DESIGN GUIDE

Nonlinear Response History Analysis

Design Guide
Nonlinear Response History Analysis

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Public Comment Draft

STRUCTURAL ENGINEERING SOCIETY NEW ZEALAND

Important Notice

This document is a preliminary version issued for industry comment. Submissions by practitioners to the editors are encouraged. The contents may be subject to further changes, additions, and deletions.

The latest version is available from the SESOC website:

https://www.sesoc.org.nz/design-resources/guidelines

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STRUCTURAL ENGINEERING SOCIETY
NEW ZEALAND

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Brendon Bradley – University of Canterbury

Reagan Chandramohan – University of Canterbury

Andreas Giannakogiorgos - Tetra Tech Coffey

Tony Holden - Aurecon

Rob Jury – Beca

Nick Kerrigan – WSP

Gareth Morris - Holmes

Stuart Oliver – Holmes

Tony Stuart - Compusoft Engineering

Timothy Sullivan - University of Canterbury

The contribution of time and expertise from members of the working group is gratefully acknowledged.

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1 Introduction

1.1 Purpose and Scope

The nonlinear response history analysis (NLRHA) provisions contained within NZS 1170.5 (SNZ, 2004) were developed more than 20 years ago. Significant developments in the selection and scaling of ground motions have occurred since the standard was originally drafted which means that the nonlinear response history analysis (NLRHA) provisions within NZS 1170.5 are out of date (Morris et al., 2019).

This guideline draws upon knowledge that has been gained since NZS 1170.5 was published and sets out a contemporary approach for establishing earthquake design actions in structures when using NLRHA. Using updated seismic hazard information contained in TS 1170.5 (SNZ, 2024) it provides recommendations for selecting and scaling ground motion records. This guideline also provides recommendations for modelling structural elements and detailed criteria for evaluating seismic performance. It also provides requirements and guidance for determining horizontal design actions for parts of structures and non-structural components.

<u>Commentary:</u>

The procedures recommended in this guideline for undertaking ground motion selection and scaling, modelling and analysis of structures, and the evaluation of their seismic performance, closely follow the relevant requirements in ACSE 7-22 and supplemented with ASCE 41, PEER TBI and LATBSDC guidelines. It is recommended that designers also refer to these documents for guidance on topics not specifically addressed in this guideline.

1.2 Building Code Compliance

This guideline provides a methodology to assist designers in meeting the relevant Building Code performance requirements when NLRHA is used to validate the seismic performance of structures. It should be noted that NLRHA is specifically excluded from the Verification Methods used to demonstrate compliance with the Building Code and as such this guideline would constitute an Alternative Solution.

2 Design Process

2.1 Overview

This chapter provides an overview of a rational design process that can be adopted when using NLRHA to validate the seismic performance of structures.

NLRHA can also be used as a design tool to establish the configuration and proportions of a structure or to verify that an already obtained structural solution meets the required design criteria including, where appropriate, that capacity design principles have been achieved.

NLRHA should not be considered as a response predictor, it nevertheless is the most mathematically correct and physically consistent analysis tool available to the structural engineer. It provides a global estimation of the plastic behaviour of the structure that is equally applicable for the analysis/design of dynamically symmetrical or non-symmetrical structures.

2.2 Confirm NZBC Compliance Pathway

The use of NLRHA to establish earthquake design actions in structures is specifically excluded from NZ Building Code (NZBC) Verification Method B1/VM1 and will therefore be considered an Alternative Solution. An alternative compliance pathway demonstrating how the design process complies with NZBC Clause B1 should be clearly defined in the project Design Features Report (SESOC, 2021).

This guideline recommends an independent peer review process to validate the design complies with the requirements of Building Code Clause B1. Furthermore, it is recommended the peer review process is initiated early to help control the risk of changes later in the design phase.

<u>Commentary</u>

NLRHA is an advanced form of structural analysis with greater design complexity when compared with linear elastic analysis techniques such as the Equivalent Static and Modal Response Spectrum methods. Application of nonlinear analysis software for use with NLRHA and interpretation of the results require considerable judgement that falls outside the bounds of competence typically required for conventional building design (PEER, 2017).

It is recommended the alternative compliance pathway be clearly defined and documented in the Design Features Report early during a project life cycle and that this guideline be adopted as part of that alternative compliance pathway. The peer review process should be initiated as early in the design process as reasonable. Early discussion and agreement of the alternative compliance pathway, related fundamental design decisions, assumptions and approaches, will help avoid changes later in the design process that could affect both project costs and schedules.

2.3 Establish Performance Objectives

Select design performance objectives and design criteria appropriate to the AS/NZS 1170.0 (SANZ, 2002) Importance Level (IL) and any project specific performance objectives when these exist.

Importance Levels are as defined in AS/NZS 1170.0 and are based on structure function and occupancy. The Importance Level dictates the building performance limit states that should be considered when designing structures, and the required annual probability of exceedance (APoE) for earthquake design actions. Performance limit states that should be considered when designing structures include:

SLS1 Serviceability Limit State 1 in accordance with TS 1170.5 for the Importance Level considered.

SLS2 Serviceability Limit State 2 in accordance with TS 1170.5 for Importance Level 4 structures only.

DCLS Damage control limit state is non-mandatory limit state at which damage to the building is controlled so there is a low probability of damage leading to significant economic loss.

ULS Ultimate limit state in accordance with TS 1170.5 for the Importance Level considered.

CALS Collapse avoidance limit state at which collapse of the structure is to be prevented with reasonable reliability in accordance with the requirements of the NZBC.

NZBC B1/VM1 requires consideration of SLS2 for Importance Level 4 structures only.

The design performance objectives and design criteria selected for a project should be clearly defined in the Design Features Report.

<u>*Commentary*</u>

NZBC Verification Method B1/VM1 has no requirement to explicitly consider CALS. This is because the margins inherent within the ULS design procedures implemented within NZ materials standards are expected to provide sufficient confidence that acceptable collapse and fatality risks are achieved. This includes seismic detailing provisions, and the application of capacity design procedures for structures that are expected to respond beyond their strength limit when subjected to earthquake shaking corresponding to ULS.

Ideally, the CALS performance evaluation prescribed herein would ensure that the annual probability of collapse due to earthquake shaking is acceptably low, in accordance with the requirements of the NZBC. However, as there is currently no data available to inform the appropriate definition of scaling factors ψ_{CALS} *and* S_p*, and given the uncertainty in actual structural deformation capacities, the fatality risk (linked to the annual probability of collapse) of buildings that results from application of these NLRHA guidelines is unknown. Until new data becomes available, these guidelines recommend consideration of CALS, in line with the NZSEE Draft Seismic Isolation Guidelines (NZSEE, 2019), in order to satisfy the intent of the NZBC to limit the annual fatality risk due to collapse (refer further discussion in Section 2.5).*

Consideration of SLS2 for other than Importance Level 4 structures would be considered a voluntary project specific performance objective.

2.4 Design Methodologies

Validation of building performance for SLS1 earthquake design actions can typically be done by means of a conventional Equivalent Static or Modal Response Spectrum analysis undertaken in accordance with TS 1170.5.

This guideline recommends validating building performance at CALS as the primary means of demonstrating compliance with the life safety performance requirements in NZBC Clause B1. Validation should be by means of a NLRHA in accordance with the methodology detailed in Section 4.

When projects require consideration of structural performance at SLS2 or DCLS, the analysis methodology for validation will need to consider the level of nonlinearity expected in the structure at these limit states. When significant nonlinearity is expected, or when the lateral load resisting system is not provided for in NZBC B1/VM1, a NLRHA is recommended.

Appendix C summarises a design process which can be adopted for projects when NLRHA is to be used to validate the seismic performance of new structures. The process aligns with recommended industry practice (NZSEE, 2022 and SESOC, 2022), whereby designers should deliberately proportion structures with enough regularity so that it is possible to identify a clear plastic mechanism. This will enable capacity design principles to be applied, so that should a structure's strength be exceeded, reliable plastic mechanisms can be developed.

It is acknowledged that this is not the only process that can be adopted to justify an Alternative Solution but is offered as an aid to designers. This document does not preclude designers from adopting other NLRHA design methodologies, including those where Building Code life safety performance requirements are primarily assessed at the ULS, provided the alternative methodologies ensure an appropriate margin beyond ULS is achieved, and adequate consideration is given to modelling uncertainty and ground motion record to record variability. Application of Building Code B1/VM1 capacity design procedures might be considered an acceptable method to provide an appropriate margin beyond ULS.

Commentary

When considering SLS1 earthquake design actions on a structure, TS 1170.5 limits the structural ductility factor, μ, to not greater than 1.25. While it is permitted to use NLRHA to validate structural performance at SLS1, the limited level of ductility demand able to be considered dictates that the structure is expected to respond in a near elastic manner and as a result simpler linear analysis techniques are likely to be adequate for conventional structures.

Equivalent Static or Modal Response Spectrum analysis may not be appropriate for validating structural performance at SLS1 for buildings with supplemental damping systems and other analysis methods such as NLRHA might be more appropriate. When using NLRHA to validate SLS1 performance, it will be necessary to select and scale earthquake ground motions to appropriately match the target spectra.

New Zealand building standards primarily use ULS design procedures to meet the life safety performance requirement in Building Code Clause B1 which requires that buildings shall have a low probability of collapse throughout their lives. As noted in Section 2.3 above this is because the margins inherent within the ULS design procedures implemented within NZ materials standards are expected to provide sufficient confidence that acceptable collapse and fatality risks are achieved. .

This document presents an alternate design methodology whereby building performance at CALS is used to demonstrate that the Building Code life safety performance requirements have been achieved. The design methodology is based on similar procedures in ASCE 7-22 (ASCE, 2022) and related US performance-based design guidelines (PEER, 2017 and LATBSDC, 2023).

It is acknowledged that explicit evaluation of structural collapse is a difficult task requiring (a) a structural model that can directly simulate the collapse behaviour, (b) the use of numerous nonlinear response history analyses, and (c) proper treatment of many types of uncertainties. Presently, accurate implementation of this process is excessively complex and not considered practical for use in routine design. This document instead maintains the simpler approach of implicitly demonstrating adequate performance through a prescribed set of analysis rules and acceptance criteria.

When compared with NLRHA using ULS intensity ground motions the design methodology recommended in this document has the following advantages:

- *Enables explicit consideration of undesirable plastic mechanisms, including development of soft stories, that have the potential to develop in structural systems beyond ULS when Building Code B1/VM1 capacity design procedures have not been used, or are not appropriate for the structural form under consideration.*
- *Permits a direct assessment of seismic design actions of seismic resisting systems with a 'hard stop' i.e. anti-seismic devices such as dampers.*
- *Seismic design actions in rocking systems are better quantified.*
- *Enables non-ductile member actions that develop in structural systems beyond ULS to be better quantified when Building Code B1/VM1 capacity design procedures have not been used.*

2.5 Seismic Design Loads

Seismic design spectra are to be determined from TS 1170.5 or from a site-specific hazard analysis for the structural performance limit states that are to be considered.

TS 1170.5 does not provide the required annual probability of exceedance which should be considered for CALS, nor what an appropriate margin beyond ULS might be. These guidelines recommend an additional scale factor, ψ_{CALS}, be used to scale relevant ULS seismic design spectra when assessing structural performance at CALS. It is proposed that an appropriate value for ψ_{CALS} is 1.5.

Recommended structural performance factors, S_p, for NLRHA are provided in Table 2-1 below.

Table 2-1 Recommended structural performance factor for NLRHA

Performance Limit State	S_p
SLS1 and SLS2	0.70
ULS and CALS	0.85

Selection and scaling of ground motions to be used for the NLRHA should be in undertaken accordance with the methodology detailed in Section 3.

<u>*Commentary*</u>

The recommended return period scaling factors, ψ_{CALS}, equal to 1.5 is consistent with the TS1170.5 requirement that potential step-change in soil behaviour should be explicitly considered for shaking intensity up to 150% times ultimate limit state. This value has some precedent within NZ design (e.g. as inferred by the Commentary to NZS1170.5:2004 (SNZ, 2004), NZS 3101 (SNZ, 2017), NZSEE Draft Seismic Isolation Guidelines (NZSEE, 2019), public draft of NZ Seismic Assessment Guidelines – Part C1 (NZSEE, 2024)), and aligns with research completed by Zaidi et al (2024). However, it is also anticipated that buildings will likely possess some reserve capacity against collapse beyond 1.5 times the ULS design intensity.

It is acknowledged in high seismic zones the computed CALS seismic design loads maybe significantly higher than those previously considered in NZS 1170.5. For those sites, and building performance limit states, where significant soil nonlinearity is anticipated (e.g. liquefaction or cyclic softening of soil strata in the subsurface) seismic site response analysis could be undertaken to better account for the potential beneficial effects this nonlinearity may have on the resulting surface ground motions (refer further discussion in Section 3.2.3.1).

Not all members of the working group developing these guidelines were in agreement with including the structural performance factor, S_p. In line with NZS1170.5 (2004), this factor aims to allow for a number of factors that are not accounted for in the analysis. Ideally, the magnitude of this factor would be set so that the seismic performance of buildings verified using the NLRHA process lies within acceptable risk ranges. To this extent, as seismic design aims to limit both the fatality risk (done here via CALS checks) and loss of amenity (done via SLS checks) it is recognised that different S_p factors for SLS and CALS may be appropriate. It is also noted that because the NLRHA approach includes many different factors and modelling assumptions compared to the equivalent static and modal response spectrum analysis methods specified in NZS1170.5, the values of S_p to be adopted with NLRHAs are likely to differ from those specified in NZS1170.5.

At the time of drafting these guidelines there appears to be no scientific data available to inform the definition of suitable S_p factors. The recommended S_p factors are intended to align with a wider industry goal to ensure that buildings designed and verified using the NLRHA approach are not penalised relative to the more simplified and less accurate equivalent static and modal response spectrum analysis methods.

It is anticipated the Seismic Risk Working Group will consider S_p *factors and CALS seismic design loads appropriate for NLRHA as part of their Stage 2 work program and that further guidance on these issues may become available in the future.*

3 Ground Motion Selection and Scaling

3.1 Overview

The selection and scaling (or modification) of ground motions for use within NLRHA is an important aspect of performance-based seismic design, and should follow the guidance provided within this section. This section of the Guidelines is intended to improve the reliability of estimates of the seismic demands and structural response of buildings when earthquake-induced ground motions occur.

The prescriptive guidance in this section is generally based on ASCE 7-22 (ASCE, 2022) and the more holistic guidance is based on Chapter 10 of Baker et al. (2021). These reference documents are deemed the most contemporary and practical approach in regards to defining to ground motions to be used in NLRHA. Recommended modifications to the prescriptive requirements of ASCE 7-22 outlined herein are for use within the New-Zealand specific TS 1170.5 (SNZ, 2024) framework, while also overcoming the inherent limitations of NZS 1170.5:2004 (SNZ, 2004), as described in detail in Morris et al. (2019), among others.

