable yielding mechanisms (e.g. column sways or shear controlled behaviour), complex unreinforced masonry structures, or other situations where complex non-linear behaviours are likely to affect global dynamic response.

4. Performance verification of new buildings with similar less-desirable configurations as those described in (3) above, or mixed systems. This could also include special studies for particular aspects of new buildings.

5. When an owner/stakeholder has specific requirements to have the building seismic performance to be evaluated. This might include, for example, insurance investigations for prospective developers/purchasers, or for post-earthquake damage evaluations.

6. Forensic investigation work.

The benefits of NLRHA have been realized on a variety of NZ projects [6, 7, 8]. Compared with the technological "hurdles" faced in past decades, NLRHA is now capable of further advancement and wider adoption due to: a) increased software capabilities and processing power, and; b) more populated experimental test databases that define constitutive force-deformation (e.g. moment-rotation, or stress-strain) relationships for various structural component types.

NLRHA as a Tool for Complete Performance Based Seismic Evaluation

NLRHA may also form part of a complete performance-based evaluation (PBE) comprising an integration of seismic hazard, response and fragilities over a wide range of hazard levels.

Despite progress internationally in improving accessibility of these techniques, PBE remain in its infancy in New Zealand practice. Most likely this is due to the level of effort required, and the scarcity and reliability of the requisite fragility data to assure sufficiently reliable conclusions. NLRHA for PBE is less commonly adopted in NZ by practitioners, however is likely to become more important in the future.

In the research community, full PBE of structures (buildings and infrastructure) holds very high promise in better informing evidence-based policy decisions and standards development, related to building resilient communities which better balance seismic risks over a range of return periods with economic factors. Confidence in ground motion input becomes immensely important in this case (as is being investigated under QuakeCoRE Flagship 1: Ground Motion Simulation).

GENERAL OVERVIEW: DETERMINING SUITABLE GROUND MOTION INPUT FOR NLRHA

Ground motions for NLRHA should provide a relatively unbiased representation of the design-level seismic hazard. This section provides an overview of the three main "tasks" of determining ground motion input, prior to performing NLRHA.

Task 1: Hazard Analysis and Ground Motion Selection

Prospective ground motions are selected based on the design-level seismic hazard. First, a probabilistic seismic hazard analysis (PSHA) is performed for a specific site to define the ground motion intensity with annual probability of exceedance (APE) consistent with the APE design criteria in NZS 1170.0. PSHA is typically used to define the seismic hazard in NZ, although deterministic analyses or checks are sometimes used. This paper herein refers specifically to PSHA.

Disaggregation of the seismic hazard is then used identify the dominant fault (or faults) rupture mechanisms, magnitudes, and rupture source-to-site distances to inform the selection of prospective ground motions. Preserving this "rupture scenario information" in the ground motion selection procedure implicitly ensures that the intensity measures, such as amplitude and duration, are notionally consistent with those expected from the rupture scenario(s) that governs the seismic hazard for a given site and structure. The rupture scenario information is used as the basis for selecting ground motions from publicly available datasets of historical earthquake records, such as the PEER NGA-West2 Strong Ground Motion Database [9], (https://ngawest2.berkeley.edu/).

Ground motion records downloaded from online databases comprise acceleration time series for two horizontal and one vertical component of motion. That is, database queries are carried out to identify prospective ground motions with similar magnitude, distance, fault mechanism, geologic condition, or other parameters. Emphasis is placed on finding ground motions that closely match the ground motion response spectra amplitude, which is well-correlated with most geotechnical and structural problems. Other ground motion intensity measures, such as significant duration, are also important considerations for selecting ground motions. These intensity measures are correlated with rupture scenario parameters (e.g. magnitude and distance), and the practice of using rupture scenario parameters as database query filters is intended to identify prospective ground motions with a consistent set of intensity measures. The ground motion selection procedure lends itself towards sophisticated algorithms, such as the Conditional Mean Spectrum (CMS) [10, 11] or Generalised Conditional Intensity Measure (GCIM) algorithms [12, 13].

There are no strict regulatory frameworks for seismic hazard analysis and ground motion selection thus, the current practical norm relies on self-regulation using general specifications to meet the fundamental objectives alluded to above. Generally, this task will be outside the expertise or means of practising structural engineers, and often requires specialist input from a consulting engineering seismologist.

Task 2: Ground Motion Scaling

Once the catalogue of prospective ground motions is collated, the ground motions must be scaled to be compatible with the design response spectra (typically a Uniform Hazard Spectrum, UHS). “Task 2” is essentially a calculation procedure to determine linear scale factors which are then used for “Task 3”.

Seismic loading standards generally contain prescriptive requirements for ground motion amplitude scaling, based on the 5% damped elastic response spectral accelerations. The most notable difference between different standards is whether “maximum response scaling” or “mean response scaling” is permitted (both of which are discussed in the later sections of this paper). Regardless, the general intent is to ensure the ground motion response spectra is scaled to be consistent with the code design response spectra over the structure’s vibration period range of interest, as illustrated schematically in Figure 1. A more comprehensive illustration of the maximum response spectra scaling (of 3 records), and the mean response spectra scaling (of 11 records) is shown in Figure 2 below.

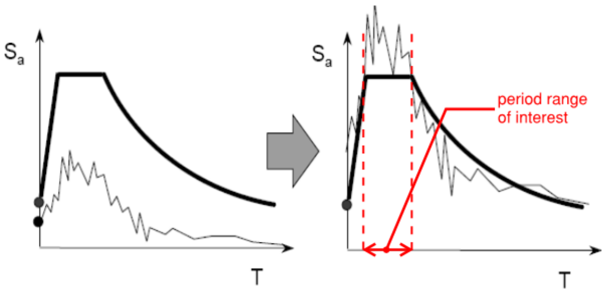

Figure 1: Schematic illustration of amplitude-scaling the ground motion response spectrum to the design spectrum over a range of vibration periods. (Modified from [14].)

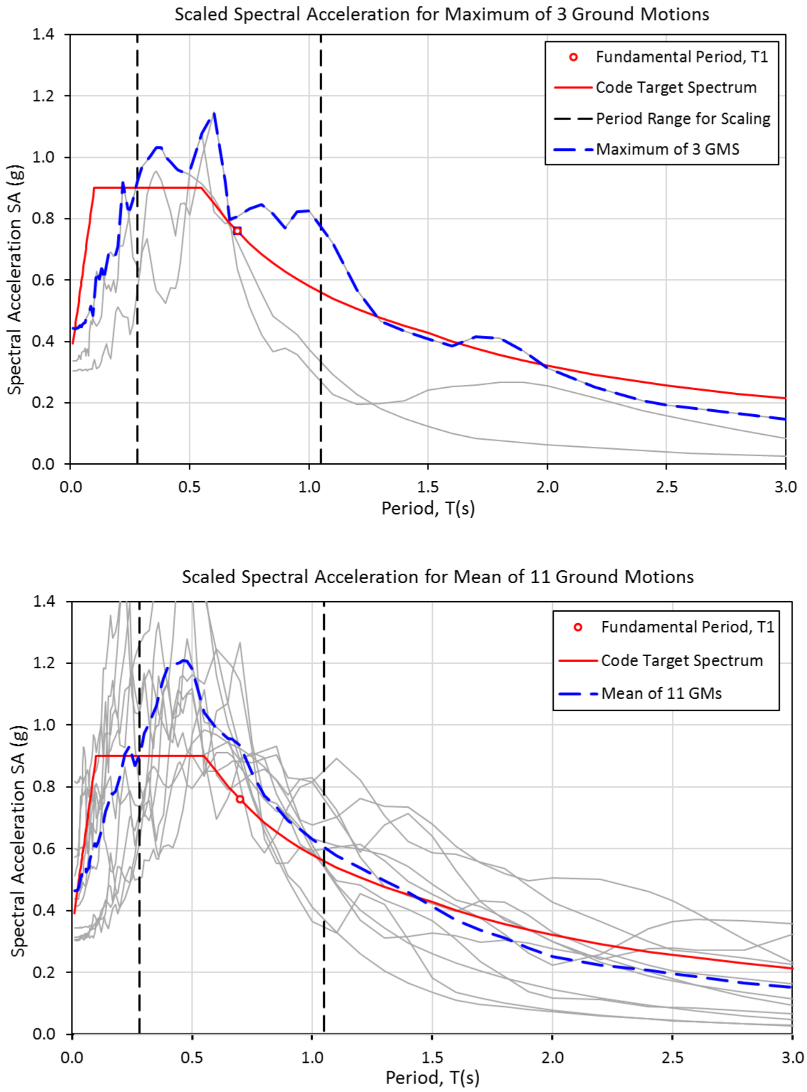

Figure 2: Examples of ground motion amplitude-scaling for maximum response scaling (top), and mean response scaling (bottom).

