PART C

C3 Earthquake Demands

Non-EPB Seismic Assessment Guidelines

ONLY TO BE USED FOR SEISMIC ASSESSMENTS OUTSIDE THE EPB METHODOLOGY

2025

New Zealand Geotechnical Society

nzsee
NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

SESOC

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

Natural Hazards Commission
Toka Tū Ake

Foreword

The Joint Committee for Seismic Assessment and Retrofit of Existing Buildings is responsible for the joint oversight of the system used to assess, communicate, manage and mitigate seismic risk in existing buildings. It reviews how the guidelines are functioning in practice, identifies areas that require further input and development, and either advises on or assists in the development of proposals for work programmes that contribute towards these objectives. The Joint Committee includes representatives from The Natural Hazards Commission Toka Tū Ake, the Ministry of Business, Innovation & Employment, and the technical societies (NZGS, NZSEE, SESOC).

The Joint Committee's Vision is that:

- Seismic retrofits are being undertaken when necessary to reduce our seismic risk over time while limiting unnecessary disruption, demolitions and carbon impacts, promoting continued use or re-use of buildings.
- Decisions on retrofitting are informed by an appropriate understanding of seismic risk and are aligned with longer term asset planning.
- Seismic assessment and retrofit guidelines help engineers focus on the most critical vulnerabilities in a building, serve the needs of the market and regulation, and evolve through a stable ongoing cycle allowing new knowledge and improvements to be included in a predictable manner, including the consideration of objectives beyond life safety.
- Engineers are supported in the implementation of Seismic Assessment and Retrofit Guidelines through a range of training and information sharing strategies, including tools for risk communication to manage unnecessary vacating of buildings.
- Society is informed about the level of risk posed by existing buildings.

Acknowledgements

Updates encompassed in this document were prepared by the Joint Committee in conjunction with a Technical Review Group formed for the *Concrete Buildings C5+* project.

Joint Committee

Nic Brooke	SESOC
Dave Brunsdon	SESOC
Caleb Dunne	NHC
Ken Elwood	MBIE/NHC
Rob Jury	NZSEE
Stuart Palmer	NZGS
Mark Ryburn	MBIE
Henry Tatham	NZSEE
Merrick Taylor	NZGS
Andy Thompson	NZSEE

Technical Review Group

Nic Brooke (Editor)
Carl Ashby
Des Bull
Rick Henry
Stuart Oliver
Umair Siddiqui
Craig Stevenson
Weng Yuen Kam

Version Record

Version	Date	Purpose/ Summary of changes
1	17 July 2017	Initial release
2A	17 March 2025	Proposed technical revision only for use for non-Earthquake Prone Building purposes.

This document is managed by the Joint Committee for Seismic Assessment and Retrofit of Existing Buildings. It may be downloaded from design.resilience.nz.

Refer to the following pages for a summary of the key changes from previous versions.

Please visit design.resilience.nz to provide feedback or to request further information about these Guidelines.

Disclaimer

This document is intended as a guideline only. This document is intended for use by trained practitioners under appropriate supervision and review. Practitioners must exercise professional skill and judgement in its application.

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This section is part of the *Non-EPB (Earthquake-Prone Building) Seismic Assessment Guidelines* which constitute a proposed technical revision to the 1 July 2017 EPB Seismic Assessment Guidelines. The *Non-EPB Seismic Assessment Guidelines* may be used for general commercial Detailed Seismic Assessments for non-EPB purposes. It is to be used in conjunction with Part A of the *EPB Seismic Assessment Guidelines*.

Engineers engaged to assess buildings identified by a territorial authority as being potentially earthquake prone in accordance with the *EPB Methodology* must continue to use *EPB Seismic Assessment Guidelines* (1 July 2017) as these are referenced in the Methodology.

Summary of Key Changes from Version 1

A limited revision of Section C3 has been undertaken. The motivation for this revision was to correct inconsistencies with updated versions of Section C5: Concrete Buildings. Errors identified in Version 1 have also been corrected, and updated references provided.

The main changes from the July 2017 version of Section C3 can be summarised as follows:

- Clarification of the vertical acceleration response spectra that should be used if necessary for assessments (C3.6)
- Clarification of application of the structural performance factor in NLSPA (C3.10.2)

Contents

C3. Earthquake Demands

C3.1 General

C3.1.1 Outline of this section

This section sets out the intended method for deriving the Ultimate Limit State (ULS) seismic demand, which is needed to evaluate the *%NBS* earthquake rating in accordance with Part A and Section C1. It also lists the available representations of the ULS seismic demand and explains what is intended for these.