This section of the guidelines focuses on the processes and procedures to obtain an ensemble of ground-motion time series which is consistent with the seismic hazard at the site, describing the steps necessary to select a hazard-consistent ensemble of ground motions for use in NLRHA. Hazard-consistent ground motions are defined as motions whose intensity measure (IM) values are consistent with either those derived from the current design code requirements, or site-specific probabilistic seismic hazard analysis (PSHA). Typical IMs considered are pseudo-spectral acceleration, Sa(g), peak ground velocity, PGV (cm/s), Arias intensity, AI (m/s), and significant duration, $D_{s5\text{-}75}$ or $D_{s5\text{-}95}$ (s), etc. Seismic design codes generally use intensity-based assessments (in contrast to 'risk-based assessments', see Baker et al. (2021, Section 10.6)), characterized by a response spectrum that approximates either a uniform hazard spectrum (UHS) or a conditional mean spectrum (CMS).

Figure 3-1 presents a high-level schematic overview of how ground-motion selection forms part of a seismic response assessment, and how guidance in this section of these Guidelines is structured. Specifically, in Section 3.2, two methods (A and B) are presented by which the target response spectra can be obtained. Section 3.3 provides additional sentiments regarding the spectral period ranges of interest. Section 3.4 and 3.5 then address criteria for the selection of ground motions and their modification (if necessary) to be compatible with the target spectrum.

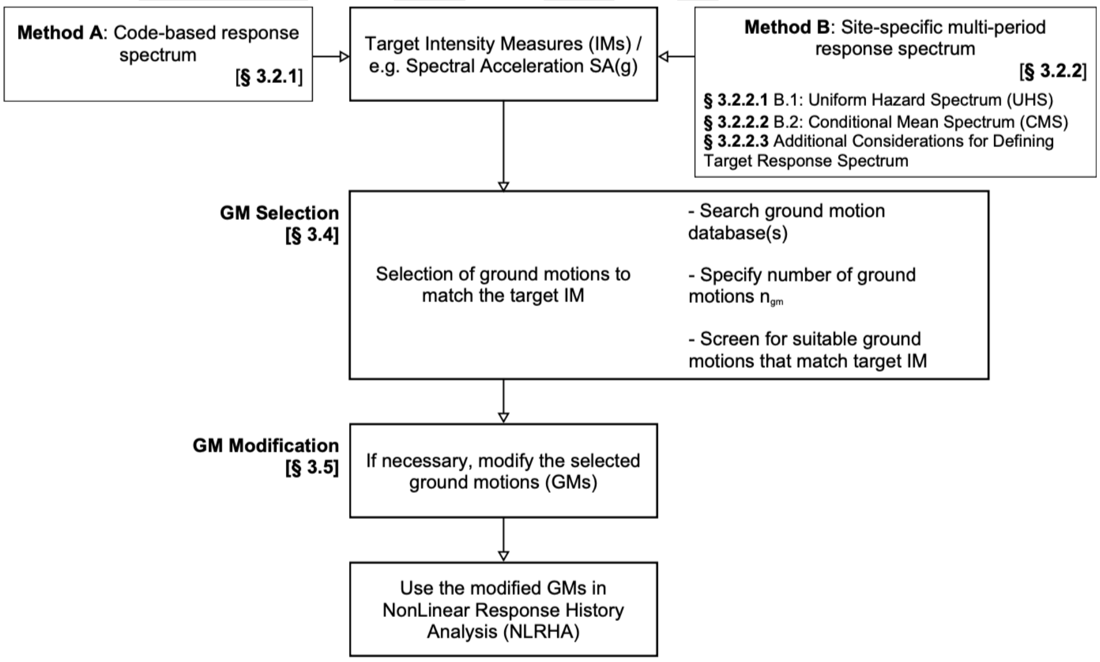

Figure 3-1: Schematic high-level overview of ground motion selection and modification process for NLRHA (modified after Baker et al. 2021)

3.2 Target Response Spectrum

A target acceleration response spectrum should be developed for each ground motion intensity level (for an associated annual probability (or rate) of exceedance, APoE) where performance limit states are being evaluated. RotD50-component ground motions (Boore, 2006) should be used to define the spectral ordinates of the target spectrum with damping taken equal to 5% critical, to remain consistent with the TS 1170.5 design response spectrum.

The target response spectrum should be developed using either Method A or Method B, which are presented below. It is permissible to use different methods for different APoEs.

Irrespective of whether Method A or B is adopted, the target spectrum maybe reduced by the S_p factor given in Section 2.5. However, where a value of $S_p < 1.0$ has been applied in the design of the structure, no further reductions in response spectra should be permitted. Kinematic soil-structure interaction (SSI) effects of base-slab averaging, embedment effects and pile kinematics are potential examples. Specifically, this restriction applies to the kinematic SSI adjustment factors presented in ASCE 7-22 Sections 19.2.3 and 19.4.

3.2.1 Method A: Code-Based Response Spectrum

A code-based response spectrum may be adopted as the target spectrum, in accordance with the design response spectrum defined in TS 1170.5. Although not explicit, it is noted that this code-based spectrum is a parametric approximation of a site's uniform hazard spectrum (UHS). In particular, the constant acceleration plateau at short vibration periods in such code-based spectra should be used with care for the selection and scaling of ground motions, as the spectral shape of observed ground motions seldom follow that generalized shape and this can result in poor least-squares fits. The influence of the short period plateau and artificial corner periods should not pose a significant constraint on the selection and scaling of ground motions.

In addition to the code-based response spectrum, relevant additional information which characterises the seismic hazard, for the purpose of ground-motion selection, can be obtained from seismic hazard disaggregation (Baker et al. 2021). Disaggregation information for the 2022 NZ National Seismic Hazard Model (NSHM) at selected geographic locations, soil conditions, vibration periods, and return periods is available at: https://nshm.gns.cri.nz/Disaggs. Because such information is not provided to suitably cover all possible combinations of these four variables, it is recommended that interpolation be undertaken when making use of the web portal. Loosely speaking, available locations within a few km will be suitable, and disaggregation is relatively insensitive to soil conditions (e.g., Vs30).

<u>*Commentary:*</u>

Directivity effects in the near-fault region are not explicitly captured within the NSHM 2022. Bradley et al. (2022) recognises there would be technical challenges in explicitly considering directivity phenomena within the ground motion models (GMMs) used for the NSHM 2022. The NSHM report by Weatherhill (2022) presents preliminary work which informed a decision where the NSHM workplan was not able to explicitly consider directivity effects. However, directivity effects are implicitly captured in the NSHM 2022 GMMs through their respective apparent aleatory standard deviations (Bradley et al., 2022). Further review of directivity effects and their implementation in TS 1170.5 is anticipated scope of Phase 2 Seismic Risk Working Group (SRWG).

Putting the NSHM 2022 aside, the capabilities and limitations of different structural analysis procedures also provides important context for these NLRHA Guidelines. Provided the ground motion selection procedure for near-fault regions is suitable, the influence of directivity phenomena on structural response is explicitly captured within the overall NLRHA procedure. In contrast, however, one of the known limitations of linear analysis methods in near-fault regions, and at high APoE, are that this is not captured by structural response predictions based on viscoelastic response spectra. This leads toward the recommendation for Phase 2 SRWG to consider *that, for certain conditions, the target response spectrum for use with linear analysis procedures may need to be different to the that adopted for use with NLRHA procedures. Precedent for this exists in ASCE 7-05 (ASCE, 2006), for example, albeit first described in the ASCE 7-10 (ASCE, 2010) Commentary C12.8.1: "Equation (12.8-7) applies to sites near major active faults (as reflected by values of S1), where pulse-type effects can increase long period demands". The current edition of the standard, ASCE 7-22, preserves this same requirement for linear analysis procedures.*

3.2.2 Method B: Site-Specific Multi-Period Response Spectrum

When a site-specific (probabilistic) seismic hazard analysis is undertaken, significantly more information is available for the development of suitable ground motions for NLRHA. Ultimately, ground-motion time series provide the causal link between the seismic hazard and the consequent dynamic structural response via NLRHA (Baker et al. Section 10.2).

In comparison to Method A, the two most notable differences that a site-specific hazard analysis, and consequent response spectra, enable are: (1) response spectra for use in ground-motion selection that are directly consistent with the underlying seismic hazard at the site (not a codified representation, including parametric shape approximations); and (2) site-specific seismic disaggregation information that is completely consistent with this response spectrum (as opposed to approximate information from a similar, but ultimately different, analysis case). When performed by appropriately qualified personnel, a site-specific hazard analysis can be considered as a more accurate and precise estimate of the seismic hazard at the site in comparison to the code-based spectrum (Method A). The site-specific seismic hazard analysis will consider, in a site-specific fashion, appropriate ground-motion phenomena related to the earthquake source (e.g., directivity), path (e.g., waveguide and other sedimentary basin impedance, reflection, and refraction phenomena), and site (e.g., specific frequencies of resonance and nonlinear surficial soil response) effects.

In this guidance, we restrict detailed discussion to the 'target' for ground motion selection in the form of a response spectrum, but it is increasingly recognised that IMs other than response spectra are important for nonlinear structural response (and especially for the majority of geotechnical structures), for which the reader is referred to Baker et al. (2021, Chapter 10), and can be considered as extensions following the same logic as the guidance herein.

<u>*Commentary:*</u>

Seismic hazard disaggregation should be presented as justification for defining the 'dominant' rupture scenarios at selected periods and their % contribution to the total hazard which will be later used as one of the selection criteria. For Method A this will likely come from the 2022 NZ NSHM web portal, whereas for Method B it should come from the site-specific seismic hazard analysis.

Method B provides a response spectrum, associated disaggregation information, and potentially other non-spectral intensity measures, that are internally consistent with the underlying seismic hazard. The site-specific seismic hazard will also include the latest scientific understanding relevant to the site of interest, including site-specific observed ground motions and/or updated fault and seismicity information in the vicinity of the site, which enable region- and site-specific modifications to the seismic hazard component models that are otherwise generally ergodic in nature (Baker et al. 2021, Chapter 8). As a result, it will generally be a more accurate and precise description as compared to the code-based seismic hazard and response spectrum obtained through Method A. The additional effort to achieve this increased accuracy and precision is likely to be warranted for higher importance structures (e.g., Importance Level 3 and 4 structures), with a desire for greater reliability in the seismic design and assessment for such structures.

This document has not included guidance on a limiting 'floor' below the TS 1170.5 design response spectrum when Method B.1 or B.2 is adopted to define the target spectrum. Development of site specific response *spectrum is deemed to be special study, and must conform to the requirements of TS 1170.5 (2024) Section 1.4. However, there limitations of how directivity effects in near-fault regions have been accounted for in TS 1170.5 (refer related discussion in Section 3.2.1 above).*

3.2.2.1 Method B.1: Uniform Hazard Spectrum (UHS)

The uniform hazard spectrum (UHS) is a spectrum with each spectral ordinate having the same annual rate (or equivalently, probability) of exceedance for the time period considered in the seismic hazard analysis. The code-based spectrum from TS 1170.5 (i.e., Method A) is a parametric approximation to the UHS from the 2022 NZ NSHM.

Method B.1 is associated with the derivation of a multi-period response spectrum from a site-specific seismic hazard analysis. To quantify the multi-period response spectrum with suitable accuracy, a minimum number of 22 vibration periods equally spaced in logarithmic space over the interval T=0-10s, should be used. Interpolation is permitted when the response period of interest is not equal to one of the discrete values that were used for the derivation of the site-specific response spectrum. While linear interpolation is permitted, it is more accurate to use logarithmic interpolation because of the way in which spectral amplitudes scale with vibration period (see Baker et al. 2021; Equation 6.26). Parametric approximations, such as truncation or smoothing, of the response spectrum are not permitted.

Seismic hazard disaggregation (see Baker et al. 2021, Section 7.2) should be presented as justification for defining the dominant fault rupture scenarios at selected Sa vibration periods and their percentage contribution to the total hazard, which will be used as one of the criteria for ground-motion selection.

Despite its common use, the UHS is widely recognised as an inappropriate target response spectrum for ground-motion selection. This is principally because the calculation of the different ordinates on the UHS (i.e., spectral values at different vibration periods) are independent from each other. It is commonly the case that the causal earthquake ruptures that dominate the seismic hazard at different vibration periods are quite different, and therefore the UHS itself does not reasonably represent a single ground motion from a scenario earthquake (which is what is assumed when individual ground motions' response spectra are selected and scaled based on the target UHS). In high seismic hazard regions and when considering infrequent return periods (e.g., 500-year return period or larger) it will often, although not always, be the case that the use of the UHS results in a conservative representation of the 'true' response spectrum for the considered return period (see Baker et al. 2021; Section 7.3).

3.2.2.2 Method B.2: Conditional Mean Spectrum (CMS)

The issues with the use of the UHS for ground-motion selection motivate the use of more hazard-consistent representations of the seismic hazard. The conditional spectrum represents the response spectrum that is probabilistically consistent with the seismic hazard, conditioned on a specific vibration period, T*, and return period of interest (Baker et al. 2021; Section 7.5). The mean value of this conditional spectrum is referred to as the conditional mean spectrum (CMS) and is a 'conventional' response spectrum that can be used in the same way for ground-motion selection that the UHS has been historically used for.

When this method is used, the following requirements (based on ASCE 7-22 Section 16.2.1.2 "Method 2") should be fulfilled, in addition to the other applicable requirements of Method B.1:

1. Two or more 'conditioning periods' (T*) should be selected, by the structural engineer, for developing conditional mean spectra. The conditioning periods should correspond closely to those periods of vibration that significantly contribute to the inelastic dynamic response of the building in two orthogonal directions. Effective 'lengthening' of the modal periods of the model should be considered in the case of peak drift responses. The importance of higher modes for peak shear deformations and absolute floor accelerations should also be considered.

2. For each selected conditioning period, a target conditional spectrum should be created that either matches or exceeds the target spectrum value, as per Section 3.2.2.1 (Method B.1), at that conditioning period. Developing the target spectrum requires the following steps:
   a. Site-specific hazard disaggregation (from the site-specific hazard analysis) should be used to identify the earthquake events that contribute most to the specified level of ground motion, at the selected period, and:
   b. The target spectrum should be developed to capture one or more spectral shapes for dominant magnitude and distance combinations revealed by the disaggregation.
3. The envelope of the target spectra should not be less than 75% of the spectral values computed using Method B.1 above, for all periods in the range specified in Section 3.3.
4. For each target response spectrum, a ground motion ensemble for response history analyses should be developed and used in accordance with Sections 3.3 through 3.5. The acceptance criteria should be evaluated independently for each of the ground motion ensembles. Said another way, the ensemble-mean responses for each conditional spectra are to be enveloped for the acceptance criteria to be evaluated.

Variations on the procedures described in this section are permitted to be used when approved by the design review. Further background on conditional spectra is given in Baker et al. (2021, Section 7.5).