As the prescribed scaling requirements are focussed solely on matching the target spectra, there is an implicit assumption that other ground motion intensity measures (such as duration) will be notionally consistent with the seismic hazard. This assumption is made on the condition that "Task 1" has been completed appropriately. However, this is often not the case, as illustrated in [12, 15].

From the perspective of life safety or collapse avoidance, it is often the non-linear engineering demand parameters which control the evaluation outcome, however this is not necessarily consistent with the scaled elastic response spectra. This essentially means that Task 2 is based on the fundamental assumption that scaling the elastic response spectra will provide a sufficient level of inelastic demands on the structure.

Task 3: Application to the Computational Model

Before running NLRHA, the last step is to apply the components of horizontal ground acceleration at the base level of the computational model. In some cases, a third component of vertical motion may also be applied.

The most important aspect of this task is to ensure that each axes of a building's lateral load resisting system is subjected to a consistent amplitude of ground shaking such that any unintended "directional bias" in the analysis is minimised. Directional bias is treated differently for sites that are classified as being "far-field", or "near-fault" (a classification which is based on the seismic hazard disaggregation). Polarization of ground shaking cannot be easily predicted for far-field sites (unless state-of-the-art simulation methods are used), which means a "random" direction of ground shaking is assumed.

Different considerations exist for near-fault regions (such as Wellington), however these considerations are complex. Inter-relating factors such as the extent to which their near-fault effects are included in an elastic design spectra, including the likelihood of pulse-like motions occurring, the extent to which their characteristics affect a particular structure, the strong influence of velocity, period and damping (and the type of damping), and whether there are multiple dominant faults/scenarios.

Polarization of near-fault ground motions can have a significant affect, particularly on medium-long period structures. While the fault-normal component of ground motion may not have markedly higher spectral accelerations, the energy content of large velocities pulses can significantly affect the structural response. This can lead to significant differences between the outputs of NLRHA, and modal response spectrum analysis [17]. As noted in the commentary C3.1.6 of NZS 1170.5:

"…for long-period structures at locations where directivity effects could be significant, it is recommended that time history analysis is carried out".

Defining a repeatable ground motion scaling/application procedure that consistently match engineering demand parameters to a return period is a significant challenge. There are no specific recommendations in NZS 1170.5 about near-fault motions, other than if a site is located in a near-fault region, ground motions with pulse features should be used in NLRHA, and that the design spectra is modified by the prescriptive "near-fault" factor.

In some standards and guidelines, the method of representing near-fault ground shaking involves rotating the as-recorded horizontal component pairs to be fault-normal (FN) and fault-parallel (FP). These rotated ground motion records are then applied at an orientation which represents the relative angle between the causative fault and the axes of the buildings lateral load resisting system. The practical challenges and complexities with these near-fault requirements are discussed in a later section in this paper.

NZS 1170.5 (2004) REQUIREMENTS FOR GROUND MOTION SELECTION AND SCALING

Ground motion record selection and scaling requirements are laid out in Sections 5.5 and 6.4 of NZS 1170.5. These requirements are summarised in this section.

Firstly, however, the authors believe it is an important reminder that the overall NLRHA framework in NZS 1170.5 is tailored towards evaluation of the Life Safety performance objective. There are no explicit requirements or modifications for the assessment of Collapse Prevention.

Minimum of 3 Ground Motions – Maximum Response

A minimum of three ground motions is required, and maximum response-spectrum amplitude scaling is prescribed. A similar requirement exists in Eurocode 8. For evaluating the NLRHA output, the maximum overall response parameter from all three ground motions is used.

Section 5.5.1: Ground Motion Selection

The ground motion selection methodology must follow the requirements of Section 5.5 (extract below):

"The ground motion records shall be selected from actual records that have a "seismological signature" (i.e. magnitude, source characteristic (including fault mechanism) and source-to-site distance) the same as (or reasonably consistent with) the signature of the events that significantly contributed to the target design spectra of the site over the period range of interest. The ground motion is to have been recorded by an instrument located at a site, the soil conditions of which are the same as (or reasonably consistent with) the soil conditions at the site."

Although the provision above refers to "actual records", the standard also states:

"Where three appropriate ground motion records are not available, simulated ground motion records may be used to make up the family."

Section 5.5.1: Vertical Ground Motion Input

Vertical ground motions are briefly touched on here, but not further throughout this paper. Section 5.5.1 of NZS 1170.5 suggests when vertical ground motion input is necessary:

"The vertical component of the record may also be necessary when considering the response of structures or parts that are sensitive to vertical accelerations such as for horizontal cantilevers or some items of equipment."

The standard does not have any requirements for how vertical components should be scaled for NLRHA, leaving it to the discretion of the engineer. In practice, the most common and intuitive approach is to apply the same scale factor that is derived for the two horizontal components. As eluded to in C3.2 of the commentary, the vertical response of the overall structure will mostly depend on whether it is heavy or light-weight, and the stiffness of the soil and foundation system. ASCE 7-16 is a useful reference to for more explicit requirements for vertical ground motion input.

Section 5.5.2: Scaling Ground Motion Records

This section contains several prescriptive requirements for the response spectra amplitude-scaling of the suite of ground motions. The individual record scale factors (k_1) are calculated first, before the "family" scale factor (k_2) is found for the entire suite. The two-step calculation details are summarised below:

1. Determine the individual record scale factors, k_1, for each horizontal component, based on minimizing the statistical goodness-of-fit parameter D_1. The D_1 parameter is defined in the Commentary section, as per Equation (1) below:

$$D_1 = \sqrt{\frac{1}{(1.3\text{-}0.4)T_1}\int_{0.4T_1}^{1.3T_1}\left(\log\frac{k_1 SA_{component}}{SA_{target}}\right)^2 dT} \quad (1)$$

   Over the prescribed period range, the intent is to achieve a statistical "best fit" by minimizing the root-mean-squared difference between the scaled response spectra and the NZS 1170.5 "target" hazard spectra. The ground motion component with the smallest scale factor k_1 is defined as the "primary" horizontal component, and the limiting range is: $0.33 < k_1 < 3.0$.

2. Determine the family scale factor, k_2, to be applied to the entire suite of records. This factor ensures that, within the prescribed period range, the envelope of the principal component spectra (scaled already by k_1), exceeds the target hazard spectra. The limiting range is: $1.0 < k_2 < 1.3$.

Sections 6.4.3 & 6.4.4: Application to Computational Model

This section requires that records are applied twice, meaning the analysis is repeated for the same ground motion but with the primary and secondary horizontal components "flipped" with respect to the building axis within the model. This essentially means that applying a minimum of three ground motions requires six separate analysis (per each mass eccentricity). NZS 1170.5 states that:

> *"Both the principal component and the scaling factors will usually be different for each analysis because T_1 is different for both situations* [orthogonal directions]*."*

In the authors' experience, different scale factors are generally not applied for each direction when using 3D models. Instead, the period range is bounded for the fundamental periods in each direction (elongating the period range), and the same scale factor is used for both components. There is no technical basis to be confident that separate scale factors would not alter the characteristics and directional bias of the records.

NZSEE (2017) ASSESSMENT GUIDELINES FOR EXISTING BUILDINGS: REQUIREMENTS FOR GROUND MOTION INPUT

For complex existing buildings, consulting engineers may adopt NLRHA as a means for conducting Detailed Seismic Assessments (DSAs) to the NZSEE (2017) Assessment Guidelines [4] (herein referred to as the *Assessment Guidelines*). The *Assessment Guidelines* directly cite Sections 5.5 and 6.4 of NZS 1170.5. Recent progress in ground motion selection and scaling methods since the publication of NZS 1170.5 is recognised in the Appendix Section C2C.2 commentary:

> *"More recent research… has indicated that the NZS 1170.5:2004 requirements for input ground motions may need to be updated. These include the minimum number of ground motion records that should be analysed and the method which should be used to assess the results. This is summarised in Table C2C.1."*

And;

> *"At this stage, it is recommended that the selection and scaling of input ground motion is independently reviewed."*

Whilst it is helpful that the "currency" of NZS 1170.5 is identified as a limitation in the *Assessment Guidelines*, defining the scope of review and competence of the reviewer are left to the project team. Aside from this paper, along with publications by the last author [18, 19], there is shortage of guidance available to consulting engineers which has been presented in the context of NZ practice.