C3.1.2 Definitions and acronyms

100%ULS seismic demand	Ultimate limit state seismic demand for new buildings used in the calculation of *%NBS*. Can be represented in a number of ways depending on the aspect under consideration.
ADRS	Acceleration-displacement response spectrum (spectra)
Importance level (IL)	Categorisation defined in the loadings standard, AS/NZS 1170.0:2002. This is used to define the ULS shaking for a new building based on the consequences of failure and is a critical aspect in determining new building standard.
PGA	Peak ground acceleration
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (push-over) relationship for the building as a whole
Site subsoil class	Categorisation of the soil profile under the building in accordance with NZS 1170.5:2004
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings

C3.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
g	Acceleration due to gravity
$K_{\delta}\ (T)$	Displacement spectral scaling factor. Varies depending on the building period, *T*.
k_{μ}	Inelastic spectrum scaling factor as defined in NZS 1170.5:2004
K_{ξ}	Spectral damping reduction factor (refer to Section C3.3)
m_{eff}	Effective mass of the equivalent SDOF system (refer to Section C2.4.2)
R	Return period factor. Will typically be R_{u} determined in accordance with NZS 1170.5:2004.
R_{u}	Return period factor appropriate for the ULS. Determined in accordance with NZS 1170.5:2004.

Symbol	Meaning
S_{a}	Spectral acceleration
S_{d}	Spectral displacement
S_{p}	Structural performance factor. Determined in accordance with NZS 1170.5:2004.
T	Period(s) of vibration for the building
T_{eff}	Effective period of vibration of the equivalent single degree of freedom representation of the building
V_{prob}	Probable shear capacity
W	Total weight of the structure
Δ_{cap}	Probable deflection capacity at the effective (equivalent) height
ξ_{sys}	Equivalent viscous damping of the system

C3.2 Method for Deriving ULS Seismic Demand

C3.2.1 General

The basis for the derivation of ULS seismic demand is the New Zealand earthquake loadings standard NZS 1170.5:2004 and Module 1 of the New Zealand Geotechnical Society and Ministry of Business, Innovation and Employment's Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016). These are assumed to define 100%ULS seismic demand or, in other words, the seismic demand that would be used to design a similar new building for the ULS at the time the assessment is undertaken.

Note:

ULS seismic demand for the purposes of defining an earthquake-prone building in accordance with these guidelines has been set in legislation as that which would have been obtained for the design of a new building from NZS 1170.5:2004 and Module 1 of the Earthquake Geotechnical Engineering Practice series dated March 2016. These documents define the seismic demand that was current at the time the legislation was enacted, which is the relevant basis for the ULS seismic demand used to calculate the earthquake-prone threshold adopted in these guidelines of 34%*NBS*.

The importance level (IL) used for the evaluation of the ULS seismic demand shall be derived from AS/NZS 1170.0:2002 based on the use/intended use of the building.

For the purposes of deriving the ULS seismic demand, the design life shall not be taken as less than 50 years unless a lower design life has been formally established with the relevant building consent authority/territorial authority.

Note:

An argument can be raised that life safety risks should not be affected by the chosen design life of the building. The rationale for this is that the life safety risk exists at any point in time (say, expressed as an annual risk) and is not affected by the total exposure period, whereas the exposure period is relevant when considering the potential economic losses (for example) over the life of the building.

While the concept of a design life less than 50 years is allowable under AS/NZS 1170.0:2002, this is on the assumption that the building will be removed when this period expires and that this intention will be noted on the building file held by the building consent authority/territorial authority. This should also apply if a building is assessed from a regulatory point of view or a consent for alteration (retrofit) is applied for. It is not intended that a chosen design life of less than 50 years is simply rolled over in perpetuity. In accordance with the intent of the New Zealand Building Code a 50 year exposure period (design life) is considered to represent an indefinite design life.

C3.2.2 Available representations

Representation of the ULS seismic demand will vary depending on the method of analysis and the particular aspect being assessed.

The range of available representations includes:

- acceleration response spectra
- displacement response spectra
- acceleration-displacement response spectra (ADRS)
- ground acceleration, velocity or displacement strong motion records
- peak ground acceleration (PGA), ground displacements, characteristic earthquakes, numbers of cycles for geotechnical considerations
- inter-storey drifts and total deformation between supports for elements supported on ledges, and
- applied accelerations and displacements on elements of the building.