<u>*Commentary:*</u>

Method B.2 Conditional Mean Spectrum presented herein is akin to ASCE 7-22 Section 16.2.1.2 "Method 2" and is elaborated upon in literature (Baker and Cornell 2006, Baker 2011, Baker et al. 2021, Section 7.5). The method addresses the typical conservatism inherent in analyses using the UHS as a target for ground motion selection and scaling. The CMS instead conditions the spectrum calculation on a Sa at a single period and then computes the mean (or distribution of) Sa values at other periods. This conditional calculation ensures the resulting spectrum is consistent with individual ground-motions that reflect the hazard at the site. The calculation is no more difficult than the calculation of a UHS and is more appropriate for use as a ground-motion selection target in risk assessment applications. The spectrum calculation requires disaggregation information, making it a site-specific calculation that cannot be generalised to other sites. It is also period-specific, in that the conditional response spectrum is based on a spectral acceleration value at a specified conditioning period. The shape of the conditional spectrum also changes as the spectral amplitude changes (even when the site and period are fixed).

Method B.2 involves a higher order of detail and requires structural periods to be used as inputs. Therefore, it may be a more time-consuming and potentially iterative process. However, the development of a site-specific UHS using Method B.1 will already involve all the prerequisite information required to develop conditional mean spectrum. Questions regarding the choice of conditioning periods often arise, given the multi-mode excitation of a building in two orthogonal directions. Studies however have shown that the results are not highly sensitive to the choice of conditioning period. This issue was examined in great detail in NIST (2011) Appendix A, prior to this method being adopted in ASCE 7. It is important to note that the chosen periods do not need to be precise and the final building design may not result in the same periods that were chosen earlier in the analysis and design process. A difference in periods on the order of 5% is considered to be reasonable without triggering the need for iteration of the CMS or post-hoc scaling adjustments. If the difference exceeds this 5%, then post-hoc adjustments such as additional scale factors may be required.

The conditional mean spectrum method has become particularly popular for evaluating the design of very long period systems at relatively large return periods. This includes high-rise buildings, or some base-isolated structures, both of which have relatively well-defined modal characteristics (and typically aided by relatively stiff concrete diaphragms). Conversely, the conditional mean spectrum may be less well suited for some cases and *involves some consideration prior to triggering the additional cost, complexity and analysis effort. An example of these cases and considerations include:*

- *For return periods less than 500 years, the dominant fault rupture scenarios may not be so clearly defined as per the requirements of Section 3.2.2.2.*
- *Where two orthogonal directions have significantly different periods. In this case, using the UHS may be more practical. Although NIST (2011) states that changing the conditioning period T* does not significantly affect results, much of NIST (2011) recommendations are based on detailed risk-based assessments using 40 ground motions per hazard level. Such a high volume of analysis is impractical in typical design scenarios and therefore this recommendation may not extrapolate to design office practice.*
- *Less clearly defined modal and inelastic properties of the system. This may apply to structures with relatively flexible diaphragms, such as timber diaphragms.*
- *Podium structures – where the modal periods and significance of mass of the podium may vary from the modal periods of the "tower" above, this may warrant an additional conditional spectra and ensemble of ground motions.*
- *Low rise buildings, due to the relatively short period range, there will be a trade-off in the value gained from this approach and the added computational expense when using Method B.1 and the minimum number of ground motions.*
- *During preliminary design/analysis phases, where the modal characteristics vary depending on the design options and model inputs are not fully vetted (such as stiffness assumptions, soil/foundation flexibility, etc).*

3.2.3 Additional Considerations for Defining Target Response Spectrum

3.2.3.1 Seismic Site Response Analysis

Empirical ground-motion models conventionally used in probabilistic seismic hazard analysis represent site conditions through simplified parameters, such as the 30-m time-averaged shear-wave velocity, Vs30, along with the 1.0 and 2.5 km/s shear-wave velocity depths, Z1.0 and Z2.5, respectively. Where significant site investigation information is available, and there is the expectation that the site may behave differently from these generic (ergodic) empirical site response models, then a seismic site response analysis may be undertaken.

Although no prescriptive guidance presented in this section, a general framework for this approach is given in Chapter 21 of ASCE 7-22. However, for application within the TS 1170.5 framework, there are two supplementary requirements and considerations:

1. The "bedrock" or "reference" response spectrum should be defined according to Method B.1 presented in Section 3.2.2.1. This requirement is on the basis that seismic site response analysis is only to be used as an extension of a site-specific hazard analysis, and should not be used in a code-based response spectrum context for obtaining ground motions at the foundation level of the structure.
2. The vibration period range considered in the selection and modification of "base" (input) ground-motions will vary depending on the purpose of the site response analysis. There are two general cases for which seismic site response analysis is performed, as listed below:
    a. To evaluate and understand the effects of subsurface stratigraphy on ground motion amplification at the surface, and the resulting surface ground motions will be subsequently used for the response history analysis of a structural system at the building surface. In this case, the recommended vibration period ranges for the input ground motion selection

should be consistent with the surface structural system, as per Section 3.3 of these Guidelines.

b. To evaluate and understand the soil deformation due to potential geological hazards (e.g., liquefaction, cyclic softening) and failure of soil strata in the subsurface. In this case, the vibration period, or period range, of interest should cover those periods which dominate the shear strain response of the susceptible strata. Note that this may or may not be associated with the first mode period of the soil deposit, depending on the overall depth of the site response analysis model considered.

In cases for which both situations (1) and (2) are of interest, then these should be considered as different analysis cases that both need to be considered, which can either be achieved through increasing the total vibration period range adopted (for Method B.1), or by effectively considering two (or more) different conditioning periods for a conditional spectrum approach (Method B.2).

<u>*Commentary:*</u>

While there are numerous benefits to such site-specific site response approaches (see Baker et al. 2021; Section 8.6), such methods involve the use of wave propagation methods and uncertainty analysis, which are likely only practical to undertake for critical infrastructure projects with experienced personnel, and which will undergo suitable participatory peer review from qualified experts either nationally or internationally. Hence, there is no prescriptive guidance presented in this section.

The Method A code-based response spectrum is not suitable for defining the "bedrock" response spectrum, due to the truncated shape for short period spectral ordinates introducing an unconservative bias in the output ground surface motions. This requirement also ensures a consistent level of detail is maintained in the overall seismic hazard analysis and response spectrum development.

Seismic site response analysis is deemed to be special study and must conform to the requirements of TS 1170.5 (2024) Section 1.4.

3.2.3.2 Vertical Seismic Effects

Most structures will not explicitly require consideration of vertical ground motions in their seismic design, as noted in Section 3.6.2, and/or will not have structural components whose capacity or deformation limits are significantly influenced by vertical excitation. As a result, TS 1170.5, like many international codes and standards, takes a simplistic prescription to the specification of vertical response spectra.

The principal challenge with prescriptive (e.g., TS 1170.5) vertical spectra for ground-motion selection is that the seismic ruptures that dominate the hazard underpinning the horizontal spectrum are often not the same as those for the vertical spectrum (e.g., vertical spectra tend to be dominated by very near source earthquakes because of the manner in which vertical ground motions attenuate). This presents the same problem as the use of the UHS for ground-motion selection. As a result, two paths are generally taken for the consideration of vertical ground motions in ground-motion selection:

1. Ground motions are selected for the horizontal (i.e., RotD50) component resultant, based on the target spectrum and associated disaggregation information, and the vertical ground motion that is associated with the selected horizontal components is directly used with the same amplitude scale factors (if any are applied); or

2. A conditional vertical ground motion response spectrum (e.g., Gulerce and Abrahamson, 2011) is obtained that is compatible with the conditional (horizontal) response spectrum. In this approach, it is plausible to modify the vertical ground motion components separately to the horizontal components.

The use of approach 2. above is technically complex, and it is expected that this would be undertaken in accordance with the same general sentiments above regarding Method B.2., including participatory peer review by suitably qualified experts.

3.3 Period Range of Interest

A period range, [T_{lower},T_{upper}] should be determined, corresponding to the vibration periods that contribute significantly to the building's lateral dynamic response.

When defining the upper bound period values, T_{upper}, the designer should allow for the period lengthening that is anticipated for each level of seismic demand. Table 3-1 presents recommended default values that do not require further justification. Alternatively, alternative values based on evidence from analysis or established ductility relationships may be used. In the latter case where default values of Table 3-1 are not adopted, then the lower limits of Table 3-1 will apply.

Table 3-1 Upper bound period values

Seismic Hazard Level	Default values of T_{upper}[1]	Lower limits on T_{upper}[1,2]
SLS2 (and lower)	$1.2T_{max}$	$1.0T_{max}$
ULS	$1.7T_{max}$	$1.3T_{max}$
CALS	$2.0T_{max}$	$1.5T_{max}$

Notes:

1. Where T_{max} = the maximum fundamental period (including both translational and torsional modes)
2. Applicable limits when structural engineer does not adopt default values

The lower bound period, T_{lower}, should generally be defined as the lesser of the period of 90% of superstructure mass participation, $T_{90\%}$ and 0.2 times T_{min}, the smallest first-mode period for the two principal horizontal directions of response, specifically: $T_{lower} = \min[T_{90\%}, 0.2T_{min}]$. However, where structures have a first mode mass participation factor greater than 0.75, the second criteria may be relaxed to $0.4T_{min}$.

Finally, where Method A has been used to define the target spectrum, caution is advised if T_{lower} is less than the 0.1 second "first corner period" (start of short period plateau) as it may produce unfavourable ground motion selection and/or scaling. Where it can be justified by the outcome of the ground motion scaling results, it may be permissible to set 0.1 seconds as a limit on T_{lower}.

<u>Commentary:</u>

The period range for scaling of ground motions is selected such that the ground motions represent the specified hazard level at the structure's fundamental response periods, periods somewhat longer than this to account for period lengthening effects associated with nonlinear response and shorter periods associated with a higher mode response. Compared to ASCE 7-22 16.2.3.1, the proposed period range is considered more suitable within the NZS context. This is because the ASCE 7-22 Chapter 16 approach is specific to the MCE_R-hazard level that is defined in the US-specific code framework.

The period range of interest described in this section is generalised for conventional lateral load resisting systems. There may be some situations where review and justification of the period range of interest may be required. This commentary intends to capture some examples and types of considerations that should apply. An immediate example of a non-conventional system is base-isolated building structures, where more specific *recommendations are provided elsewhere (such as the NZSEE 2019 Draft Seismic Isolation Guidelines (NZSEE, 2019), and ASCE 7-22 Chapter 17, for example).*

There are certain cases where the prescriptive value of 0.2T_{min} is not a good indicator for lower bound period, and 0.4T_{min} is presented herein as an exception which captures the first mode dominated structures where higher mode effects are less significant. It is worth noting that $T_{90\%}$ is sometimes not practically achievable for all cases – wall buildings with flexible diaphragms are a particular example where the superstructure response includes many local modes. The period range of interest may require some engineering judgement.

In many cases, the substructure is included in the structural model, and this inclusion substantially affects the proportions of seismic mass of the system model. Unless the substructure and foundation system are explicitly designed using the results of the response history analyses, the 90% modal mass requirement pertains only to the superstructure behaviour; therefore, the period range of interest for ground motion scaling does not need to include the very short periods associated with the subgrade behaviour.

In cases where there are two or more "towers" above a common podium, the period range of interest may need to account for each of the modal responses of the towers and an enveloping assumption may be required. This becomes important if the NLRHA will be used as the basis for evaluating the design of the podium substructure (including foundations). Whenever a project involves multiple towers, it is important early in a project that the designer clearly identifies for the peer reviewer(s) the substructure design assumptions, scope of the structural model, and the specific decisions made for the ground motion selection, scaling, and application to the structural model.

3.4 Ground Motion Selection

As illustrated in Figure 3-1, ground-motion selection requires (1) target IMs (spectral accelerations); (2) database(s) of ground motions to select from; (3) selection criteria; and (4) evaluation criteria for the selected ensemble of ground motions. Further conceptual background is given in Baker et al. (2021, Section 10.4).

Ground motions are then selected from ground-motion database(s) such that their Sa values have a distribution that is statistically consistent with the target spectrum (i.e., this set of ground motions is an 'ensemble'). These ground motions, after scaling or modification, can then be used to compute the seismic response of the structure of interest.

Associated with the definition of the target is the specification of how many ground motions should be identified from the selection process. An ensemble of not less than 11 ground motions (horizontal record pairs) should be selected for each target spectrum, with the exception of SLS1 seismic demand level which a minimum of 7 record pairs may be used. Ground motions should consist of pairs of orthogonal horizontal ground-motion components (where vertical earthquake effects are required, a vertical ground motion component should also be selected).

A database(s) of recorded or simulated candidate ground motions must be available. Databases may contain a very large number of motions, and therefore some degree of preselection, or screening, will typically be applied by imposing bounds upon the ground-motion causal parameters. With the IM target and prospective ground motion database(s) defined, particular algorithms will identify motions that match the target in some manner, and the quality of this match is assessed using one or more evaluation criteria. Readers interested in specific algorithmic details are referred to Baker et al. (2021; Section 10.4).

Ground motions should be selected from events within the same general tectonic regime, have generally consistent magnitudes and source-to-site distances as those dominating the target spectrum (as inferred from seismic hazard disaggregation), and should have a spectral shape similar to the target spectrum.

For near-fault sites, as defined in Section 3.6.1, and other sites where ground shaking can exhibit directionality and impulsive characteristics, the proportion of ground motions with near-fault and rupture directivity effects should reflect the probability that shaking, for the associated hazard level, will exhibit these effects.

Where available ground-motion databases provide an insufficient number of recorded ground motions, based on initial selection criteria, it is be permitted to supplement the available records with simulated (synthetic) ground motions and/or heuristically relax some selection criteria (e.g., Baker et al. 2021; Section 10.4.2).

Similar to the consideration of recorded ground motions, the potential use of simulated ground motions should be consistent with the magnitudes, source characteristics, fault distances, and site conditions controlling the target spectrum and the underlying methods and models used in the development of the simulations should have appropriately considered validity in the context of this use of the simulated ground motions.

<u>*Commentary:*</u>

An increased number of ground motion time series enables a higher degree of confidence in the statistical values (e.g., mean) computed from the limited number of response history analyses performed. Historically values as low as 3 or 7 ground motions have been considered. Studies (e.g., Bradley 2011, 2014) have shown that such limited numbers of analyses leads to significant practical problems in the estimation of mean seismic demands with confidence. As a result, more recent guidance (e.g., ASCE 7-16, 7-22) has established 11 ground motion time series (i.e., 11 sets of bi-directional or tri-directional records obtained from a seismic instrument) as the standard. This increase from 7 to 11 records leads to an approximate 20% reduction in the standard error estimate of the sample mean, and there is certainly benefit of considering more than the minimum of 11 ground motions (Baker et al. 2021, Chapter 8), and the industry-accepted minimum number is likely to continue to increase in the future. It is also worth noting that obtaining a large number of suitable historically-recorded ground motion time series may become challenging for seismic scenarios that are poorly represented in recorded databases, hence while there is a general sentiment that "more records is better" from a statistical error perspective, this record quality consideration will impose a constraint on how many records are practically suitable to adopt.