Alternative Verification Methodology: ASCE 41-13

Clause C1.6.2 of the *Assessment Guidelines* facilitates the use of the ASCE 41-13 [20] "Tier 3 Assessment" procedure using NLRHA as an acceptable alternative verification methodology. While ASCE 41-13 is stipulated by the *Assessment Guidelines*, it is now the predecessor to ASCE 41-17.

Using this alternative verification methodology can be attractive due to the more modern approach for NLRHA outlined in ASCE 41. The *Assessment Guidelines* requires that:

> *"The acceptance criteria and recommendations given in ASCE 41, unless specifically modified for NLTHA in these guidelines, should be used in their entirety this includes the process of selection and scaling of strong motion records."*

This alternative approach results in a building risk rating which is calibrated at two return periods on the hazard spectrum; Life Safety (LS) at ULS, and Collapse Prevention (CP) under more severe ground shaking. This varies from the one ULS check required by the *Assessment Guidelines*. In many cases this can provide a more transparent and robust measure of resilience to shaking—as the fragility of controlling system mechanisms to some higher level of demand is reviewed directly, rather than by reliance on margins in the material/component strain limits applied at the ULS.

The ground motion requirements in ASCE 41-13 are similar to those from previous standards from the decade prior, whereas ASCE 41-17 includes significant modifications to be consistent with ASCE 7-16, which incorporates a more contemporary approach to ground motion input for NLRHA. For this reason, these methods (discussed later herein) have been applied to for some NZ projects.

NZ DRAFT BASE ISOLATION GUIDELINES

Base-isolated (BI) structures are not directly covered by the NZBC, which means NLRHA is required to verify the seismic performance as an acceptable alternative solution. The NZ draft BI Guidelines (currently in development) cite the requirements of NZS 1170.5 for ground motion selection and scaling, with the authors understanding that the main modifications to be:

- The period range for scaling calculations is modified to account for upper bound and lower bound properties of the isolators.
- The family scale factor k_2 can be calculated based on scaling to 0.90 times the target hazard spectra. This "90% floor" also appears in ASCE 7-16 (whereas in that standard it applies for all types of structures).
- The "average of 7" ground motions is permitted as an alternative to the "maximum of 3" approach. As discussed later, both the "maximum of 3" and "average of 7" approaches do not always provide reliable demand parameters [2, 18, 21].
- As a permissible alternative to the conventional uniform hazard spectrum (UHS), the target hazard spectra may be defined based on the condition mean spectra, or conditional spectra. The authors note that "Method 2" in ASCE 7-16 is an existing framework which could be adopted, in lieu of the UHS.

LIMITATIONS OF NZS 1170.5 (2004) FOR GROUND MOTION SELECTION AND SCALING

The ground motion selection and scaling requirements in NZS 1170.5 were written more than 16 years ago. As noted previously, aspects of these procedures are cited within the *Assessment Guidelines* and draft BI Guidelines, yet several limitations in current ground motion selection and scaling practices are encountered in NZ engineering practice. Some of the limitations in the NZS 1170.5 procedure are discussed in the below sections.

Maximum Response of 3 (or More) Ground Motions

The specification of a minimum of 3 records dates back to NZS 4203 (1992) [22] which, in part, was to deal with the trade-off in analysis run times. However, within the past 30 years, computational efficiency has improved significantly. In today's consulting offices, analysis run times are less constrained due to multi-core processors and cloud-computing capabilities.

Seismic demands have a large record-to-record variability. As stated earlier, the demand parameters of interest in NLRHA are usually non-linear parameters such as component strains; not the elastic spectral accelerations or spectral displacements. Elastic spectra might arbitrarily give impressions on variability in demands between records which is misleading. This variation in demand parameters of real interest is likely to be difficult to quantify, and for such a limited number of ground motions must be treated as unknown. In the majority of cases, demands from the "maximum of 3" approach in NZS 1170.5 may be conservative. There remains, however, a significant and undesirable probability of 15 to 30 percent that the "maximum of 3" demand is actually lower than the true mean seismic demand [23, 24]. The higher spread of demands about the true mean value could be significant in terms of the outcome of the analysis and the decisions which result from it. The poor handling of variability in demand parameters is a significant shortcoming of the maximum of three approach.

Even if greater than 3 ground motions are used, the specific wording is such that, in order to comply with NZS 1170.5, the evaluation must be based on the maximum response. This requirement provides a major disincentive for including greater variability in the analysis. By increasing the number of ground motions, statistically the maximum response will also increase, which can ultimately shift from the evaluation from being potentially unconservative, to being systematically over conservative by an unknown amount [18].

In attempt to mitigate the above issues, consulting engineers in NZ have commonly adopted the mean response from at least 7 ground motions, using ASCE 7-10 (now superseded) as an alternative method. The latest version ASCE 7-16 requires a minimum of 11 records (discussed later herein), and results in substantially lower scatter about the true mean value.

Ground Motion Selection (and Seismic Hazard Analysis)

The requirements in Section 5.5 are somewhat vague. However it is generally accepted that the nature of the seismic hazard (i.e. the so-called '*seismological signature*') underlying the design spectrum should be ascertained based on hazard disaggregation. Disaggregation results are not directly available without a site-specific seismic hazard analysis study being performed. Once obtained, site specific hazard spectra could be used for other purposes such as to set the hazard level for the project in lieu of the NZS 1170.5 design spectrum.

Seismic hazard analysis and ground motion selection is an area of the NZ earthquake engineering profession that is currently self-regulated. NZ Territorial authorities (TA's) generally do not require seismic hazard analysis to independently peer reviewed. In particular, there are a growing number of seismology/geotechnical engineering consultants offering services of seismic hazard analysis. Given that C5.5 is somewhat open to interpretation, the authors of this paper, among others [25], believe that the potential risks to consistency and quality from self-regulation could be better mitigated by having an acceptable framework procedure introduced to NZ practice. For efficiency, the proposed framework could be adopted as interim guidance in the first instance, and later cited in future versions of NZS 1170.5. This notion of reaching "accepted practice" is particularly important if site specific hazard spectra are proposed to set the design demands, rather than simply to aid in selection of ground motions. Appropriate design hazard spectrum is set by a greater number of considerations than the UHS alone.

Across NZ, there are property owners looking to understand the uncertainties inherent within the NZS 1170.5:2004 design hazard spectra and whether it is fit for their purposes. In the interim, there may be a greater number of site-specific PSHA studies commissioned by property owners looking to understand the future capacity of their assets, and how potential future-proofing can be implemented sooner.

As a note of caution, whilst site-specific seismic hazard analysis represents the latest in scientific understanding if executed appropriately, it can introduce complexity when it produces results which significantly depart the current NZS 1170.5 designed hazard spectra. This can arise from updated seismic hazard models (and other inputs such as revised GMPEs used for PSHA), from simplifications in the transfer of generic regional PSHA into codified design spectra, from complex issues such as basin-edge effects, and from known deterministic limits which are applied over PSHA to fulfil some stated objective such as minimum (or maximum) acceptable levels of robustness. All of the above issues are not uncommon, and require careful navigation and review from the consulting engineering seismologist. An example of this issue is presented in [26], for a project in Nelson.

Prescribed Period Range for Scaling Calculations

NZS 1170.5 prescribes the period range of (0.4-1.3)T_1 for scaling calculations, as shown as the integral bounds of Equation (1), where T_1 is the structure fundamental period. The prescribed upper bound and lower bound period is discussed below. In the authors view, generalising the period range of all buildings is simple, but not always appropriate as the dynamic vibration characteristics and material response is truly structure-specific. Amendment 1 (2016) of NZS 1170.5 recognises this point in the commentary; as the prescribed period range is considered to be inappropriate for base-isolated structures (as noted earlier), and for rocking structures.

It should be noted that any apparent un-conservatism associated with the prescribed period range may not result in an un-conservative NLRHA output. Holistically, this may be offset by other factors in the NZS 1170.5 framework for conducting NLRHA, such as taking the maximum response, and by scaling to a design spectrum derived from a uniform hazard spectrum. This being said, NZS 1170.5 is also based on scaling the response spectra to "minimise misfit", which is deliberately inhibiting variability in the elastic response spectra of each record. If the record-to-record variability is retained, then factors such as the prescribed period-range may not be so problematic [19]. Altering the period range from that prescribed by NZS 1170.5 may only be required if there are major concerns specific to the structural response.