When using time history analysis techniques it may be appropriate to determine the *%NBS* by scaling input motions. In these circumstances the scaling should only be applied to the ground accelerations and displacements and not to the duration of shaking, which should remain as appropriate for the ULS.

Likewise, when running traditional analysis for a target *%NBS* (say 34*%NBS* for a simple earthquake-prone check) it is only the response spectral ordinates that are scaled. The duration of shaking remains unchanged from that implied by the 100%ULS seismic demands.

Note:

While it is acknowledged that some engineers will be more familiar with the elastic based representations of NZS 1170.5:2004 and the allowance for ductility through application of an assumed global ductile capability, the thrust of these guidelines is to take account of the nonlinear deformation capability of the building directly using the displacement-based simple lateral mechanism analysis (SLaMA) approach and the ADRS representation of the seismic demand.

C3.3 Horizontal Acceleration Response Spectra

When a horizontal acceleration response spectrum is used to establish the ULS seismic demand, the spectrum shall be derived in accordance with NZS 1170.5:2004 Clauses 5.2.2.1 and 5.2.2.2 including an appropriate value for S_{p}, which may vary depending on the particular aspect being assessed (refer to Section C3.10.2).

When required, horizontal acceleration response spectra for different damping values may be obtained by multiplying the spectral ordinates of the 5% damped elastic spectrum determined as above (i.e. setting $k_{\mu} = 1$) by the spectral damping reduction factor, K_{ξ}:

$$K_{\xi} = [7/(2 + \xi_{\mathrm{sys}})]^{0.5} \qquad \text{...C3.1}$$

where:

ξ_{sys} = equivalent viscous damping of the system (refer to Appendix C2D for calculation of ξ_{sys}).

Note:

Priestley et al. (2007) provides some guidance on damping and the resulting reduction in spectral demand for seismic assessment. Equation C3.1 is presented as part of this guidance.

While Kong and Kowalsky (2016) have recently noted that the above equation appears to be quite reasonable for large magnitude events, studies such as those by Akkar et al. (2014) and Rezaeian et al. (2014) indicate that the actual damping-dependent spectral scaling factor should be a function of several factors including magnitude, epicentral distance (and depth) and period of vibration.

Pennucci et al. (2011), on the other hand, demonstrated that more representative inelastic (effective period) spectra for use with the displacement-based design/assessment approach could be obtained by scaling the displacement spectrum using ductility-dependent, as opposed to damping-dependent, spectral scaling factors. However, Pennucci et al. (2011) also point out that scaling factors should be a function of spectral shape and the results presented by Stafford et al. (2016) indicate that such inelastic spectra should again depend on magnitude and period.

For sites affected by near-field ground motions containing velocity pulses, Priestley et al. (2007) recommended changing the exponent within Equation C3.1 from 0.5 to 0.25 to account for the limited benefit of hysteretic energy dissipation characteristics on inelastic displacement demands induced by velocity pulse characterised near-field motions.

However, results presented in Sullivan et al. (2013) suggest that when the effective period of a structure is assessed to be less than the velocity pulse period for the site then no change is required to the scaling recommended for far-field motions. In contrast, when the velocity pulse period is equal to or larger than the pulse period, the inelastic displacement demands tend to be equal to the elastic spectral displacement demands (suggesting no benefit of hysteretic response).

Near-fault effects have traditionally been associated with larger magnitude earthquakes. However, Bradley (2015) indicated that near-fault effects were also discernible in the moderate magnitude Christchurch near-fault events.

NZS 1170.5:2004 currently adjusts the acceleration response hazard spectrum for near-field effects using the near-fault factor. This addresses the increased amplitude of the expected motion for larger magnitude earthquakes (also taking into account the directional nature on the expected frequency of occurrence) but does not otherwise address the effect of the reduction in the ability to dissipate energy, and therefore the reduced effect of the ability of nonlinear behaviour (ductility) to reduce a building's response.

It is clear that additional research is needed to determine how best to account for near-field effects in design and assessment and the extent to which this phenomenon needs to be allowed for. It might be expected that future revisions of NZS 1170.5:2004 will need to address this issue which may increase demand requirements. This could also lead to the need to reconsider the level of damping that might be available and the expected effect of this. However, in the interim, it is recommended that Equation C3.1 continues to be used for all sites.