As discussed in Section 10.4.1 of Baker et al. (2021), for a given number of ground motion time series, there is appreciably more uncertainty in estimating the standard deviation of the distribution of seismic demand than the mean. As a result, with the use of 11 ground motions, it is the mean seismic demand that should be the primary metric of focus (albeit with additional consideration for 'unacceptable" levels of response from individual records).Near Fault (NF) conditions require special consideration during the ground motion selection process, i.e ASCE 7-22 Section 11.4.1 defines near-fault conditions as fault distances < 15 km for M_W > 7 earthquakes, and fault distances < 10 km for M_W > 6 earthquakes. Ground motions for sites subject to forward rupture directivity effects have an increased likelihood of having pulse-like characteristics in their velocity-time series. When disaggregation results indicate controlling faults meet these criteria, the site should *be considered as a near-fault site. Important considerations include:*

- *Number of pulse-like records. Although NZS1170.5:2004 prescribed one-third of records (i.e. 1 in 3) should have directivity effects, this requirement was set without consideration of site- or structure-specific information. These guidelines do not prescribe a specific proportion of pulse-like records, as this should be evaluated on a project specific basis.*
- *Period of the pulse & checks relative to the structural periods of interest.*
- *Pulse should be maintained (not diminished) and reviewed following ground motion modification.*

Databases / Ground Motion Portals (basic for initial reference only as there are and will be few other sources):

- *NGA-West2 Database (Ancheta et al. 2014)*
- *NGA-Subduction Database (Mazzoni, 2022)*
- *COSMOS*

- *GeoNet Strong Motion Database*
- *Center for Engineering Strong Motion Data (CESMD)*
- *Engineering Strong-Motion Database (ESM)*

Recorded ground-motion databases can generally be used as a source of prospective ground motions. However, there is a paucity of recorded ground motions for causal parameter combinations that are often of interest in seismic design and assessment, and thus desired for ground-motion time series. Recorded ground motions are very unevenly distributed in magnitude-distance space, when examining NGA-West2 for example, with many magnitude-distance combinations having few observations. Thus, recorded ground-motion databases do not sufficiently sample the multidimensional parameter space to make ground-motion selection straightforward. Although ground-motion instrumentation networks continue to increase in density, this under sampling problem is likely to persist into the future (Baker et al., 2021).

As mentioned, ground-motion selection should be performed in a hazard-consistent manner, and this is to be reflected through both the disaggregation of the seismic hazard (or at least on the basis of some information around the tectonic regime/zones likely to affect the site) and the target IM distribution (Sa). The disaggregation results identify the implicit causal parameters (rupture magnitude, source-to-site distance, etc.) that result in the ground-motion hazard but are not measures of the ground motions themselves. The IM targets provide an explicit description of the characteristics of the ground motions that result in the ground-motion hazard via the vector of IMs (Baker et al., 2021).

The distinction between implicit causal parameters and explicit IMs is a fundamental concept in ground-motion selection. Historically, the emphasis was placed on implicit causal parameters in ground-motion selection. However, it is now widely appreciated that a focus on explicit IMs is more important (noting that implicit causal parameters from disaggregation affect the development of the IM targets).

3.5 Ground Motion Modification

Observed or simulated ground motions, selected through the processes outlined in Section 3.4, should either be amplitude-scaled following Section 3.5.1 or, by alternative modification methods following Section 3.5.2.

3.5.1 Amplitude Scaling

For each horizontal ground motion pair, a RotD50 response spectrum should be constructed from the two horizontal ground motion components. Each ground motion should be scaled, with an identical scale factor applied to both horizontal components, such that the average of the RotD50 response spectra from all ground motions generally matches or exceeds the target response spectrum over the period range defined in Section 3.3. The average of the RotD50 response spectrum from all the ground motions should not fall below 90% of the target response spectrum for any period within the same period range.

<u>Commentary:</u>

Amplitude scaling is the most common and widely accepted approach to modifying "seed" ground motion time histories for NLRHA. Linear scale factors can be determined using mathematical algorithm's which minimize the mean-squared error. Examples of this calculation process are given in Baker et al. (2021, Section 7.5).

As ground-motion selections are made conditional on 'comparing' IMs, recorded motions are typically "scaled" by multiplying their acceleration amplitudes, such that they are consistent with this conditioning. Similar to causal parameter bounds, the literature also contains many suggestions for limits on ground-motion scaling. In most cases, these recommendations are also based upon intuition rather than any quantitative analysis (e.g., see the discussions in Watson-Lamprey and Abrahamson, 2006; Luco and Bazzurro, 2007). Scaling a ground motion can produce combinations of IM values that are not naturally seen in a ground motion with the target

IM. However, if only ground motions with reasonable IM combinations are selected, or if the response metric of interest is not sensitive to the IMs that have been distorted, then scaling is unlikely to introduce any significant biases (Luco and Bazzurro, 2007; Bradley, 2010b).

The only notable difference between these guidelines and ASCE 7-22 Section 16.2.3.2 is that spectral ordinates are not defined as RotD100 maximum direction spectrum. As RotD50 spectral ordinates (Boore, 2006) define the TS 1170.5 target response spectrum (as noted earlier) then for internal consistency in the procedure the amplitude scaling must also be performed in the RotD50 domain.

The justification for allowing amplitude scaling to 90% of the target spectrum is intended to implicitly offset an inherent conservatism that is typically introduced by amplitude scaling ground motions across the entire period range of interest. The specification of 90% of the Target being acceptable for ASCE 7-22 16.2.3.2 is generally deemed appropriate for situations when the scaled mean response spectrum is governed by the "extremities" of the period range of interest (ie towards upper bound or lower bound period). The relaxation to 90% of the target is less appropriate broadly over the fundamental periods. The former case is typically the common observation of amplitude scaling; however, the latter case is possible and should be recognized during the review of the scaled response spectrum.

When Method A is used as the target spectrum, there may be an undesirable influence of the short period plateau with respect to GM scaling. The plateau can cause artificial problems with the scaling when T_{lower} is less than the corner period. Additional caution and review is recommended in such cases.

When amplitude scaling is adopted for moderate-to-longer period systems, the most successfully "refined" and accurate outcomes that can be practically achieved are often due to the use of conditional mean target spectrum (i.e., Method B.2) as the target response spectrum, and/or increasing the number of ground motions beyond the minimum of 11, where 13 to 15 ground motions offers a potentially reasonable trade-off between computational demands vs refinement in the scaled ensemble of ground motions.

A generalized form of the amplitude scaling procedure in NZS 1170.5:2004 sections 5.5.2 and C5.5.2 have been included in the commentary of these guidelines as an acceptable "deemed to comply" scaling procedure. These expressions are understood to be commonly used by practicing engineers in New Zealand. Firstly, individual record scale factors, k_1, are calculated for each record are calculated first, before the "family" scale factor (k_2) is found for the entire suite. More detail on the two-step calculation procedure is shown below:

1. *Determine the individual record scale factors, k_1, based on minimizing the statistical goodness-of-fit parameter D1. The D1 parameter is shown in Equation C3-1:*

$$D_1 = \sqrt{\frac{1}{(T_{UB} - T_{LB})} \int_{T_{LB}}^{T_{UB}} \left(\log \frac{k_1 SA_{\text{component}}}{SA_{\text{target}}} \right)^2 dT} \qquad \textit{C3-1}$$

Over the prescribed period range, the intent is to achieve a statistical "best fit" by minimizing the root-mean-squared difference between the scaled RotD50 response spectrum and target RotD50 response spectrum.

2. *Determine the "family scale factor", k_2, to be applied to the entire suite of records. This factor ensures that the suite mean spectrum (scaled already by k1 for individual records) is scaled by k_2 such that exceeds the target response spectrum for every period over the period range of interest.*

$$k_2 = \left(\frac{SA_{target}}{\text{mean}(SA_n)} \right) \qquad \textit{C3-2}$$

It should be noted that, as amplitude scaling is performed on a mean-spectral acceleration basis, it is advisable to review the impacts the scale factors have for other intensity measures. When a scale factor (k) is applied to a ground-motion time series, it does not result in a proportional change to all IMs. Response spectral ordinates are multiplied by the scaling factor (k), but Arias intensity is affected by the square of scaling factor (k^2), while the significant duration is entirely unaffected by scaling.

It is also recommended to review whether the scaled ensemble-maximum response is within reasonable bounds. Typically, the ratio of the scaled ensemble-maximum response spectra is approximately 1.3 to 1.5 times the ensemble mean response spectra, and anything greater than this may warrant iterations in the ground motion selection and/or scaling procedure. For as-recorded ground motions which have large velocity pulses, the use of scale factors > 1, may result in extremely severe ground motions. This reflects a typical challenge of near-fault regions of extreme seismicity, such as in Wellington, where ensemble-mean acceptance criteria is often achieved more easily compared to ensuring acceptable responses for extreme individual records.

There are publicly and commercially available tools which can assist with performing amplitude scaling, such as the PEER NGA West-2 Database (Ancheta et al., 2014), QuakeManager (by Earthquake Solutions), SeismoSelect (by SEiSMOSOFT), etc.

3.5.2 Alternative Ground Motion Modification Methods

In addition to traditional amplitude-scaling of ground motions, there are various other methods of developing or modifying ground motions. The use of alternative methods is permitted provided they are sufficiently justified, documented, and based on recent scientific studies, site-specific considerations and are peer-reviewed.

One example of alternative modification methods is "spectral-matching", which selectively modifies parts of a ground motion rather than amplitude scaling the entire signal by a constant. The Mean Spectrum Matching methodology (Mazzoni et al. 2012) is permitted, without any subsequent limitations and penalties. Tight Spectral Matching is not permitted by these Guidelines.

<u>Commentary:</u>

Tight Spectral matching is not permitted by these guidelines due to concerns associated with diminishing record-to-record variability when undertaken in either the H1/H2 component domain (Abrahamson, 1992), or the RotD50 domain. It should be noted that, while tight spectral matching in the RotD100 domain can produce a higher degree of record-to-record variability at the component level (and maintain dispersion in engineering demand parameters from analysis), this variation of the tight matching is not expected to be applicable for NZ due to the RotD50 target spectrum. Within the ASCE 7-22 framework, tight spectral matching also triggers other unwanted complexities later in the building performance evaluation criteria and post-processing (which have been deliberately avoided in these guidelines).

3.6 Applying Ground Motions to the Structural Model

Typically, only horizontal ground motions will need to be considered according to Section 3.6.1 for the majority of NLRHA applications. However, Section 3.6.2 highlights some cases when vertical ground motion components may be required for a separate analysis.

3.6.1 Horizontal Ground Motion Components

Each pair of horizontal ground motions need only be applied once to the computational model, and shall met the other requirements of this section listed below.

For near-fault sites, defined as having dominant earthquake rupture distance D ≤ 5km, each pair of horizontal ground motion components representative of a nearby fault source should be identified as "near-fault" records, and rotated to the fault-normal and fault-parallel directions of the causative fault and applied to the building in such orientation.

For all other selected ground motions at near-fault sites, and for all ground motions at other sites, these should be identified as "far-field" records in which, probabilistically speaking, ground motions are historically shown to be non-polarised (e.g., Shahi and Baker, 2014). Each pair of far-field horizontal ground motion components should be applied to the building at orthogonal orientations, such that the average (or mean) of the component response spectrum for the records applied in each direction is within ±10% of the mean of the component response spectra of all records applied for the period range specified in Section 3.3.

Note that all subduction source events which are representative of the Hikurangi subduction fault may be classified as "far-field", including the moderate-to-near source subduction condition that exists in Wellington. Essentially, the notion of 'near-source' is restricted to active shallow crustal earthquakes for which such phenomena have been well studied.

<u>Commentary</u>

Each pair of horizontal ground motions need only be applied once to the computational model, i.e., specific bi-directional pairs of ground motions do not need to be applied at multiple orientations. The rationale for this is associated with the relative variability of seismic response that results from (1) different ground motion time series vs. (2) the same time series applied at different orientation angles. Because ground motions are generally unpolarised at far-field sites, then the variation in seismic demand due to different orientation angles is small relative to considering different ground motions. Said another way, it would be significantly better to consider 22 different ground motion pairs, applied to the structure at random orientation angles, than to consider 11 ground motion pairs, and apply each at two different orientation angles.

The classification of Near-Fault and Far-Field sites stated in this section relates to directionality of ground motions, and is a distinction made for the purposes of applying fault-normal and fault-parallel components within the NLRHA procedure. The guidelines differ slightly from ASCE 7-22 as a shortened distance criteria is used to make the classification of Near-Fault vs. Far-Field.

In these guidelines, the notation 'D' is chosen for consistency with notation in TS 1170.5, which represents the closest distance to the fault. By definition 'D' is the same R_{rup}, (the latter being more common notation in technical literature).

The strict definition of Near-Fault sites in ASCE 7-22 is the distance criteria is up to 15km, however there are concerns regarding the lack of evidence to support the nominated distance. Lack of consensus among subject matter experts is apparent on this issue, as there is limited evidence that ground motions are strongly polarized for distances greater than 5km. It was decided to use a reduced distance of 5km for these guidelines. This modification from ASCE 7-22 is expected to reduce the likelihood of introducing an unintentional bias in the application of motions for "far field" sites, where one axis of the structural model experiences significantly greater demand than the orthogonal axis.

Since the orientation of ground motions in the near-fault environment is highly uncertain, it is inadvisable for a building designer to take advantage of a directional reduction that may or may not develop. This uncertainty can be investigated and managed in various ways. For example, the geotechnical engineer may need to report a closer examination of ground motion parameters, such as orbital plots of spectral displacement demands at vibration periods of significantly.

Checking the ±10% directional bias limit over a full ensemble of 11 records is considered practically straightforward, as is discussed in Morris et al. (2019) alongside an example. Soon after this requirement was first introduced in ASCE 7-16, many US-based practitioners cited difficulty that the ±10% threshold was not easily attainable, however this was the result of misinterpretation of these requirements and lack earlier published examples prior to Morris et al. (2019).

3.6.2 Vertical Ground Motion Components

Unless otherwise allowed for in the design process, an explicit vertical response history analysis may be necessary when specific structural components are expected to be sensitive to vertical ground motion. Some common cases include:

- Long horizontal spans and/or large cantilevers supporting gravity load;
- Horizontal prestressed components;
- Major discontinuous vertical elements of the gravity force-resisting system.

Commentary:

In the common cases described in this section, there is industry precedent that for the purposes of the 'vertical response history analysis', the vertical component of ground motion need not be combined with the effects of the horizontal response. In such cases, the vertical ground motion components are typically independently selected and scaled to a vertical target response spectrum over a vertical period range of interest. It is therefore possible to decouple the vertical and lateral response analyses, using separate analysis models and input assumptions for each. This is further explained in the context of the following considerations:

- *Many cases which are sensitive to vertical accelerations form part of the gravity force-resisting system of buildings, not the lateral-force resisting system.*
- *To properly capture vertical response to ground shaking, it is necessary to accurately model the stiffness and distribution of mass in the vertical load system, including the axial stiffness of columns and horizontal framing of floors and/or roof systems. The outcome is often increased model size and complexity. This additional model complexity may not be appropriate for the primary lateral analysis model*

In the above situation, it is more likely that the vertical response spectra needs to developed according to Section 3.2.3.2. approach #2.