The **upper bound period** (T_{UB}) should implicitly account for period lengthening due to material yielding and should be a function of the level of inelastic behaviour that is expected. If the secant period is used, the upper bound 1.3T_1 corresponds to a generic system with a global system displacement ductility of approximately 1.7. Whilst perhaps on the low side, in many cases this is likely to be reasonable, as actual global system displacement ductility for many buildings will be substantially lower than those of the governing components corresponding to the language of force-based design procedures in NZS 1170 (and its predecessor standards).

Although it is not clearly stated in NZS 1170.5, the upper bound period $1.3T_1$ is intended for assessing the Ultimate Limit State (ULS) performance level. If a higher ground shaking hazard level is also being included, such as Collapse Prevention (CP), the upper bound period $1.3T_1$ is less likely to appropriate and an increased value may be required. Appropriate values of T_{UB} can be determined in various ways, such as performing a Nonlinear Static Pushover (NSP), to better estimate the expected global displacement ductility at a given performance level. A more sophisticated approach is to perform an initial NLRHA, followed by running an Eigen-analysis of the yielded structure to confirm the elongated first-mode periods. This second approach is more complex and would depend on the manner in which stiffness degradation is formulated in components, and the degraded component stiffnesses being used at the instant of the Eigen-analysis.

The **lower bound period** (T_{LB}) intends to account for higher mode effects. Higher modes are important either for their own response in isolation, or for their contribution to linear or non-linear fundamental behaviour. For typical frame structures, the second mode is approximately 1/3 of the fundamental mode ($0.33T_1$), while for wall structures the second mode is often 1/6 of the fundamental mode ($0.17T_1$). The prescribed lower value $0.4T_1$ does not encompass the higher modes of vibration for either cases, and so perhaps implicitly presumes that the most damage occurs from response in the fundamental mode. The influence of higher mode contributions to the fundamental mode response would then be strongly dependant on the characteristics of the three selected motions outside the scaling range; which could be highly variable. Whilst the authors note this as a short-coming of the method, increasing the scaling range is not necessarily proposed as a solution, as this could be similarly problematic given the other shortcomings (discussed below).

Notwithstanding the above comments, the lower bound period $0.4T_1$ is generally considered to be unsuitable for unreinforced masonry (URM) buildings. Typically, URM buildings do not have well defined global fundamental modes, but rather a large number of local modes. Lower mass participation occurs in a larger number of these modes due to common construction of light-weight flexible timber floor/roof diaphragms and most of the seismic mass being attributed to the heavy URM walls (rather than the light-weight diaphragms). As a rational means of dealing with this shortcoming, the lower bound of $0.4T_1$ can been substituted with $T_{90\%}$ (the period at which 90% mass participation is achieved), or as near as practically achievable.

Prescribed D_1 Limit vs. Spectrum Short-Period Plateau

The specified limit in the Standard is $D_1 \leq \log(1.5)$, yet the commentary states this is *"not very restrictive"* and a better fit based on log(1.3) should be aimed for. A requirement of C5.5.2 (c), vii) is to *"Reject records that are not of reasonable fit"*. The authors note two main reasons where it is not practical to meet the prescribed D_1 limit:

- D_1 values are sometimes found to exceed the limit of log(1.5) where the ground motion response spectra coincides with the 'artificial' (i.e. non-physical) plateau of the target spectrum at short-periods. In the author's experience, this can lead to apparent trade-offs between the prescriptive limits for D_1 and k_1, and the prescribed period range. Holistically this trade-off is considered by the authors to be unimportant, as variability in the procedure is more important to ensure the overall evaluation based on the maximum response of 3 records is not unconservative [23].
- D_1 values are completely dependent on the available ground motions for the causal magnitude-rupture (M_w-R_{rup}) properties which must be notionally consistent with the hazard disaggregation results. For a single ground motion, the spectral shape is not expected to be consistent with the NZS 1170.5 design spectrum, as this represents the predicted ground motion intensity from different dominant earthquakes.

The authors recommend that the causal parameters M_w-R_{rup} are treated as a higher priority, and some leniency in the values of D_1 is reasonable. It is noted there is no equivalent D_1 goodness-of-fit parameter specified in ASCE 7-16, or in Eurocode 8.

Reasons for Large Scale factors:

Typically, the prescribed lower limit of $k_1 > 0.33$ is not problematic, although the upper limit of $k_1 < 3.0$ and/or $k_2 < 1.3$ are often exceeded for a variety of genuine reasons, as noted below.

1. High-importance structures designed for rare ground motion intensity levels

High importance level structures are designed for larger ground motions that have longer corresponding return periods (i.e. 1/2500-years for an Importance Level 4 structure). Ground motion databases have a shortage in historic records for extreme ground motions caused by M_w>7.5-8 events with rupture distances less than 20km from strong motion instruments. These extreme ground motions have not yet been recorded in sufficient quantity, which means that often lower intensity motions must have larger scale factors applied to them in order to meet the target hazard spectrum.

2. Subduction-source ground motions

Relative to databases of ground motions from active shallow crustal earthquakes, a small set of subduction-source records are available in open source databases. Given the complexity of simulating subduction-source ruptures, there is a high degree of reliance on the small amount of subduction event data. Of the subduction records that do exist, the spectral amplitudes are generally very low due to large epicentral distances (between 60-470km), which represents subduction zones that are located further offshore than in the true physical case of the North Island of New Zealand. Due to a shortage of historical (and simulated) data, large individual scale factors (i.e. $k_1 > 3.0$) are a common requirement for subduction-source ground motions.

Across the North Island of NZ, seismic hazard disaggregation typically confirms that subduction sources typically contribute between 20-40% of the total seismic hazard. This essentially means that a suite of 3 records total should contain 1 subduction-source record. Similarly, for a suite of 11 records, there will typically be 3 or 4 subduction-source records.

3. Inconsistencies between the NZS 1170.5 design spectrum and the site-specific seismic hazard spectrum

As eluded to earlier, the ground motion selection methodology (i.e. "Task 1") generally requires that records are selected based on the site-specific seismic hazard disaggregation, rather than the NZS 1170.5 design hazard spectrum. The rupture scenario information from the hazard disaggregation do not always guarantee the ground motion response spectra provide a good match with the NZS 1170.5 design spectra.

The prospective record selection process involves some initial scale factors to confirm (in-advance) that records do not have excessively high scale factors when scaled to the NZS 1170.5 design spectrum (i.e. “Task 2”). The inherent shape of the site-specific hazard spectrum, and differences with the NZS 1170.5 design spectrum both influence final scale factors, as will become evident in the following two example issues.

Firstly, across NZ there are several regions of relatively low-moderate seismicity (i.e. $Z \leq 0.20$) which the site-specific hazard spectra is often significantly lower than the NZS 1170.5 design spectra. An example is shown in

Figure ***3*** for Auckland [19]. The NZS 1170.5 design spectra for Auckland is approximately 2.5 times larger than the site-specific hazard spectrum. There is a relatively linear difference in spectral amplitude across all vibration periods. In effort to remain close to the NZS 1170.5 scale factor limits, the most practical work-around is to impose intentionally low scale factors during the initial selection of prospective ground motions. Despite adopting this work-around, it is not uncommon for some individual scale factors k_1 to be between 4 and 6.

Secondly, a slightly different scaling-affected issue arises for scenarios where there is a significant difference between the inherent spectral shape of the site-specific hazard spectrum and the NZS 1170.5 design spectrum. An example of this issue is shown in

Figure ***4*** for a Site Class C site in the Wellington CBD. This particular site does not have any basin-edge effects. The apparent “mis-match” in spectral shape shown in

Figure ***4*** generally results in abnormally high scale factors, as the scaling factor becomes governed by the “extremities” of the period range. In this example, the upper bound period (T=2-3sec) will govern the scaling, while at all other periods the response spectra may be significantly larger than the NZS 1170.5 target.

When this mis-match in spectral shape occurs, moderate-long period structures are most affected because the prescribed period range is large in an absolute sense. Overall, this issue is irrespective of the NZ (and US) scaling procedures, as it is the adequacy of the NZS 1170.5 design spectrum which is the governing factor.

In both of these cases, it may be beneficial to adopt the site-specific hazard spectrum as an alternative solution to the NZS 1170.5 spectrum, subject to the discussion on limitations of their use provided earlier. This option may not be available for Engineering Assessments completed strictly under the EPB legislation (for statutory purposes), as those ratings are calibrated specifically to the ULS shaking demands as set out in the current version of NZS 1170.5:2004.