C3.4 Horizontal Displacement Response Spectra

For displacement based methods, a displacement response spectrum is required. For the purposes of these guidelines it is considered appropriate to derive the 5% damped spectral displacement spectrum by multiplying the ordinates of the 5% damped elastic acceleration spectrum from Section C3.3 by the factor:

$$K_{\delta}(T) = g(T/2\pi)^2 \qquad \ldots C3.2$$

Displacement spectra for different damping values may be obtained by multiplying the 5% damped displacement spectrum by the factor K_{ξ}, calculated using Equation C3.1.

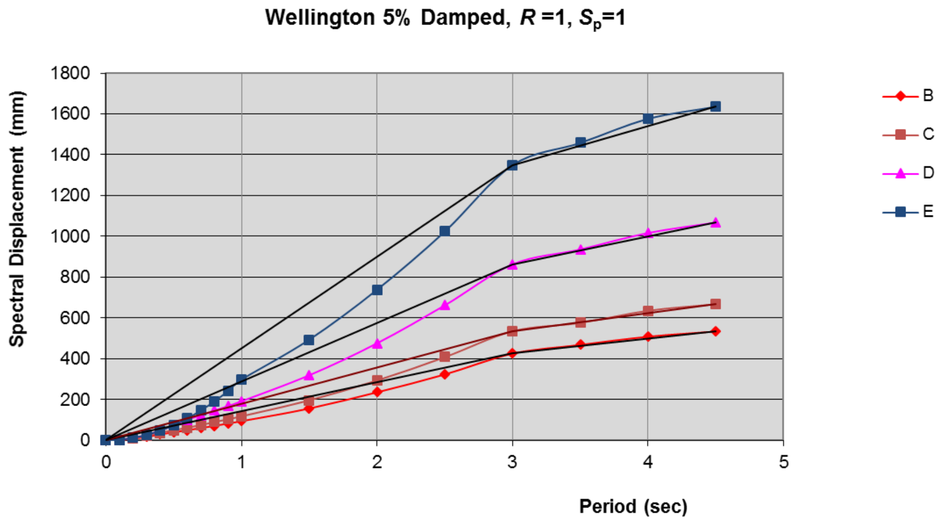
Figure C3.1 illustrates the shape of the resulting displacement spectra for Wellington, Christchurch and Auckland for different subsoil conditions. The effect of the application of K_{ξ} is illustrated in Figure C3.2. These figures show the spectra suitable for general purposes, i.e. not the bracketed values from Table 3.1 in NZS 1170.5:2004.

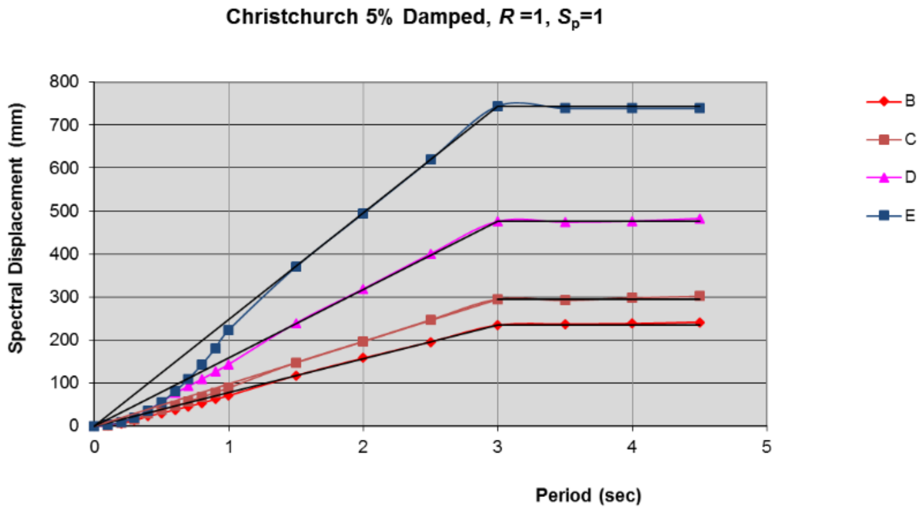
Examination of the displacement spectra in Figures C3.1 and C3.2 reveals several interesting points.

First, the significance of the soil type is much more apparent when seismicity is expressed in terms of displacement, rather than acceleration, spectra.