For other cases where the effects of a coupled horizontal and vertical response could potentially impact the design of the lateral load resisting system, then vertical ground motions will likely be applied to the structural model concurrently with horizontal components. This does not require a separation vertical response spectrum, and is more consistent with Section 3.2.3.2. approach #1.

3.7 Documentation

The procedure for the selection and modification of ground motions should be individually presented or included in the final Design Features Report including the following project-specific information:

- The seismic hazard analysis that the ground-motion selection is based upon (whether a site-specific PSHA or a code-based prescriptive hazard)
- Target(s) (Response spectrum, scenario spectrum, design spectrum)
- Structural period(s) of interest
- Geotechnical parameters including soil characteristics used for the selection of acceleration histories
- Selection criteria including hazard disaggregation to determine 'controlling' scenarios at selected periods and their % contributing to the overall hazard
- Properties of the selected ground motions in terms of IMs and causal parameters, and their match to the target IM distributions
- Plot selected vs target(s)
- Modified records vs target(s)

- Modified GMs with their various IMs

Commentary:

Because ground-motion selection can consider a wide variety of algorithmic variations and parameter values, and because the numerical outputs can be sensitive to those inputs, providing a careful explanation of chosen inputs is crucial. Without clear documentation, it is challenging to understand and interpret results from a study, reproduce a calculation, or critically examine the assumptions that have been made.

4 Analysis and Performance Evaluation

4.1 Overview

This chapter provides recommendations for developing analysis models for use in the performance evaluation of structures via NLRHA. The procedures contained within this section are applicable for use to demonstrate acceptable performance with respect to serviceability (SLS1/SLS2) and collapse avoidance (CALS) limit states.

The load combination for determination of relevant design action effect used to evaluate seismic performance is defined by AS/NZS 1170.0:2002 (SANZ, 2022) Section 4.2.2(f) as:

$$E_d = G + \psi_E Q + E_{LS} \quad \text{4-1}$$

where $\mathrm{E_d}$ is the design action effect, G is the permanent action (self-weight or 'dead' action), and ψ_E the earthquake combination factor (as defined in Table 4.1 of AS/NZS 1170.0:2002) to be considered with imposed action, Q, and E_{LS} is the earthquake action for the limit state under consideration (i.e. SLS or CALS as applicable).

The non-seismic portion of the loading, $E_{d,ns}$, includes the permanent and imposed actions and is defined below:

$$E_{d,ns} = G + \psi_E Q \quad \text{4-2}$$

<u>Commentary:</u>

E_d and $E_{d,ns}$ should also include the action effects arising from other long-term loading conditions (e.g., lateral earth pressures, hydrostatic, etc.).

4.2 Modelling and Analysis

This chapter provides recommendations and guidance for developing a numerical model of the structure for use with nonlinear response history analysis.

4.2.1 Modelling

A three-dimensional mathematical model of the structure should be created for the purposes of determining member forces and structural displacements. The model should include the strength and stiffness of elements that are significant in the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure. These items are considered to form the "Primary" structure. The strength and stiffness of elements that do not significantly contribute to the strength and stiffness of the structure may be classified as "Secondary" and omitted from the analysis model and assessed separately considering the resulting deformation demands.

In addition, the model should ensure that:

1. The stiffness properties of concrete and masonry elements consider the effects of cracked sections. Where shown by analysis that an element is unlikely to crack for the considered seismic demand, gross section properties may be assumed.
2. The contribution of joint flexibility to displacement and drift is included.

For structures that have subterranean levels, the structural model should extend to the foundation level and ground motions should be input at the foundation level.

All elements that significantly affect seismic response when subjected to the specified ground motions should be included. Modelling of element nonlinear hysteretic behaviour should be consistent with the mechanism under consideration. The strength and stiffness ascribed to nonlinear elements should be representative of the expected value. Degradation in element strength or stiffness should be included in the hysteretic models unless it can be demonstrated that the deformation demands are not sufficient to produce these effects.

Analysis models should be capable of representing the flexibility of floor diaphragms where this is significant to the structure's response. Diaphragms at horizontal and vertical discontinuities in lateral resistance should be explicitly modelled in a manner that permits capturing the force transfers and resulting deformations.

Commentary:

As permitted by TS1170.5 Clause C2.6, structural elements are required to be designated as either "Primary" or "Secondary". Currently TS 1170.5 does not provide guidance on how this distinction should be made and in leu of this reference should be made to the provisions included within ASCE41-23 Section 7.2.4.3.

Expected material properties should be considered in the analysis model which represent the median (50th percentile), attempting to characterise the expected performance as closely as possible. This is consistent with ASCE 7-22 (ASCE, 2022) when considering MCE_R, and TS 1170.5 (SNZ, 2024) when considering SLS demand. In typical levels in building structures, it will generally be sufficient to consider diaphragms as rigid in plane. At locations where significant transfer is likely (e.g., podium and other setback levels), or where vertical offsets are present, a flexible diaphragm model should be adopted.

Diaphragm demands associated with transfer of seismic shears between lateral force resisting elements are highly sensitive to the stiffness assumptions in the model. It is recommended that analyses consider suitable bounding values for the stiffness of the diaphragm and the lateral force resisting structure at the level under consideration. Guidance on this is provided in PEER/ATC 72-1 (ATC, 2010).

Where the ground motion suite includes the vertical component, care should be taken to ensure that the analysis model is conditioned so that the vertical response of the structure is adequately represented.

4.2.2 Seismic Weight and Mass

The modelling of, and demands on, elements in the analysis model should be determined considering earthquake effects acting in combination with the anticipated in-service loads as identified in Section 4.1.

The seismic weight to be considered at each level, W_i, for determination of dynamic characteristics and P-Delta effects, should be determined in accordance with the requirements of Section 4.2 of TS 1170.5.

4.2.3 P-Delta Effects

The analysis should include P-Delta effects which consider the spatial distribution of gravity loads. Including the spatial distribution of gravity loads on plan is generally necessary to ensure that the P-Delta contribution to torsional effects, often referred to as P-Theta effects, is captured (Haselton, 2017).

Commentary:

P-Delta effects should be included regardless of the value of the storey stability coefficient determined via TS 1170.5. Spatial distribution of gravity loads on plan may be achieved either as distributed loads to beam and wall elements, and/or as point loads to columns. For most building type structures, small displacement theory may be assumed with the geometric stiffness established with consideration of only the gravity loading. Where it is necessary to also consider local member instabilities, the geometric stiffness should be updated at each load *step to consider the updated axial demand. Where the structure deformations are significant (e.g., cable structures) the geometric stiffness should be updated at each load step to also consider the new deformed shape.*

4.2.4 Torsion

Inherent eccentricity resulting from any offset in the centres of mass and stiffness at each level should be accounted for in the analysis.

In addition, accidental eccentricity should be considered consisting of an assumed displacement of the centre of mass each way from its actual location by a distance equal to 5% of the diaphragm dimension parallel to the direction of mass shift. The required displacement of the centre of mass need not be applied in both orthogonal directions at the same time.

Commentary:

When considering accidental torsion for linear, force-based procedures, TS 1170.5 (and its predecessors) requires consideration of an accidental mass eccentricity equal to 10% of the diaphragm dimension. Of this, 4-5% is said to be attributable to the effects of asymmetric failure (Elms, 1976). When undertaking NLRHA, any changes in the plan location of the centre of rigidity resulting from asymmetric yielding can be explicitly accounted for. For this reason, this document considers that the contribution of asymmetric failure to the accidental torsion requirements are not warranted. The adopted value of 5% has been selected accepting the approximate nature of the quoted values, and for consistency with the Draft Guideline for the Design of Seismic Isolation Systems for Buildings (NZSEE 2019).

4.2.5 Damping

Hysteretic energy dissipation of structural members should be modelled directly. Additional inherent damping, not associated with inelastic behaviour of elements, should be modelled appropriate to the structure type.

The target elastic equivalent viscous damping ratio, ξ, should be calculated using Equation 4-3:

$$\xi = \frac{0.2}{\sqrt{h}} \leq 0.05 \qquad 4\text{-}3$$

where h is the height of the structure in meters and should not include any minor top stories that are of markedly lower stiffness and mass than the stories below (e.g., plantrooms, and lightweight penthouse structures). Where below grade basement conditions occur, h is typically assumed to be the height above-grade, and ξ is subject to the below restrictions:

- When evaluating unclad structural steel buildings for the case when the members of the lateral load resisting system are connected by means of welded connections, ξ should not exceed 0.01.
- For all other situations, ξ need not be taken less than 0.025 when evaluating ULS and CALS.
- For structures using seismic isolation technology or enhanced energy dissipation technology, the equivalent viscous damping ratio selected should conform with the relevant guidelines.

Higher target elastic equivalent viscous damping ratios are permitted if substantiated through analysis or test data.

Equivalent viscous damping can be modeled using rational methods. Where equivalent viscous damping is implemented using mass and stiffness proportional methods, some suggested bounds to the target equivalent viscous damping ratios are listed below:

1. The average equivalent viscous damping ratio, weighted by the mass participation ratios over all modes required to achieve 90% mass participation, should not exceed the target equivalent viscous damping ratio; and,
2. The damping ratio provided in the highest translational mode required to achieve 90% mass participation is no more than eight times that considered for the first translational mode, unless substantiated through analysis or test data; and,
3. The elastic equivalent viscous damping ratio for all modes in the range of 0.2 times and 1.5 times the fundamental period in each direction is no more than the target elastic effective viscous damping ratio. Alternative period range values may be justifiable in some instances (refer to Commentary of Section 3.3 for relevant sets of considerations regarding modal response characteristics).

Commentary:

The provisions for inclusion of equivalent viscous damping are based on ASCE 41-23 (ASCE, 2023), and approaches documented in other publicly available guidelines (e.g., LATBSDC 2023). In comparison with ASCE 7-22, these documents provide more useful guidance and requirements for equivalent viscous damping. Care should be taken when including initial stiffness proportional damping to ensure that spurious load paths are not introduced which may unduly influence the analysis results.

4.2.6 Explicit Foundation Modelling

Foundation flexibility, including piles and the supporting soils with which they interact, should be included in the analysis when they significantly affect the dynamic properties of the building. When the stiffness and/or damping of the ground is included in the model, horizontal input ground motions should be applied to the horizontal grounded joints of the soil elements rather than being applied to the foundation directly. When foundation flexibility is included within the model, the sensitivity of the analysis results to variations in the assumed strength and stiffness should be evaluated.

4.3 Analysis Results and Acceptance Criteria

Where the analysis is used to demonstrate the performance of the structure with respect to life-safety requirements (represented by CALS), the global acceptance criteria of Section 4.3.1 and the element-level acceptance criteria of Section 4.3.2 should be satisfied.

Not more than one ground motion from the suite should produce unacceptable response as defined in Section 4.3.1.1. Where a ground motion produces unacceptable response, the design action effect should be taken equal to 120% of the of the median value of the entire suite of analyses, but not less than the mean value obtained from the suite of analyses producing acceptable response. When no unacceptable response is present, the design action effect should be taken equal to the mean value from the suite of analyses. Mean values should only be determined for result quantities of the same sign.

Commentary:

The criteria contained in this section are consistent with ASCE 7-22, Section 16.4 with the following exceptions:

- *ASCE 7-22 does not allow any unacceptable response when ground motion records are scaled via spectral matching. The proposed approaches to spectral matching included in this document ensure that adequate record-to-record variability is maintained and as such this limitation is considered redundant.*

- *ASCE 7-22 provisions only allow unacceptable response for Category I & II structures. It is considered that this limitation is unnecessary when considering CALS derived in accordance with TS 1170.5 which directly includes consideration of building Importance Level.*

When evaluating response quantities via a mean, care should be taken to ensure that the considered values are of the same sign to ensure that the calculation is not unduly affected by any asymmetry in the response (e.g., as may be the case for elements subject to significant permanent actions).

4.3.1 Global Acceptance Criteria

4.3.1.1 Unacceptable Response

Unacceptable response to ground motion consists of any of the following:

1. Analytical solution fails to converge,
2. Predicted demands on deformation-controlled elements exceed the valid range of modelling,
3. Predicted demands on force-controlled elements exceed their expected strength (i.e. considering expected material properties and a strength reduction factor equal to 1.0),
4. Predicted deformation demands on elements not explicitly modelled exceed the deformation limits at which the members are no longer able to carry their gravity loads,
5. Peak transient storey drift ratio exceeds 150% of the permissible value of mean transient storey drift, as per Section 4.3.1.2.

<u>*Commentary:*</u>

Unacceptable Response: The criteria for unacceptable response follow the provisions of ASCE 7-22, Section 16.4.1.1. The exception to this is the consideration of residual storey drift which the commentary to ASCE 7-22 identifies is not associated with life-safety performance objective. Whilst there may be some instances where consideration of residual storey drift has some merit, it is considered to be beyond the scope of New Zealand Building Code requirements and thus omitted in this document.

4.3.1.2 Transient Storey Drift

The transient storey drift ratio is computed as the absolute value of the largest difference of the deflections of vertically aligned points at the top and bottom of the storey under consideration along any of the edges of the structure within a single response history analysis. The mean transient storey drift for the suite of ground motions should be used to evaluate the adequacy of the structure to each limit state as below:

- For serviceability limit states (SLS1 and SLS2), the relevant criteria from TS 1170.5 should be adopted directly.
- For collapse avoidance limit state (CALS), the mean transient storey drift should be divided by ψ_{CALS} and compared to the ULS requirements included within TS 1170.5 Section 7.5.1.

<u>*Commentary:*</u>

NZS1170.5:2004 identifies that for the purposes of determining drift demands, the results for any ground motions which include forward-directivity effects should be scaled by 0.67. It should be noted that this post-hoc scaling of drift demands is not permitted to be applied when using this guideline.

4.3.1.3 Transient Horizontal Deflection

The transient horizontal deflection should be computed at all applicable locations for comparison with the distance to adjoining sites. The mean transient horizontal deflection for the suite of ground motions should be used to evaluate the adequacy of the structure with respect to the requirements of TS 1170.5.

The mean transient horizontal deflection at any point may be taken as that determined considering CALS divided by ψ_{CALS} when evaluating performance with respect to the ULS requirements of Section 7.4 of TS 1170.5.

4.3.2 Element-Level Acceptance Criteria

All element actions are required to be classified either as force-controlled or deformation-controlled, in accordance with relevant materials standards.

For each element action, the design action effect, E_d, is computed with consideration of the level of observed unacceptable response as outlined in Section 4.3.