4. NZSEE (2017) Assessment Guidelines – Tier 3 Assessment based on ASCE 41 at Collapse Prevention

When the alternative verification methodology outlined in C1.6.2 of the *Assessment Guidelines* is followed, and ASCE 41 is adopted, there are additional requirements to assess the Collapse Prevention (CP) performance level. The requirement for undertaking a specific CP evaluation is required only for the ASCE 41 alternative methodology, which means a CP evaluation is not required if the NZSEE *Assessment Guidelines* are followed in their entirety.

For ground motion input for ASCE 41 evaluation, the *Assessment Guidelines* state that the same suite of ground motions is to be used for the evaluation of both the Life Safety (LS) and the CP performance levels. For the CP evaluation, the ground motion suite is to be linearly scaled up by the factors given in Table C1.1. When following this approach, it is not uncommon (in the authors experience) to find scale factors be greater than 10 for some individual records for the CP evaluation. For some individual records with large velocity-pulses, the amplification by the additional scale factor for CP evaluation may lead to an overly severe input ground motion.

From a pragmatic viewpoint, even if ground motion characteristics are mis-represented by large scale factors, the general intent in assessing and ensuring a significant margin to collapse beyond ULS shaking is probably achieved, though likely at the expense of even greater uncertainty in a reported building score which commercial decisions may be based. The authors have been involved in cases where ULS suites have been factored for CP, and also in cases where specific suites better suited to the higher return periods have been obtained instead. The latter comprised situations where project outcomes were materially affected by the CLS score to the point that proceeding with such high scale factors on the LS-suite of ground motions could not be justified.

Caution is recommended when following this requirement stated in Table C1.1 of the Assessment Guidelines.

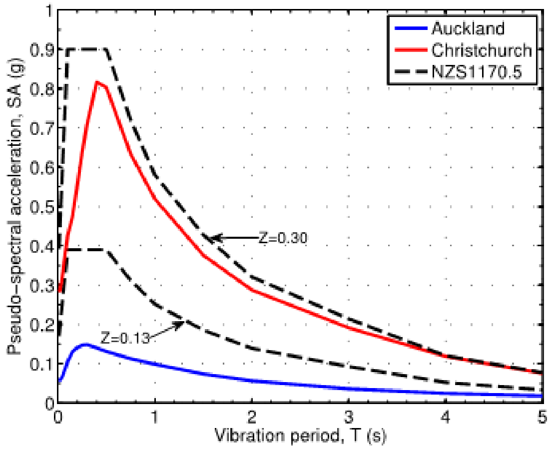

Figure 3: Site-specific UHS for 1/ 500 year return period at site class D sites in Auckland and Christchurch compared to the NZS 1170.5 design spectrum (from [19]).

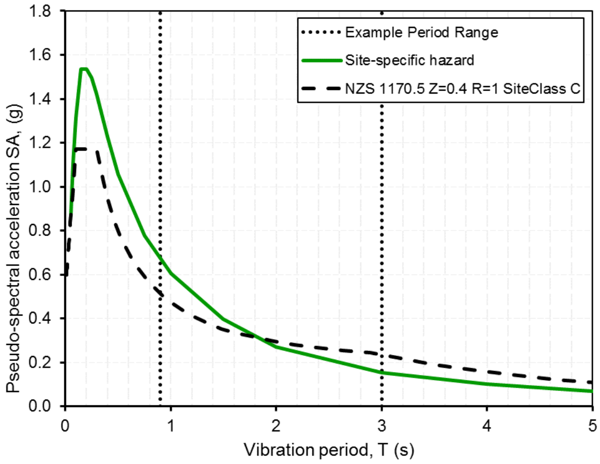

Figure 4: Site-specific UHS for the 1/500 year return period at a site class C site in Wellington (with no basin-edge effects).

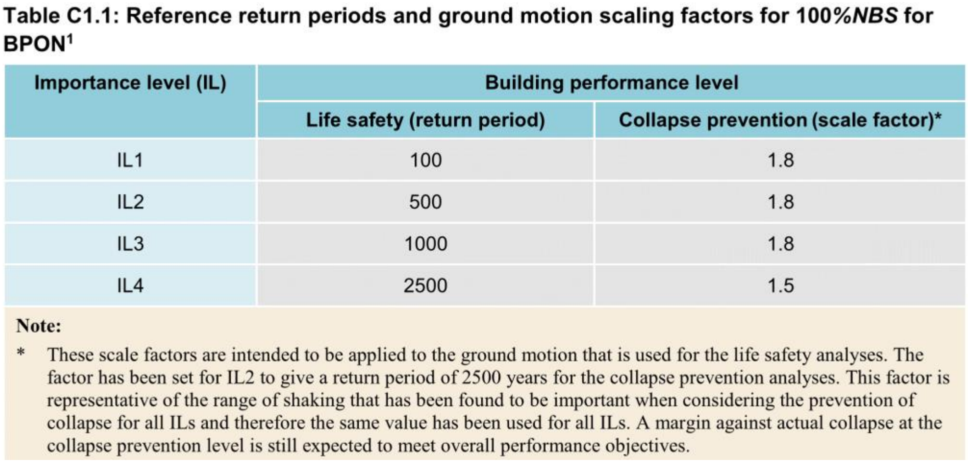

Table C1.1: Reference return periods and ground motion scaling factors for 100%*NBS* for BPON[1]

Importance level (IL)	Building performance level	
	Life safety (return period)	Collapse prevention (scale factor)*
IL1	100	1.8
IL2	500	1.8
IL3	1000	1.8
IL4	2500	1.5

Note:

* These scale factors are intended to be applied to the ground motion that is used for the life safety analyses. The factor has been set for IL2 to give a return period of 2500 years for the collapse prevention analyses. This factor is representative of the range of shaking that has been found to be important when considering the prevention of collapse for all ILs and therefore the same value has been used for all ILs. A margin against actual collapse at the collapse prevention level is still expected to meet overall performance objectives.

Figure 5: Extract from NZSEE Assessment Guidelines [4] for Collapse Prevention Scale Factor.

SUMMARY OF ASCE 7-16 & ASCE 41-17

Background

ASCE 7-16 and ASCE 41-17 are the current seismic loading and design standards developed by the American Society of Civil Engineers (ASCE). The two standards neatly reference one another.

ASCE 7-16 *"Minimum Design Loads for New Buildings*" contains the most contemporary and non-prescriptive guidance available for ground motion input for NLRHA. In US practice, the Life Safety performance of new buildings is typically addressed using MRSA for design. NLRHA is performed specifically to assess Collapse Prevention (CP) at the risk-targeted maximum considered event (denoted as MCE_R).

The theory and application of ASCE 7-16 is presented in detail in references: [28, 29, 30, 31]. In Section 16, the ground motion requirements in ASCE 7-16 contain substantial changes from the predecessor standard, ASCE 7-10 [14]. The most significant changes include increasing the minimum number of ground motions from 7, to 11; and; changing the ground motion intensity measure from the square-root sum of the squares (SRSS) spectral acceleration to the maximum direction spectral acceleration (often denoted as *RotD100*). The use of RotD100 is a favourable simplification from past approaches, as it allows "Task 2" ground motion scaling to be independent of directionality/polarity. In the ASCE 7-16 framework, directionality is now addressed within requirements of Section 16.2.4 (i.e. "Task 3").

ASCE 41-17 *"Seismic Evaluation and Retrofit of Existing Buildings*" includes evaluations of both Life Safety and Collapse Prevention, along with other performance objectives (as outlined in Sections 2.2 and 2.3 of the standard). For Tier 3 evaluations using NLRHA, the ground motion input requirements cite those in Section 16.2 of ASCE 7-16, along with some modifications to better represent existing buildings (discussed below).

For context, it is worth noting that the current 2016 California Building Code (CBC) currently cites the respective forerunners standards; ASCE 41-13 and ASCE 7-10, and it's anticipated that the latest standards will be cited in the 2019 CBC. This means that, at the time of writing this paper, consulting engineers in California have varying (and often limited) practical experience with applying the ground motion requirements in ASCE 41-17 and ASCE 7-16.