Second, apart from some nonlinearity for low periods, the curves are well represented by straight lines from the origin as shown on Figure C3.2. For sites where near-fault effects are not an issue the displacement spectra are well represented by a bilinear relationship pivoting around the displacement at T = 3 seconds and with a horizontal leg beyond 3 seconds. For a site where near-fault effects are specified the displacement spectra can be approximated by a bilinear relationship between T = 0, 3 and 4.5 seconds. These are approximations, the validity of which will need to be confirmed. It is expected that the straight-line approximations indicated are sufficiently accurate to be used as the basis for assessments and design of retrofit works. However, this should not preclude a more precise or direct evaluation should circumstances warrant or allow.

Third, the displacement spectra obtained do not represent the tendency of the spectral displacement to converge to the peak ground displacement at long periods but maintain the spectra conservatively at constant peak displacement response values (or increase these for sites where near-fault effects are specified).





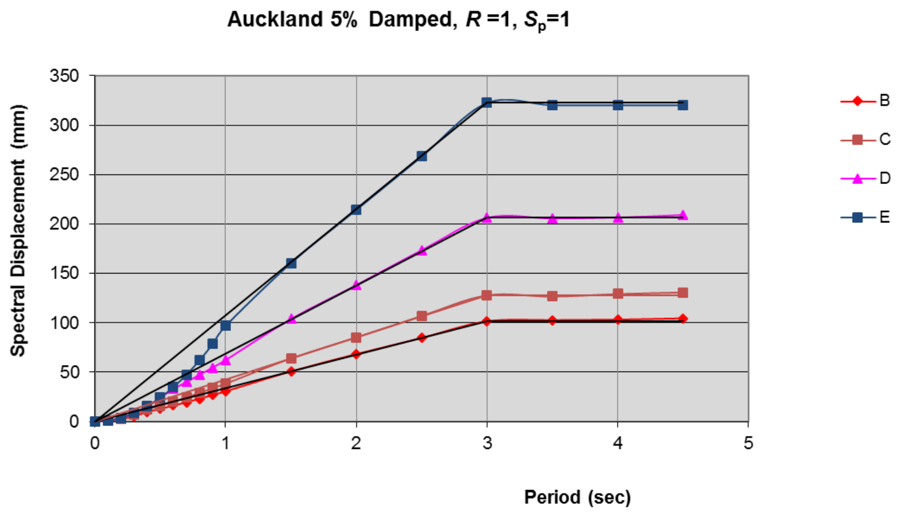
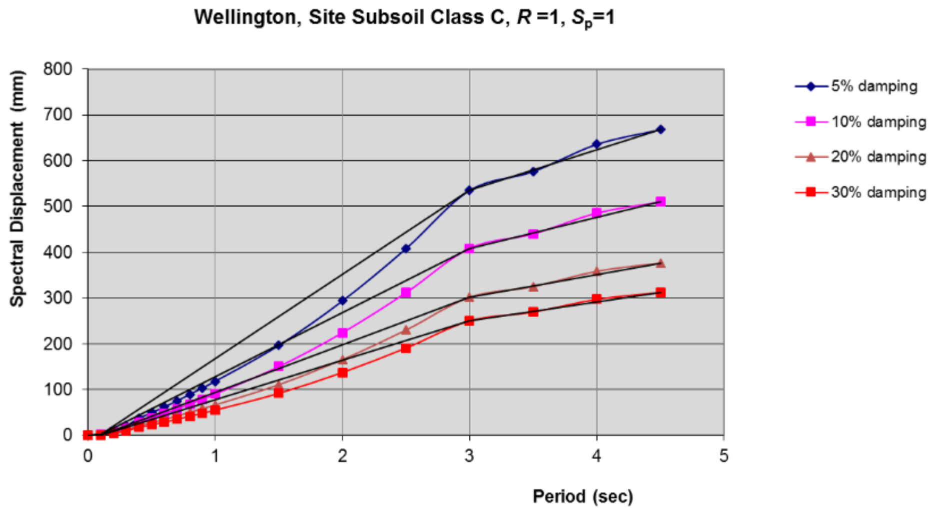
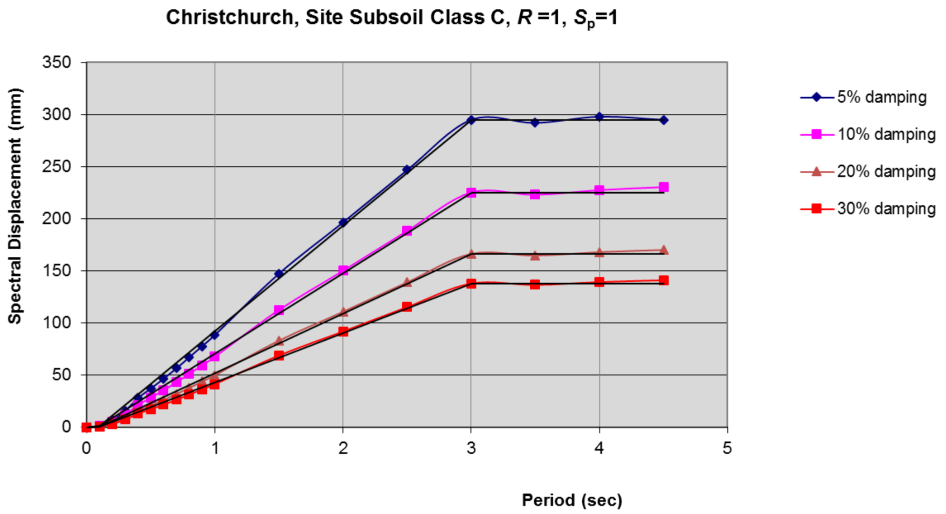