Force-controlled actions are evaluated for acceptability in accordance with Section 4.3.2.1. Deformation-controlled actions are evaluated for acceptability in accordance with Section 4.3.2.2.

4.3.2.1 Force-Controlled Actions

Force controlled actions are required to be evaluated with respect to the demands derived considering CALS. All actions should be classified as either “Critical”, “Non-Critical”, or “Ordinary” as appropriate and satisfy Equation 4-5:

$$G + \psi_E Q + \gamma\left(E_d - E_{d,ns}\right) \leq \phi R_n \qquad \text{4-5}$$

where G, ψ_E , Q, and $E_{d,ns}$ are as defined previously, γ is a factor to account for record-to-record variability and should be taken equal to 1.3, E_d is the design action effect evaluated in accordance with Section 4.1, R_n is the nominal (characteristic) strength of the component determined in accordance with the relevant material standard, and ϕ is the strength reduction factor.

The strength reduction factor, ϕ, may be taken equal to 0.9 for Ordinary actions, 1.0 for Non-Critical actions and should be taken as specified for non-capacity protected items in accordance with the applicable material standard for Critical actions. Premature failure of connections which would prevent the reliable formation of the assumed inelastic mechanism should be prevented. This may be achieved by ensuring that the connections of deformation-controlled elements are designed in accordance with the principles of capacity design. Design of connections for seismic isolation or enhanced energy dissipation devices should conform with the relevant guidelines.

<u>Commentary:</u>

The consideration of force-controlled actions closely follows the provisions of ASCE 7-22, Ch. 16.4.2.1 with several notable exceptions:

- *The governing equation has been modified to align with the requirements of AS/NZS 1170.0:2002 Section 4.2.2(f) which does not require consideration of either 0.9G (without live-load) or 1.2G in conjunction with seismic demand.*
- *The direct contribution of vertical ground motion in this equation has been omitted to be consistent with the requirements of linear design procedures of TS 1170.5.*
- *The influence of building Importance Level has been omitted on the basis that it has been included in the definition of the target spectrum.*

Special attention must be paid to use of equations that contain the $(E_d - E_{d,ns})$ *term. Since the superposition rules do not apply to nonlinear analysis, in cases where gravity force distribution is highly unsymmetrical and/or in cases where strong directionality exists in building response where forces in one direction along an axis are significantly larger than the same forces in the other direction of the same axis, orbital plots or contours should be plotted to make sure that straight use of the* $(E_d - E_{d,ns})$ *term does not produce unconservative results.*

4.3.2.2 Deformation-Controlled Actions

The valid range of modelling for deformation-controlled element actions should be consistent with that established in the applicable material design standard. Where the material design standard does not specify the valid range of modelling, this parameter should be established as the maximum value of the parameter at which the element model is capable of replicating the hysteretic behaviour and load-carrying capability observed in laboratory testing of similar elements. It should be permitted to extend the valid range of modelling for an element beyond these deformations if the element strength and stiffness are degraded to negligible values once these deformations are reached.

Commentary

It is an expectation of this document that future revisions to TS 1170.5 will direct New Zealand materials standards to specify suitable deformation limits for both SLS and CALS. In the absence of this, contemporaneous international literature (e.g., ASCE 41, ACI 318 (ACI, 2019), ANSI/AISC 341 (AISC, 2022)) should be consulted for suitable values.

It is a requirement that the force-deformation relationships adopted in the analysis do not extend beyond the range determined via testing. This does not preclude the element from experiencing deformations exceeding this value, but rather that if this deformation is to occur, the resistance of the element is reduced to a value such that it no longer contributes to the strength of the structure.

4.3.3 Secondary Elements

Secondary elements which are not part of the primary seismic force-resisting system, and are not specifically included in the analysis, should be demonstrated to be capable of supporting their applicable gravity loads using the mean building displacements from the suite of nonlinear response history analyses.

5 Parts and Components

5.1 Overview

This chapter provides guidance for determining horizontal design actions for parts of structures and non-structural components when NLRHA is used as the structural analysis method. In addition to the conventional procedure detailed in Section 8 of TS 1170.5 (SNZ, 2024), this chapter provides two additional methods for computing design actions for parts and components:

1. Use output from NLRHA to determine design response coefficient for parts and components, $C_p(T_p)$ for use within TS 1170.5.
2. Develop project specific floor response spectra.

Further guidance on these two methods is provided below.

5.2 Using NLRHA to Determine Parts Design Response Coefficient

Horizontal design earthquake actions on parts and components, F_{ph}, is defined in TS 1170.5 as:

$$F_{ph} = \frac{C_p(T_p)}{\Omega_p} R_p W_p \leq \frac{7.5\,PGA\,W_p}{\Omega_p} \qquad 5\text{-}1$$

Where:

$C_p(T_p)$ the horizontal design coefficient of the part or component determined from TS 1170.5 Clause 8.2

Ω_p the part or component reserve capacity factor, taken as 1.5 for ULS and 1.0 for SLS1 and SLS2, unless demonstrated to be greater

R_p the part or component risk factor as given by TS 1170.5 Table 8.1

PGA the peak ground acceleration, determined from TS 1170.5 Clause 3.1.2

W_p the weight of the part or component

When using NLRHA to design a structure with rigid diaphragms, the horizontal design coefficient, $C_p(T_p)$, can be computed directly from the analysis using Equation 5-2:

$$C_p(T_p) = a_i \left[\frac{C_i(T_p)}{C_{ph}}\right] \qquad 5\text{-}2$$

Where:

a_i mean of the maximum values of peak floor acceleration at the centre of mass of the support level, obtained from each ground motion for the limit state being considered.

$C_i(T_p)$ the part or component spectral shape coefficient, determined from TS 1170.5 Clause 8.5

C_{ph} the part or component horizontal response factor, determined from TS 1170.5 Clause 8.6

When assessing, a_i, the maximum values of acceleration at the support level, accidental eccentricity effects may be neglected.

<u>Commentary</u>

When NLRHA is used to design a structure, there are several options available for calculating the design response coefficient, $C_p(T_p)$, for parts and components: $C_p(T_p)$ can either be computed using the basic equation in TS 1170.5, Equation 8.2(1), or the designer may choose to take advantage of the output from the NLRHA. The provisions in Section 5.2 have been developed from ASCE 7-22 (ASCE, 2022).

The intent is that the entire suite of ground motions used to design the structure should also be used to determine $C_p(T_p)$. Structures with significant horizontal irregularities may experience large torsional response. ATC-120 project (ATC, 2017) investigated the influence of torsional response of the structure on floor accelerations experienced by components and concluded that due to the complexity of the problem and the limited information available, additional study is needed before it may be directly included in the design equations.

Work completed by Haymes et al (2023) shows the provisions included in this section provide reliable estimates of horizontal design earthquake actions on parts and components in base isolated buildings when $T_{p,long}$ is taken as the effective period of the isolation system, T_{eff}. A limited review of the resulting floor spectra obtained via NLRHA indicates that the provisions are also likely to provide reasonable estimates for buildings with fluid viscous dampers.

When assessing, a_i, the maximum values of acceleration at the support level, accidental eccentricity effects may be neglected. The mean response value was judged to be adequate for computing $C_p(T_p)$ and is consistent with what was used when the TS 1170.5 parts and components provisions were developed.

Equation 5-2 was developed for buildings with rigid diaphragms. For buildings where diaphragm flexibility is relevant , a_i can tentatively be taken as either:

1. *The mean of the maximum values of peak floor acceleration recorded anywhere within the flexible diaphragm at the support level, obtained from each ground motion for the limit state being considered, or*
2. *The mean of the peak floor acceleration values, obtained for each ground motion, in the flexible diaphragm at the location where the part is supported for the limit state being considered.*

For buildings with flexible diaphragms, $T_{p,long}$ should be taken as the largest period of vibration of the vertical and horizontal (i.e. including flexible diaphragms) lateral load resisting systems, in each direction being considered.

5.3 Development of Project Specific Floor Spectra

When using NLRHA to analyse a building, horizontal design earthquake actions on parts and components can be determined using project-specific floor response spectra. This procedure is outlined below.

Floor acceleration response spectra should be computed directly from floor acceleration time-history output from NLRHA models (Chopra, 2014). A critical damping ratio of 5% should be used when computing floor response spectra unless it can be demonstrated that an alternative damping value is appropriate. Absolute floor accelerations should be used and the acceleration time history data should be extracted from the analysis model at the location of interest for each ground motion record, in each direction. Analysis models should represent the flexibility of floor diaphragms when this is significant to the response of the structure or part being considered. Accidental eccentricity effects may be neglected when computing floor spectra.

Idealised floor spectra should be developed at each location of interest using the following procedure:

1. Floor spectra should be computed as the mean from the suite of ground motions, for each horizontal direction. The envelope of these two mean spectra should be used to develop the design spectrum.
2. A design spectrum should be developed which is linear between points of interest up to point E like that illustrated in Figures 5-1 and 5-2 below:
   a. The design spectrum should envelope the output spectra at all periods.
   b. Point A should be taken as the mean peak floor acceleration.
   c. The line from point A to B should be horizontal and extend to 0.06s.

d. The line from point B to C should be vertical and extend to the spectral acceleration plateau.

e. The spectral acceleration plateau should extend from point C to D, where point D is set so it is not less than a period equal to the lower of:

   i. The period at which the peak spectral acceleration demand is observed multiplied by 1.5, or

   ii. The period at which the peak spectral acceleration demand is observed plus 1.0s.

f. The line from point D to E can be vertical or have a negative slope.

g. Beyond point E a spectral shape consistent with the displacement demands can be used.

Point E on the idealised design spectra is analogous to $T_{p,long}$ in TS 1170.5 and represents the upper bound of the period range over which we could expect resonant excitation of a part due to the dynamic response of the building. For base isolated buildings $T_{p,long}$ can be taken as the effective period of the isolated building, T_{eff}, as defined in the Guideline for the Design of Seismic Isolation Systems for Buildings (NZSEE, 2019).

Referring to Figure 5-1 below, which illustrates an example of an idealised floor spectra for a conventional building, the orange and blue lines represent the computed mean floor spectra in the orthogonal horizontal directions. Spectral peaks at ~0.45s and ~1.3s correspond to the second and first translational modes of the building respectively. In this instance point D has been extended beyond the minimum values recommended above to envelope the underlying spectra and a vertical drop between points D & E has been adopted (refer to the commentary for details of the dashed black line which starts at Point B).

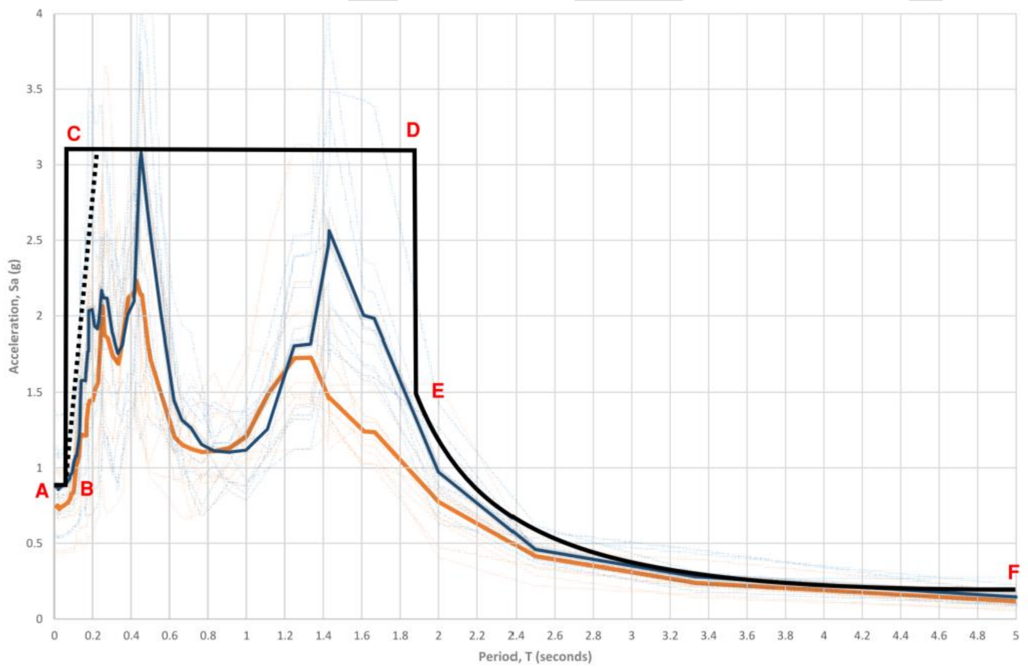

Figure 5-1: Example of an idealised floor spectra for a conventional building.

Referring to Figure 5-2 below, which illustrates an example of an idealised floor spectra for a base isolated building, points A – D have been derived in a similar manner to that shown in Figure 5-1 above. Point E represents T_{eff} of the isolated building. The sharp drop off in spectra beyond point D is not observed, and engineering judgement will be required to draw the idealised spectra for this case. In this instance a sloped line between points D and E has been adopted. Behaviour beyond point F is likely to be displacement controlled.

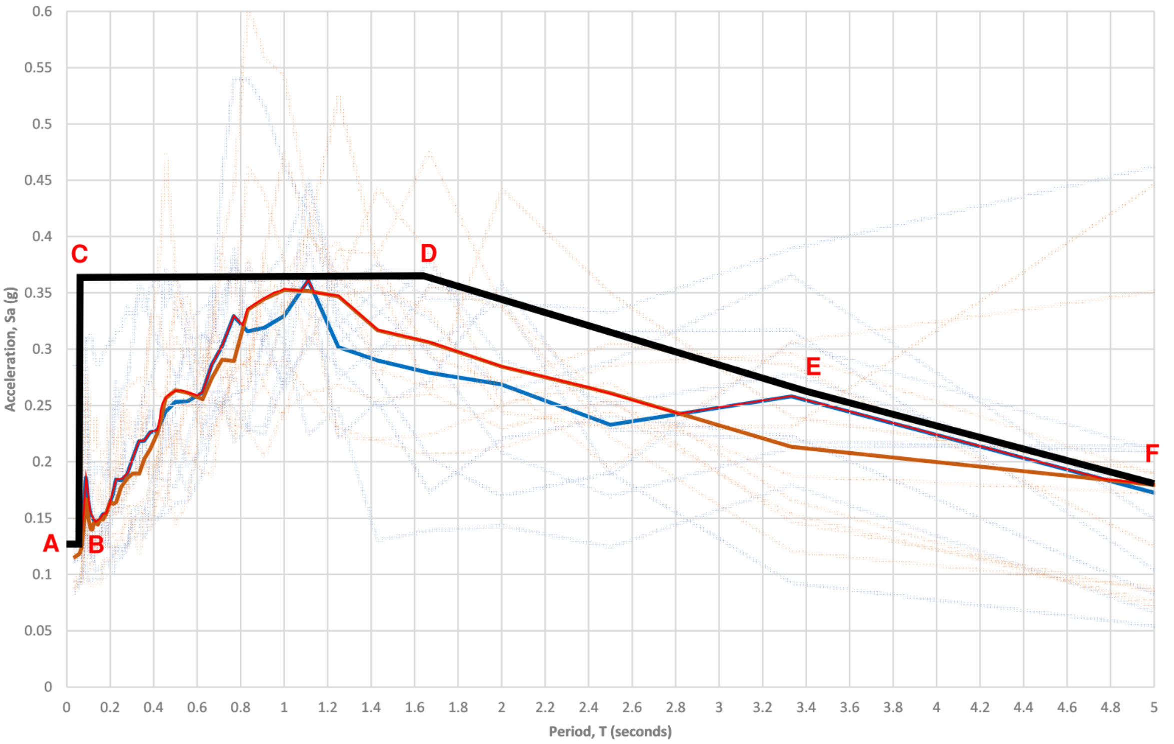

Figure 5-2: Example of an idealised floor spectra for a base isolated building.