Mean Response of a Minimum of 11 Records

Earlier versions of ASCE 7 permitted the use of either the "mean of 7, or maximum of 3" ground motions, with practitioners preferring to use the "mean of 7". In US practice, it was not uncommon for consulting engineers to increase the number of ground motions to 10 (or more) in order to improve the overall reliability of the NLRHA outputs. More recently, it became apparent that the mean of 7 ground motions may be un-conservative [18], while a minimum of 11 greatly increases the confidence of seismic response predictions, without a significant increase in computational effort [2, 21].

Whilst using a minimum of 11 records is perceived to be a major increase in the required computational workload relative to NZS 1170.5, it is offset by the fact that records only need to be applied once to the computational model. Compared to the NZS 1170.5 requirement in Section 6.4.4, the records would be applied twice with the primary record components flipped in orthogonal directions, meaning six analyses are required. Ultimately this means using 11 records per ASCE 7-16 only increases the minimum level of computational effort from NZS 1170.5 (i.e. using 3 records) by a factor of two.

Section 16.2.2: Ground Motion Selection

Section 16.2.2 of ASCE 7-16 outlines the requirements for ground motion selection. Further guidance is provided in the commentary section C16.2.2, with reference to NIST GCR 11-917-15 "*Selecting and Scaling Earthquake Ground Motions for Performing Response-History Analysis"* [32]. This NIST guidance document was originally prepared to address the needs from the profession to have *"improved procedures for selecting and scaling earthquake ground motions for performing response-history analysis* [32].*"*

Peer review of seismic hazard analysis and ground motion selection varies in the US. Some authorities having jurisdiction have preferred peer reviewers, while others rely on the peer reviewer to be nominated by the consulting engineers within the project team.

Section 16.2.3: Ground Motion Modification (Scaling)

Compared to NZS 1170.5, the US standards contain fewer prescriptive requirements. There are no specific limits on scale factors as a basis for rejecting records. Scale factors between 0.25-4.0 are recommended as initial target values [28], though the final scale factors are treated overall as being less important than the causal M_w-R_{rup} properties, along with maintaining other ground motion intensity measures (such as duration).

There is no goodness of fit parameter (like D_1 which exists in NZS 1170.5).

The defined period ranges of interest are shown below for both ASCE 7-16 and ASCE 41-17, respectively:

	Period range for scaling calculations	
Standard	**Lower bound, T_{LB}**	**Upper bound, T_{UB}**
ASCE 7-16	min($0.2T_{min}$, T_{90})	$2.0T_{max}$[a]
ASCE 41-17	$0.2T_{min}$	$1.5T_{max}$[b]

T_{min} = minimum value of $T_{1,x}$ and $T_{1,y}$ (the fundamental period in two orthogonal directions).

$T_{90\%}$ = the period at which 90% mass participation is obtained (mass of the super-structure) in two orthogonal directions.

T_{max} = the maximum fundamental period (including both translational and torsional modes).

[a] The upper bound period in ASCE 7-16 can be reduced, but not less than $1.5T_1$, provided there is justification based on the response reported from the NLRHA at the MCE_R level of shaking.

[b] The upper bound period shall not be less than 1.0 sec

The ASCE 7-16 period range is generally consistent with Eurocode 8 (Part 1) but, in contrast, is evidently much larger than that specified in NZS 1170.5. At this point it is important to clarify that, the degree of displacement ductility that is adopted in US seismic design is comparable to NZ (a common misconception is that structures in the US are designed for larger levels of ductility). As shown, ASCE 41-17 relaxes the upper-bound period on the basis that existing buildings do not develop the same degree of global ductility compared to a new designed ductile structure.

Section 16.2.4: Application to Computational Model

For "far-field" sites, the as-recorded horizontal components applied to the model shall not introduce a directional bias greater than ±10% in each orthogonal direction. This requirement is favourable as it means records do not need to be applied twice with the primary record components flipped (as is common in NZ practice to NZS 1170.5). This requirement represents the fact that the record-to-record variability in the suite of 11 records is far greater than the variability associated with flipping the horizontal components, therefore there is little-no benefit in performing 22 analyses compared to 11 analyses which meet the ±10% rule.

Figure 6 illustrates the checks for limiting directional bias. Firstly, Figure 6(a) represent the case of having all of the as-recorded horizontal ground motion components H1 applied to the X-direction of the computational model (and thus all as-recorded H2 components are applied to the model Z-direction). Figure 6(b) plots the corresponding ratio of the mean X-direction spectra (and Z-direction spectra, respectively), divided by the mean of all horizontal components (i.e. the mean of all H1 and H2). The maximum directional bias shown in Figure 6(b) is 15%, occurring at T=0.8sec, while the bias exceeds 10% over the wider period range of 0.55 to 0.95sec. Overall, the requirements of 16.2.4 are not met.

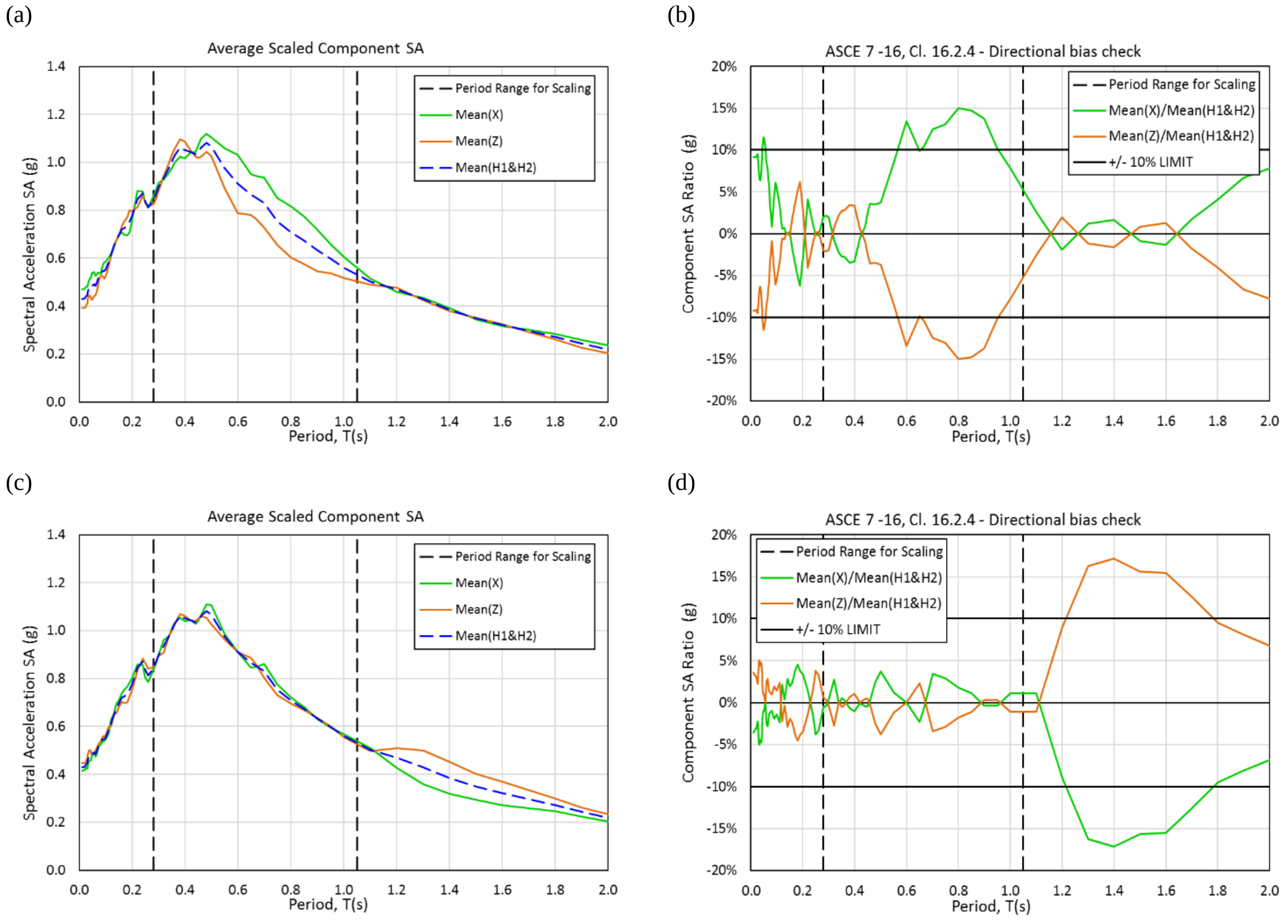

Figure 6: Directional bias checks to 16.2.4 of ASCE 7-16 for: (a)-(b) all as-recorded H1 components applied to the model X-direction, and; (c)-(d) the most optimized configuration of the as-recorded H1 and H2 components for the model X- and Z-directions (in this case, Y-direction = vertical).

Secondly, for the same suite of ground motions, Figure 6(c) shows the application of the ground component pairs H1 and H2 after being fully optimised in terms of determining the X- and Z-direction application to the computational model. The corresponding Figure 6(d) shows that the maximum directional bias has been reduced down to 3.7%, within the period range of interest. In the experience of the first author, achieving a directional bias of about 3-4% is routinely possible for suites containing 11 records, but this generally increases to 5-6% for larger overall period ranges. Meeting the 10% criteria is practically straightforward. Figure 6(c) and (d) also shows that, in order to minimize the directional bias within the period range of interest, there is an increased bias for T>1.2seconds. Within the framework of ASCE 7-16 this is considered as unimportant, due to being beyond the prescribed period range of interest.

For “near-fault” sites; horizontal components are to be rotated to the fault-normal (FN) and fault-parallel (FP) directions of the causative fault and be applied to the building in such orientation. As noted earlier, the rotation of as-recorded motions can have a marginal increase the spectral accelerations in FN orientation, though the main effect of this rotation is to represent the large velocity pulses which tend to occur near-fault. There are some practical challenges associated with these “near-fault” requirements, as discussed in the following section.

LIMITATIONS OF ASCE 7-16 AND ASCE 41-17 PROCEDURES FOR GROUND MOTION

The primary focus of this paper is to discuss NZ practice, rather than provide a detailed examination of ASCE 41-17 and ASCE 7-16. However, given that these standards may be adopted for NZ projects (particularly for assessments using ASCE 41), this section provides some notes of caution.

Short Period Buildings and 1.0 sec Minimum Upper Bound

The one notable limitation in ASCE 41-17 that affects short period buildings is the unique requirement of setting a minimum upper bound period (T_{UB}) of 1.0 sec. As the conventional value for T_{UB} is $1.5T_1$, evidently this requirement influences buildings where $T_1 \leq 0.66$ sec. The relevant discussion in commentary is extracted below:

> *“… to prevent underestimation of the period of stiff, short period buildings where significant period elongation due to structural softening and soil–structure interaction may significantly increase the effective period of response.”*

This brief commentary is relatively vague and does not provide much technical insight. It is unusual that this requirement appears only in ASCE 41-17, and not in ASCE 7-16 (for new buildings). The reliability of this ASCE 41-17 requirement was tested via a NLRHA case study, of which the main conclusions are summarised below.

The computational model represents a five-storey reinforced concrete shear wall building constructed in Christchurch (NZ) in 1989. The building is founded on shallow mat slab foundations. Soil-structure interaction was represented by modelling non-linear soil springs. The building’s largest translational periods are $T_{1,x}$ = 0.53 sec, and $T_{1,y}$ = 0.48 sec, respectively. NLRHA was completed using twelve input ground motions selected from the PEER NGA database. The response spectrum scaling and subsequent NLRHA was repeated for two cases; first $T_{UB} = 1.5*(0.53) = 0.80$ sec, and secondly T_{UB} =1.0 sec. For the latter case, the ground motion scale factors increased by approximately 10 percent.

At the completion of each NLRHA case, an Eigen analysis was performed on the damaged structure to evaluate the extent of period lengthening. The ratios $T_{lengthend}/T_{1,max}$ were compared; the first case equal to 1.42, and the second equal to 1.44. These results are not markedly different from one another, and both align with T_{UB} being equal to $1.5T_{max}$. The difference in building drifts was approximately 5-12 percent (across each storey), and force-controlled components were directly affected by the 10 percent increase in ground motion scale factors. Overall, this study showed the minimum T_{UB} = 1.0 sec can add an unnecessary additional challenge for the seismic retrofit of existing buildings.

Long Period Buildings: Case Example

The effect of large period ranges on scale factors is presented through a case study example for a relatively slender high-rise building in Auckland (NZ). The building is defined as an Importance Level 2 structure, located on a Site Class B site. The largest period T_{max} is 6.4 sec, with 5.9 sec in the orthogonal direction. The resulting period range, based on ASCE 41-17, is 1.2 to 9.6 sec.

Due to the period range being so large, the initial response spectrum scaling of ground motions is governed by the relative short-fall in the mean spectra at the band of periods from 1.2 to 2.0 sec, as can be seen in Figure 7. To overcome this response spectra “shortfall” towards the lower bound period, an initially large family scale factor of k_2 = 2.40 was calculated. At the fundamental periods, the ratio of scaled response spectrum to the code target spectrum, $SA(T_1)_{mean}/SA(T_1)_{code}$, is approximately 2.4, hence the initial ground motion selection and scaling is overly conservative and not suitable for the evaluation.

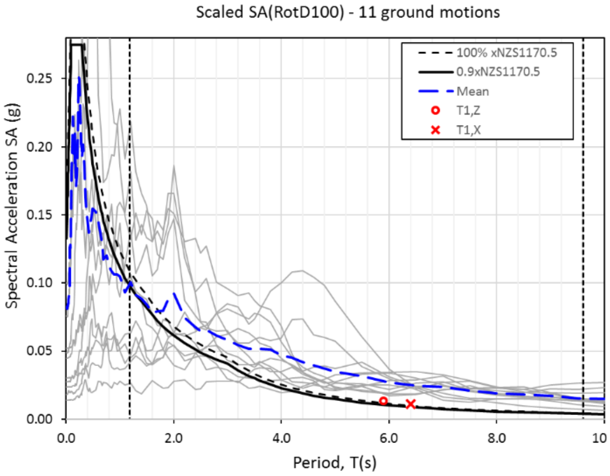

Figure 7: Scaled response spectrum with the mean spectra governed by the lower bound period, $0.2T_{min}$.

To mitigate the issues with the initial scaling, the record suite was expanded to 13 records, with two supplementary records with increased response spectra for T = 1.2 to 2.0 sec. The re-scaled response spectrum is shown in Figure 8. The k_2 scale factor reduced to 1.3, and at the fundamental periods the ratio $SA(T_1)_{mean}$ /$SA(T_1)_{code}$ was reduced to approximately 1.4. Although not shown, the corresponding spectral displacement demands at long periods were significantly reduced from the initially excessive values.

Increasing the number of input ground motions had relatively few implications for this particular project. As the structure did not exhibit much material non-linearity, analysis run times were not a constraint. The amplification of higher modes in the additional ground motions were not problematic for this particular structure.

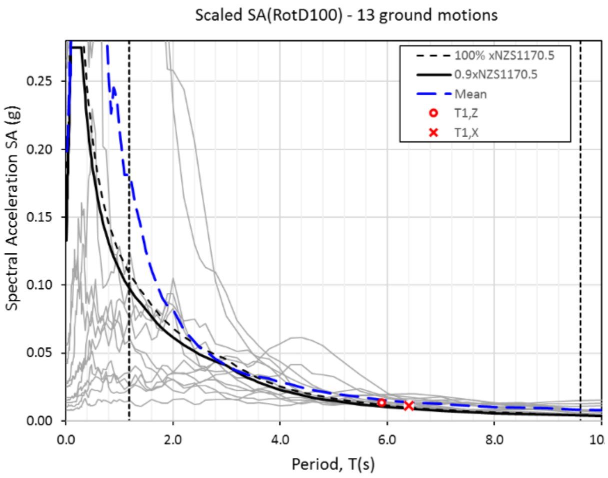

Figure 8: Scaled response spectrum with the mean spectra governed by the lower bound period, 0.2T_{min}.

Uniform Hazard Spectrum (UHS) vs. Multiple Scenario Spectrum

The uniform hazard spectrum (UHS) is generally considered to be a conservative target spectrum due to the UHS being an enveloped result from the PSHA over the entire range of vibration periods. For example, say that moderate magnitude earthquakes at close distance dominate the hazard at shorter periods, and larger magnitude earthquakes at further distance dominate the hazard at longer periods. In this example, the UHS envelopes both earthquake scenarios. Thus, by definition, the UHS produces a target design spectrum that is not representative of any single earthquake scenario. The UHS is considered to be practical for linear static and modal response spectrum analysis methods. However, the UHS can be problematic for NLRHA when selecting a prospective ground motion based on a "seismologic signature", and then scaling the ground motion to a target UHS that envelope multiple seismologic signatures.