For parts designed for the peak acceleration of a 5% damped spectrum, the part response factor C_{ph} may be determined as per Table 8.3 of TS 1170.5:

1. Between points A and B the C_{ph} factor for rigid components can be used.
2. Between points C and D the C_{ph} factor for flexible components can be used.
3. Beyond point E the C_{ph} factor for long period components can be used.

For those cases when the line on the design spectra between points D and E has a slope C_{ph} can be determined via linear interpolation using the values in Table 8.3 of TS 1170.5 for flexible and long period components.

If critical damping ratios greater than 5% are used to compute the spectrum the C_{ph} factors in TS 1170.5 will no longer be applicable and C_{ph} should be taken as 1.0 unless it can be demonstrated that an alternative damping value is appropriate.

Note that design of diaphragms and large structural components using parts and components spectra is generally not appropriate.

<u>Commentary</u>

This procedure differs from the procedure in 5.2 in that the $C_i(T_p)$ is specifically represented by development of floor spectra from a numerical model of the lateral force resisting system. This procedure assumes that appropriate element properties are used and sensitivity analyses are carried out such that the floor spectra will be a reasonable representation of structure behaviour – for example upper bound/ probable capacity models will generally govern.

Many analysis packages specifically calculate such spectra, or the designer may develop spectra using the acceleration/ time data at a point. It is recommended that the design spectra be the envelope of the two directions, to eliminate potential confusion in passing information to the part designer, who may be under separate engagement.

Peaks in the spectra generally correlate with the translational modes of the primary structure. Where the structure remains largely elastic, there tends to be distinct spectral peaks, often at the first and second modes. Where this occurs, the acceleration demand may be taken as linear between peaks, i.e. parts should not be designed to be in the low point between them, as there is significant uncertainty in the calculation of the period of both the structure and the part.

The 1.5 multiplier on the extension of the peak demand is intended to account for uncertainties in the period calculation of both the structure and the part. The multiplier is based on engineering judgement. The 1.0s maximum is for long periods, where uncertainty is less and the 1.5 multiplier is judged to be unduly conservative.

Specific calculation of C_{ph} is theoretically possible for a project by development of inelastic spectra based on the acceleration trace at a point, however this is likely to be very time intensive and therefore has not been explored within this guideline.

Many parts in real buildings are very stiff, such that they move with the floor and do not experience significant resonance. This is represented in TS 1170.5 by the $C_i(T_p)$ factor of 1.0 for rigid parts. Refer to the TS 1170.5 commentary for guidance on the classification of what constitutes a rigid part. Generally, this is a qualitative assessment due to the uncertainties and potential inaccuracies in calculating a period when displacements are so small. A rigid part is likely to have a period of 0.06s or less.

When parts have a period between 0.06s and the period of the first spectra peak, linear interpolation between the peak floor acceleration and the peak spectral acceleration may be used. The rising part of the spectra should be linear between point B, and the start of the spectral acceleration plateau which should have a period of not more than 0.5 times the period of the first spectra peak. The resulting design spectrum should envelope the output spectra over this period range (refer to Figure 1 where the black dashed line starting from point B is an example of how the interpolation is intended to be implemented). For this situation it is recommended that the period of the part be determined by means of experimental testing. If the period of the part is determined by means of calculation, due consideration should be given to all sources of flexibility that may be present including connections, isolation mounts, and other support structure.

Where the rising part of the spectra is used, the base build engineer should verify the period provided by the designer of the part and C_{ph} should be taken as 1.0. If verification with the base building engineer is not done, the peak spectral acceleration should be used.

6 References

Abrahamson, N. A., (1992). *Non-stationary spectral matching*, Seismol. Res. Lett. 63, 30

Ancheta, T.D., Darragh, R.B., Stewart, J.P., Seyhan, E., Silva, W.J., Chiou, B.S.-J., Wooddell, K.E., Graves, R.W., Kottke, A.R., Boore, D.M., Kishida, T., and Donahue, J.L., (2014). *NGA-West 2 database*, Earthquake Spectra, 30, 989–1005

ACI (2019), *ACI 318-19 Building Code Requirements for Structural Concrete*, American Concrete Institute, Farmington Hills, MI

AISC (2022), *ANSI/AISC 341-22 Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL

ASCE (2006), *ASCE/SEI 7-05 Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA

ASCE (2010), *ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA

ASCE (2022), *ASCE/SEI 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, American Society of Civil Engineers, Reston, VA

ASCE (2023), *ASCE/SEI 41-23 Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, Reston, VA

ATC (2010), *PEER/ATC 72-1 Modeling and acceptance criteria for seismic design and analysis of buildings*, Applied Technology Council, Redwood City, CA 94065

ATC (2017), *NIST GCR 17-917-44 Seismic Analysis, Design and Installation of Nonstructural Components and Systems – Background and Recommendations for Future Work*, Applied Technology Council, Redwood City, CA 94065

Baker, J., Bradley, B., & Stafford, P. (2021). *Seismic Hazard and Risk Analysis*. Cambridge: Cambridge University Press.

Baker, J.W. and Cornell, C.A. (2006). *Spectral shape, epsilon and record selection*. Earthquake Engineering & Structural Dynamics, 35(9), 1077-1095.

Baker, J.W. (2011). *The Conditional Mean Spectrum: a tool for ground motion selection*, Journal of Structural Engineering, Vol. 137, No. 3, pp. 322-331.

Boore, D. M., J. Watson-Lamprey, and N. A. Abrahamson (2006), *Orientation-independent measures of ground motion*, Bull. Seismol. Soc. Am.96, 1502–1511.

Bradley, B. A., 2010. *A generalized conditional intensity measure approach and holistic ground motion selection*. Earthquake Engineering and Structural Dynamics, 39(12), 1321–1342. DOI: 10.1002/eqe.995

Bradley, B. A., 2011. Design Seismic Demands from Seismic Response Analyses: A Probability-Based Approach. *Earthquake Spectra*, **27**(1), 213–224.

Bradley, B. A., 2014. Seismic performance criteria based on response history analysis: Alternative metrics for practical application in NZ. *Bulletin of the New Zealand Society for Earthquake Engineering*, **47**(3), 224–228.

CEN (2004), *Eurocode 8 : Design Provisions for Earthquake Resistance of Structures*, European Committee for Standardisation, Brussels, Belgium

Chopra, A.K. (2014), *Dynamics of Structures, Theory and Applications to Earthquake Engineering*, 4th edition, Pearson Education, Essex, England

Elms, D.G. (1976), *Seismic Torsional Effects on Buildings*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 9, No. 1, 1976

Golesorkhi and Gouchon (2023). *Near-Fault Ground Motions*, White Paper prepared for the 2026 NEHRP Provisions Update Committee (PUC) Issue Team #7 Nonlinear Analysis Working Group-1, Ground Motions.

Gulerce, Z., and Abrahamson, N. A., 2011. *Site-Specific Design Spectra for Vertical Ground Motion*. Earthquake Spectra, 27(4), 1023–1047.

Haselton CB, Fry A, Hamburger RO, Baker JW, Luco N, Elwood KJ, Hooper JD, Charney FA, Zimmerman R, Pekelnicky RG and Whittaker AS (2017). *Response history analysis for the design of new buildings in the NEHRP provisions and ASCE/SEI 7 standard: Part II - Structural analysis procedures and acceptance criteria*. Earthquake Spectra, 33(2): 397–417.

Haymes, K., Sullivan, T.J., Chandramohan, R. and Weibe, L., *Procedures for the Prediction of Floor Response Spectra in Base Isolated Buildings*, Proceedings 18th World Conference of Earthquake Engineering Conference 2024, Milan, Italy

Huang, Y-N., Whittaker, A. S., and Luco, N. (2008). *Maximum Spectral Demands in the Near-Fault Region*. Earthquake Spectra, Volume 24, No. 1, pp. 319-341.

LATBSDC (2023), *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings*, Los Angeles Tall Buildings Structural Design Council, Los Angeles, CA

Luco, N., and Bazzurro, P., 2007. *Does amplitude scaling of ground motion records result in biased nonlinear structural drift responses?* Earthquake Engineering and Structural Dynamics, 36(13), 1813–1835. DOI: 10.1002/eqe.695

Mazzoni, S. (2022): *NGA-Subduction Portal: Ground-Motion Record Selection and Download*. The B. John Garrick Institute for the Risk Sciences. Interactive Resource. (https://doi.org/10.34948/N3D59V)

Mazzoni, S., Hachem, M., and Sinclair, M., (2012). *An improved approach for ground-motion suite selection and modification for use in response history analysis*, Proceedings of Fifteenth World Conference on Earthquake Engineering, Lisbon, 2012.

Morris, G.J., Thompson, A.J., Dismuke, J.A. and Bradley, B.A. (2019). *Ground Motion Input for Nonlinear Response History Analysis: Practical Limitations of NZS 1170.5 and Comparison to US Standards*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 52, No. 3, 2019

National Institute of Standards and Technology (NIST), (2011). *Selecting and Scaling Earthquake Ground Motions for Performing Response History Analysis, NIST/GCR 11-917-15*, prepared by the NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, MD.

NZSEE (2019), *Draft Guidelines for the Design of Seismic Isolation Systems for Buildings*, New Zealand Society for Earthquake Engineering, Wellington, NZ

NZSEE (2022), *Earthquake Design for Uncertainty: Advisory*, Jointly prepared by New Zealand Society for Earthquake Engineering, Structural Engineering Society New Zealand and New Zealand Geotechnical Society, Wellington, NZ

NZSEE (2024), *Engineering Assessment Guidelines - Part C1 General Issues (Public Draft),* New Zealand Society for Earthquake Engineering, Wellington, NZ

PEER (2017), *Guidelines for Performance-Based Seismic Design of Tall Buildings*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA

Priestley, M.J.N., Calvi, G.M. and Kolwasky, M.J. (2007). *Displacement-based seismic design of structures*, IUSSS Press. Pavia, Italy.

SANZ (2002), *AS/NZS 1170.0 Structural Design Actions Part 0: General Principles*, Standards Australia / New Zealand, Wellington, NZ

SESOC (2021), *SESOC Commercial Design Features Report Template*, Structural Engineering Society New Zealand Wellington, NZ

SESOC (2022), *Interim Design Guidance - Design of Conventional Structural Systems*, Structural Engineering Society New Zealand Wellington, NZ

Shahi, S. K and Baker, J. W. (2012). *Directionality of Ground Motions in the NGA-West2 Database*. Presentation downloaded from PEER Website. https://apps.peer.berkeley.edu/ngawest2/wp-content/uploads/2012/07/Shahi_Baker_NGA2_directionality-Nov-15-2012-copy.pdf

Shahi, S. K., and Baker, J. W., 2014. NGA-West2 Models for Ground Motion Directionality. Earthquake Spectra, 30(3), 1285–1300. DOI: 10.1193/040913EQS097M

SNZ (2004), *NZS 1170.5 Structural Design Actions Part 5: Earthquake Actions – New Zealand*, Standards New Zealand, Wellington, NZ

SNZ (2017), *NZS 3101 inc. A1 – A3 Concrete Structures Standard*, Standards New Zealand, Wellington, NZ

SNZ (2024), *TS 1170.5 Structural Design Actions Part 5: Earthquake Actions – New Zealand*, Standards New Zealand, Wellington, NZ

Tremayne, B. & Kelly, T. E. (2005), *Time History Analysis as a Method of Implementing Performance Based Design*, Conference Proceedings of the New Zealand Society for Earthquake Engineering, Taupo, New Zealand, March, 2005.

Watson-Lamprey, J. (2006). *Selection of ground motion time series and limits on scaling*. Soil Dynamics and Earthquake Engineering, 26, 477–482.

Watson-Lamprey, J. A. and Boore, D. M. (2007). *Beyond SaGMRotI: Conversion to SaArb, SaSN, and SaMaxRot*. Proceedings of Bulletin of Seismological Society of America, Vol. 96, No. 5, pp. 1511-1524.

Zaidi, F. Hulsey, A. & Elwood, K.J. (2024). *A risk-based framework to account for step-change behaviour in seismic assessment, Proceedings of NZSEE 2024 Annual Conference, Wellington.*

Appendix A Industry Research Needs

Table A-1 Summary of industry research needs identified during the preparation of this document.