Particularly for long period buildings, such as base-isolated and tall buildings (such as the case example in Figure 8), the scale factors can become very large due to the very broad period range. Reducing conservatism in the overall NLRHA can be alleviated by replacing the UHS with the "multiple scenario spectra", as is referred to as Method 2 in ASCE 7-16. This approach is more commonly known as the Conditional Mean Spectra, or Conditional Spectra [11]. From a practitioner's perspective, there are several advantages and disadvantages to using the multiple scenario spectra in lieu of the UHS. The main advantage is reducing some apparent conservatism in the seismic hazard, and the main disadvantage is additional computational effort and iterations required before the final NLRHA. Using Method 2 on NZ projects would be an alternative solution to the NZBC, which adds regulatory challenges and requirements for peer review.

In the authors opinion, using the multiple scenario spectra is better suited to regions where dominant earthquake events are clearly distinguished, such as moderate and high seismic regions. Considering the previous discussion of the Auckland case study, this region is of relatively low seismicity, which means the scenario hazard is not well defined, unless the scenario used to set the minimum Z value for Auckland were to be used. However, there is enough uncertainty in other sources that this is not necessarily a clearly governing scenario either. Auckland also carries proportionally high risk due to the greater magnitude of social and economic consequence from a large event, so maintaining some conservatism by using the UHS is considered a prudent approach.

"Near-Fault" Requirements for Fault-Normal/Fault Parallel Ground Motion Pairs.

In the authors opinion, the near-fault requirements in Cl 16.2.4 have some practical limitations worth noting:

1. Rotating the entire suite of records to be FN/FP may be inadequate, on the basis that only a sub-set of the selected motions will contain pulse-like characteristics. Internationally, it is common practice, but not mandatory, to rotate only the pulse-like motions.
2. Many high seismicity regions have several causative faults in the near-fault region that have high relative contributions to the total seismic hazard. In such cases the fault orientations may vary, which may justify performing NLRHA with a greater number of ground motion application angles to represent more than one near-fault scenario.
3. Forward-directivity effects are better understood for shallow crustal earthquake hazards, whereas near-fault subduction hazards are more complex. In Wellington (NZ), the subduction hazard is off-shore, and there is a relative mis-alignment of slip direction, rupture propagation direction, and source-to-site distances in the relevant directions. For practical applications, the as-recorded subduction ground motions may be more suitable (in lieu of the prescribed fault-normal / fault-parallel motions).
4. Basin and topographic effects in near-fault regions (such as Wellington [28]) is a complex subject that influences seismic wave propagation. The uncertainties associated with basin and topographic effects may be greater than the near-fault requirements.

"ABOVE MINIMUM" REQUIREMENTS FOR TALL BUILDINGS

So far, this paper has discussed the minimum requirements of NZ and US seismic loading standards. Over the past decade, there has been greater emphasis internationally on Tall Building seismic safety/resiliency initiatives. In these cases, advanced engineering designs/assessments may carry higher risk, due to greater consequence.

For tall buildings in the US, engineers are required to go beyond the minimum requirements of ASCE 7 and use more advanced performance evaluations based on guidelines such as the PEER Tall Building Initiative (TBI) [34], and guidelines by the Los Angeles Tall Buildings Structural Design Council (LATBSDC) [35]. Both documents essentially adopt the principles of ASCE 7, with the addition of more stringent criteria to ensure more reliable performance of tall buildings. In terms of document hierarchy, the PEER-TBI document sits above ASCE 7-16, while LATBSDC requirements are more onerous than those in PEER-TBI. Authorities having jurisdictions may voluntarily choose to adopt these guidelines in addition to enforcing ASCE 7, or ASCE 41.

Examples of the ground motion requirements for tall buildings, which are additional to ASCE 7-16, are briefly summarized below:

- For Risk Category III buildings (equivalent to IL3 within NZS 1170), the PEER TBI states that one unacceptable result is permitted if the suite contains not less than 20 ground motions. The LATBSDC document does not permit any unacceptable responses.
- Performance at the Serviceability level earthquake (SLE) must be verified by response history analysis (RHA). The SLE has a probability of exceedance of 50% in 30 years, which corresponds to a 1/43-year return period. The response spectra for the SLE level of shaking shall be based on 2.5% critical damping. PEER-TBI requires a minimum of 7 ground motions if NLRHA is performed (taking the mean response), or alternatively a Linear RHA may be

performed using a minimum of 3 ground motions (taking the maximum response). LATBSDC recommends a minimum of 11 records for NLRHA at the SLE level.

Both PEER-TBI and LATBSDC guidelines have the same requirements as follows:

- Ground motion significant durations (i.e. $D_{5\text{-}95\%}$) shall be reported for peer reviewers. Long-period structures require longer significant duration to develop full dynamic excitation, therefore the ground motion records must be of a sufficient duration of strong shaking.
- For sites where pulse-type motions must be considered in the suite of motions, not less than five records in the pulse and non-pulse subsets shall be used.
- A "multi-source" requirement exists where a minimum of five ground motions is required to represent any "category" of hazard which has similar magnitude and rupture combinations that contributes at least 20% to the total seismic hazard.

Evidently, to satisfy last two requirements above, the total number of ground motions may need to be increased.

OTHER CONSIDERATIONS NOT WITHIN THE SCOPE OF THIS PAPER

Frequency-scaling procedures, or so-called "spectral matching", is not permitted in NZS 1170.5. Despite common usage historically in US practice, some concerns remain regarding the unintended modification or removal of important characteristics and reduced record-to-record variability. This may lead to unconservative performance evaluations—particularly for near-fault regions. Some additional mitigating requirements for spectral matching were introduced in ASCE 7-16 and in ASCE 41-17 advising caution to these issues. However, the clauses are open-ended, and it is not immediately clear how they might be fulfilled. This method of scaling was therefore not discussed in this paper.

Simulated ground motions are not discussed in this paper, although these will become important in the future. Despite being permitted by NZS 1170.5, conventional practice has been to use historic as-recorded motions. Further discussion can be found in [36].

This paper does not deal with considerations related to risk-targeted demands or risk-adjusted hazard (which have been used in US standards since 2010). For the purposes of this paper, this is simply interpreted as an alternative means by which to set seismic demands which ground motions can be then be scaled to. The distinction between targeting uniform risk in lieu of uniform hazard could be relevant in a more detailed examination than this paper pursues.

CONCLUSIONS

This paper summarised the ground motion selection and scaling requirements of the NZ loading standard NZS 1170.5. Some of the salient limitations that often need to be addressed on an ad-hoc basis when conducting NLRHA were discussed within.

As NLRHA is becoming increasingly popular, there will be greater needs to have contemporary guidance available for determining the ground motion input. This paper may assist with fulfilling the interim needs of practicing engineers as it promotes greater awareness and discussion of this subject matter within the NZ engineering community. It will be beneficial for future publications to include some of the topics that were excluded scope of this paper (such as simulated ground motions, for example).

This paper discussed the following specific issues for NZ practice:

- Ground motion selection and scaling requirements in NZS 1170.5 (2004) are relatively out-dated; however these have been cited in more modern documents such as the NZSEE (2017) *Assessment Guidelines* for existing buildings, and the Draft NZ Base-Isolation guidelines (currently in development).
- Self-regulation of seismic hazard analysis and ground motion selection is the current norm in NZ practice. The requirements for ground motion selection are relatively open to interpretation, whereas it would be beneficial to have an accepted framework procedure.
- Scaling and applying records to the computational model may require leniency from the prescriptions given in NZS 1170.5, in favour of more fundamentally important factors such as causal magnitude and rupture distances. This paper discussed common limitations for the prescribed period range for scaling calculations, the goodness of fit rules, and genuine reasons for which large scale factors cannot be easily avoided.
- The US standards ASCE 7-16 and ASCE 41-17 were summarised in this paper. These standards offer an attractive alternative to NZS 1170.5, as these standards were developed based on relatively modern research.
- More onerous requirements for tall buildings were also briefly summarised to help promote awareness of the "above-minimum" requirements which exist in the US.

ACKNOWLEDGMENTS

The motivation for this paper stemmed from numerous discussions with engineers from Holmes Consulting (NZ) and Holmes Structures (US). Dan Bech (Holmes Structures) is thanked for sharing his case study nonlinear response history analysis work to trial the ASCE 41-17 scaling provisions for short period buildings.

The first two authors wish to acknowledge their discussions with Karim Tarbali (previously at University of Canterbury, currently at Politecnico di Milano), and with last author, whilst working on an array of NZ projects where nonlinear response history analysis has been carried out. Two anonymous reviewers are also acknowledged for their constructive comments on this paper.

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