No	Description	Section Reference
1	<u>Determination of suitable value for ψ_{CALS}.</u> Section 2.5 proposes that ψ_{CALS} be taken equal to 1.5. Research is required to confirm appropriate value(s) for this term which ensure the targeted outcomes are achieved. The actual value would likely be dependent on the fragility of the given structural form and the hazard curve for the location under consideration. It is noted the NZSEE base isolation guidelines indicate that ψ_{CALS} may be sensitive to importance level.	2.5
2	<u>Structural Performance Factor, S_p</u> Additional research is required to determine whether the current implementation of S_p within these guidelines to reduce the target spectrum provides the desired outcomes. This includes addressing concerns that response parameters determined by means of a NLRHA should not result in building designs that are more conservative than would be the case if conventional Equivalent Static or Modal Response Spectrum analysis methods were undertaken in accordance with TS 1170.5.	2.5
3	<u>2022 NZ National Seismic Hazard Model Web Portal</u> As noted in Section 3.2 if would be advantageous to expand the functionality of the 2022 NZ NSHM web portal to: ▪ include disaggregation information for all locations. ▪ Provide additional information identifying the proportion of ground motions which have forward directivity effects for each location. ▪ Information to enable the development of CMS.	3.2
4	<u>Near Fault Directivity Effects</u> Section 3.2.1 discussed some of the currently known challenges associated with capturing near-fault directivity effects & implementation within the TS 1170.5 framework. Two of these key issues are summarized as follows: ▪ As noted in Bradley et al. (2022), the NSHM 2022 workplan did not include explicit directivity effects and therefore post-hoc adjustments may be required to the PSHA outputs. ▪ Unlike the NLRHA procedure, one limitation of the use of Linear analysis procedures for near-fault regions, and at high APoE, is the inability to represent the influence of directivity phenomena on structural response. As a potential means of addressing this issue, one potential solution would be to have a different target response spectrum for NLRHA and linear analysis procedures. Linear procedures could maintain a prescriptive set of near-fault factors which amplify the long period spectrum. Precedent for this dates back to the 2005 Edition of ASCE 7 (i.e. ASCE 7-05), for example, albeit first described in the ASCE 7-10 Commentary C12.8.1 which states: "Equation (12.8-7) applies to sites near major active faults (as reflected by values of S1), where pulse-type effects can increase long period demands". This research need would potentially involve benchmarking analysis & evaluation of the differences in structural response between NLRHA and linear procedures. A contemporary research study similar to the prior work of Tremayne & Kelly (2005) is recommended.	3.2.1

5	Vertical Ground Motions As detailed in Section 3.2.3.1the provisions in TS 1170.5 for selecting and scaling vertical ground motions are challenging to implement. Research is needed to determine if approach (1) detailed in Section 3.2.3.2 will be provide an acceptable level of seismic performance for some buildings or if the more complicated approach (2) should be adopted.	3.2.3.1
6	Classification of structural elements as either "Primary" or "Secondary". Commentary Clause C2.6 of TS1170.5 introduces the concept of classifying structural elements as either "Primary" or "Secondary" but little information or guidance is provided on how this distinction should be made. This distinction applies to all analysis types and additional guidance should be provided (e.g. as included in ASCE 41-23 Section 7.2.4.3)	4.2.1
7	Transient Storey Drift When evaluating CALS performance, the transient storey drift is scaled down to allow direct comparison to the ULS drift limits from TS1170.5. Members of the working group questioned the need to explicitly consider transient store drift for CALS given the level of rigour included in the analysis with regards to stability checks, and if it were to be checked whether this is an appropriate measure. Additional research is required to identify whether drift limits are required to be considered for CALS demand and where appropriate provide a recommended limit (noting that it may be suitable to consider different limits depending on whether the analysis considers small or large-displacements).	4.3.1.2
8	Design Criteria for Force-Controlled Actions Members of the working group identified shortcomings with the ASCE7 approach to force-controlled actions and instead favoured the use of the peak response from the suite. This approach would have several advantages in terms of workflow and is consistent with capacity design principles adopted within NZ standards. It is also noted that this would effectively mimic the provisions included in Section 4.3.1.1 for Unacceptable Response and therefore specific consideration of force-controlled actions would not be required. However, the working group identified additional research is required to recommend this approach in place of the established procedures in ASCE7 to ensure an appropriate level of seismic performance is achieved.	4.3.2.1
9	Deformation Limits New Zealand materials standards do not currently specify suitable deformation limits that can be used for CALS (and for SLS2 and DCLS). In the absence of this information designers are currently referring to international standards such as ASCE 41 (ASCE, 2023), ACI 318 (ACI, 2019) and ANSI/AISC 341 (AISC, 2022). To support the uptake of NLRHA in New Zealand it would be beneficial if the New Zealand materials standards were able to specify the necessary deformation limits. Furthermore, TS1170.5 should explicitly specify how the deformation limits should be derived (e.g., median vs 95th %ile, etc.).	4.3.2.2
10	Parts and Component Demands for Buildings with Flexible Diaphragms The provisions in TS 1170.5 and Section 5.2 of this document were derived from ASCE 7-22. As noted in NIST GCR 18-917-43 (ATC, 2017), the ASCE 7-22 provisions did not included consideration of amplification in floor acceleration due to diaphragm flexibility. Midspan PFA is often substantially larger than PFA adjacent shear wall locations. Research is needed to determine if the increased flexibility associated with flexible diaphragms results in parts and component demands that are significantly higher than would be computed using the provisions in TS 1170.5.	5.2

11	Parts and Component Demands for Buildings with Supplemental Damping The provisions in Section 5.2 were derived from ASCE 7-22. ASCE 7-22 references NIST GCR 18-917-43 as basis for the procedure. It is unclear from NIST GCR 18-917-43 how much consideration was given to buildings with supplemental damping when the recommended procedures in this document were developed. In the preparation of this document output from a limited number of buildings with supplemental damping was reviewed. It was judged the proposed procedures would likely provide reasonable estimates of parts and component demands for typical buildings, however more research is needed to confirm this.	5.2
11	Project Specific Floor Spectra The proceed in Section 5.3 for defining the width of the spectral acceleration plateau (i.e. between points C and D) is based on engineering judgement, however more research would be beneficial to confirm if it is providing an acceptable level of seismic performance for building parts and components.	5.3

Appendix B Typical Force-Controlled Actions and Categories

In these guidelines, member actions are classified as either deformation or force controlled. Table B-1 provides a list of typical force-controlled actions and suggested categories. Individual design and peer review teams should consider this list when formulating the categorisation of component actions for specific projects and supplement and modify as is appropriate to those projects. Information in Table B-1 has generally been sourced from ASCE 7-22.

When actions on force-controlled elements are limited by formation of a well-defined yield mechanism via capacity design provisions outlined in an appropriate New Zealand material standard these need not be evaluated in accordance with the criteria for force-controlled elements (refer Section 4.3.2.1).

Table B-1 Suggested force-controlled actions and categories.

Action		**Category**		
		Critical	Ordinary	Non-Critical
Steel Moment Resisting Frames	Axial compression forces in columns caused by combined gravity and seismic demands	X		
	Combined axial, bending and shear in column splices	X		
	Tension in column base connections[1]	X		
	Shear forces beams and columns		X	
	Actions other than tension in column base connections[1]		X	
	Welded or bolted joints between moment frame beams and columns[2]		X	
Steel Braced Frames (CBF, EBF and BRBF)	Axial compression forces in columns caused by combined gravity and seismic demands	X		
	Combined axial, bending and shear in column splices	X		
	Tension in brace and collector beam connections	X		
	Actions in column base connections[1]	X		
	Axial forces in EBF braces[1]		X	
	Axial tension forces in columns[1]		X	
Reinforced Concrete MRF	Axial compression forces in columns caused by combined gravity and seismic demands	X		
	Shear forces beams and columns	X		
	Shear forces in beam column joints	X		
	Splices in longitudinal beam and column reinforcement		X	
Reinforced Concrete Structural Walls	Shear forces in structural walls in cases where there is limited ability to redistribute actions to adjacent wall panels[3]	X		
	Axial (plus bending) compression in in structural walls	X		
	Axial compression in outrigger columns	X		
	Axial (plus bending) tension in outrigger column splices	X		
	Shear forces in structural walls not categorised as critical		X	

Action		Category		
		Critical	Ordinary	Non-Critical
	Actions associated with other failure modes that would not result in widespread collapse or significantly reduce the overall stability of the structure.		X	
Other Components	Shear forces in piles and pile cap connections[1]	X		
	Shear forces shallow foundations[1]	X		
	Punching shear in slabs without shear reinforcing[1]	X		
	In-plane forces in diaphragms that transfer a substantial amount of force (from more than one storey)	X		
	Actions in members and connections of elements supporting discontinues frames and walls	X		
	Axial forces in diaphragm collectors		X	
	In-plane forces in diaphragms not categorised as critical		X	
	Axial forces in piles		X	
	All other force-controlled actions[4]		X	

Notes:

1. Unless modelled inelastically, in which case can be treated as a deformation-controlled action.
2. As distinct from the inelastic action of the overall connection.
3. Refer to ASCE 7-22 Section C16.4.2.1 for guidance on when shear forces in structural walls can be categorized as critical force-controlled actions.
4. Other force-controlled actions should be categorised considering the criticality of the action to overall building performance. The default category recommended is ordinary. Force-controlled actions those whose failure is unlikely to lead to structural collapse or substantive loss of the seismic resistance of the structure can be categorized as noncritical.

Appendix C Design Methodology

C.1 Overview

This appendix outlines a design methodology which can be adopted for projects when NLRHA is to be used to validate the seismic performance of new structures. Figure C-1 below summarises the design methodology. The process aligns with recommended industry practice (NZSEE, 2022 and SESOC, 2022), whereby designers should deliberately proportion structures with enough regularity so that it is possible to identify a clear plastic mechanism. This will enable capacity design principles to be applied, so that should a structure's strength be exceeded, reliable plastic mechanisms can be developed.

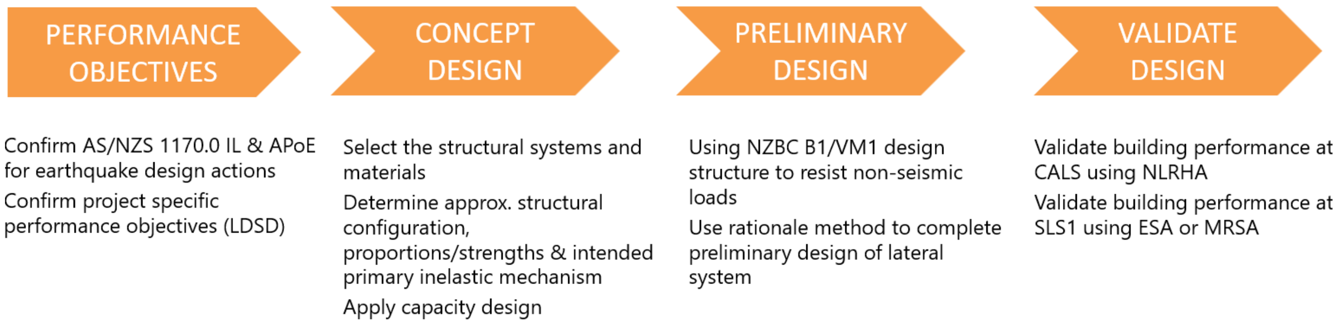

Figure C-1: Overview of design methodology

To undertake a NLRHA it is necessary to develop a preliminary design of the primary structural systems to a sufficient level of detail to enable the necessary analysis inputs to be quantified. A process for developing a preliminary design is detailed in the following sections.

C.2 Establish Performance Objectives

Select design performance objectives and design criteria appropriate to the AS/NZS 1170.0 (SANZ, 2002) Importance Level (IL). Identify any specific performance objectives when these exist for the project.

The design performance objectives and design criteria selected for a project should be clearly defined in the Design Features Report.

<u>*Commentary*</u>

Refer to Section 2.3 for further information related to building performance objectives and performance limit states that should be considered when designing structures.

C.3 Concept Design

Select the structural systems and materials; their approximate configuration, proportions, and strengths; and the intended primary mechanisms of inelastic behaviour. Apply capacity design principles to establish the target plastic mechanisms.

For all members of the structural system, define deformation-controlled (ductile) actions and force-controlled (non-ductile) actions. Categorise each forced-controlled action as being Critical, Ordinary, or Noncritical.

The selected structural configuration, structural systems and materials of construction, intended mechanisms of inelastic behaviour, and member categorisation into deformation and force-controlled actions; should be clearly defined in the Design Features Report.

Commentary

Deformation-controlled actions are those that are expected to undergo inelastic behaviour in response to earthquake shaking and that are evaluated for their ability to sustain such behaviour. Force-controlled actions are not expected to exceed their yield strength when responding to earthquake actions and are evaluated on the basis of available strength.

Critical force-controlled actions are those whose failure is likely to lead to partial or total structural collapse. Noncritical force-controlled actions are those whose failure is unlikely to lead to structural collapse. Ordinary force-controlled actions are those whose failure might lead to local collapse but are unlikely to affect the overall stability of the structure.

Appendix B provides a list of typical force-controlled actions and suggested categories. Individual design and peer review teams should consider this list when formulating the categorisation of component actions for specific projects and supplement and modify as is appropriate to those projects.

C.4 Preliminary Design

Design the structure to resist dead, live, wind, snow and other non-seismic loads to be as detailed in NZBC B1/VM1. Use a rational method to complete the preliminary seismic design of the primary lateral load resisting systems so the target plastic mechanisms identified during the concept design phase will likely be attained and the design will likely be capable of meeting the project performance objectives identified in Section C.2.

Seismic performance of the preliminary design can then validated by means of a NLRHA in accordance with the methodology detailed in Section 4.

Commentary

To perform a meaningful NLRHA it is necessary to develop the structural design to a sufficient level of detail to enable the determination of its stiffness, strength, mass, as well as the hysteretic properties of the members that are expected to undergo inelastic strain under CALS intensity ground motions. Seismic design methods that might be considered for the preliminary seismic design include:

- *Equivalent Static or Modal Response Spectrum analysis undertaken in accordance with TS 1170.5, ASCE 7-22 or Eurocode 8 (CEN, 2004)*
- *Performance based seismic design guidelines such as PEER (2017) and LATSDC (2023)*
- *Direct Displacement Base Design (DDBD) procedures developed by Priestley et al. (2007)*
- *Draft NZSEE Seismic Isolation Guidelines (NZSEE, 2019)*

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Appendix D Notation

Unless stated otherwise, this guideline uses the following notations:

a_i	Mean of the maximum values of peak floor acceleration (g) (see 5.2)
AI	Arias intensity (see 3.1)
$C_i(T_p)$	Spectral shape coefficient for a part or component (see 5.2)
$C_p(T_p)$	Horizontal design coefficient for a part or component (see 5.2)
C_{ph}	Horizontal response factor for a part or component (see 5.2)
D	Earthquake rupture distance (km) (see 3.6.1)
D_{s5-75}	5% - 75% significant duration interval (s) (see 3.1)
D_{s5-95}	5% - 95% significant duration interval (s) (see 3.1)
E_d	Design action effect (see 4.1)
$E_{d,ns}$	Non-seismic portion of action (N) (see 4.1)
E_{LS}	Earthquake action for the limit state under consideration (N) (see 4.1)
G	Permanent (self-weight or 'dead') action (N) (see 4.1)
h	Height of structure above grade (m) (see 4.2.5)
PGA	Peak ground acceleration (see 5.2)
Q	Imposed or 'live' action (N) (see 4.1)
R_n	Nominal (characteristic) capacity (N, Nm) (see 4.3.2.1)
R_p	Risk factor of a part or component (see 5.2)
S_p	Structural performance factor (see 2.5)
T_1	Fundamental period of a structure in the direction being considered (s)
$T_{90\%}$	The period at which 90% of superstructure mass participation is attained (s) (see 3.3)
T_{eff}	Effective fundamental period of a structure in the direction being considered (s) (see 5.3)
T_{lower}	Lower bound period of interest (s) (see 3.3)
T_{max}	Maximum period of interest (s) (see 3.3)
T_{min}	Smallest first-mode period for the two principal horizontal directions of response (s) (see 3.3)

T_{upper}	Upper bound period of interest (s) (see 3.3)
$V_{s(30)}$	Time averaged shear velocity over 30 m depth from the ground surface (m/s) (see 3.2.3.1)
W_p	Weight of a part or component (see 5.2) (N)
γ	Record-to-record variability scaling factor (see 4.3.2.1)
ξ	Target elastic equivalent viscous damping ratio (see 4.2.5)
ϕ	Capacity reduction factor (see 4.3.2.1)
ψ_{CALS}	CALS return period scaling factor (see 2.5)
ψ_E	Earthquake imposed action combination factor (see 4.1)
Ω_P	Reserve capacity factor of a part or component (see 5.2)

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