Figure C3.1: Displacement spectra at 5% damping for R = 1, S_p = 1 for various site subsoil classes and including appropriate near fault factor





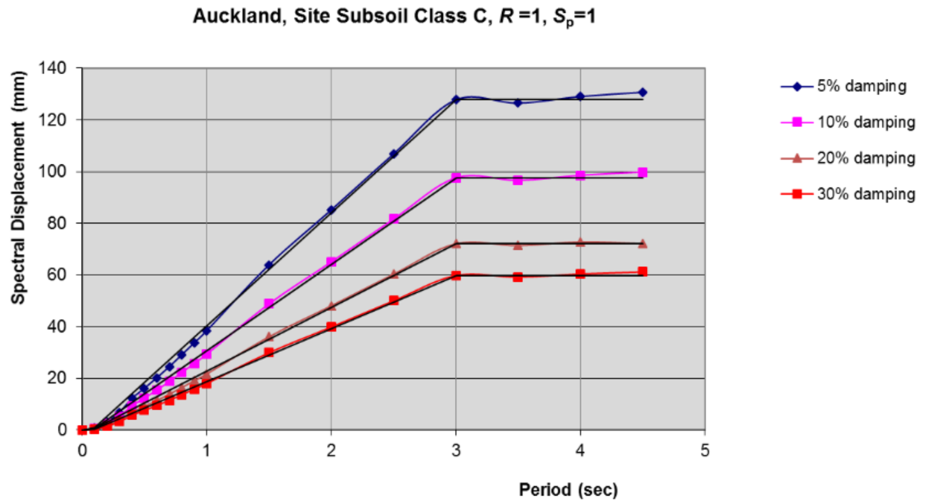


Figure C3.2: Displacement spectra for different damping levels and site subsoil class C and including appropriate near fault factor

C3.5 Horizontal Acceleration-Displacement Response Spectra (ADRS)

The acceleration and displacement spectra derived in the previous two sections for a particular site and level of damping can be usefully presented in the form of an acceleration-displacement response spectrum (Mahaney et al., 1993). The ordinates of such a spectrum are spectral acceleration and spectral displacement. An example of such representations is shown in Figure C3.3 for Wellington, Christchurch and Auckland for a 500 year return period (R_u = 1), S_p = 1 and site subsoil class C.

When constructing an acceleration-displacement spectrum for a particular level of damping both the acceleration and the displacement ordinates must be multiplied by K_ξ and the appropriate value of S_p.

Acceleration-displacement spectra are particularly useful when assessing the *%NBS* of a building from the results of a nonlinear pushover analysis. The acceleration and displacement results from a pushover analysis need to be converted to spectral acceleration and spectral displacement (as described below) before comparisons are possible with the acceleration-displacement spectra described above.

Note:

When a pushover curve has been derived from the combination of various structural systems of different ductile capability (using, for example, the SLaMA method), it may be more useful to incorporate the various S_p factors into the combined system pushover curve and compare against the ADRS calculated assuming S_p = 1 (refer to Section C3.10.2).

The conversion can be carried out as follows, assuming that elastic response is a good predictor of inelastic response and/or response in the first mode dominates (neither will always be the case):

$$S_a = V_{prob}/m_{eff}g \qquad \text{...C3.3}$$

$$S_d = \Delta_{cap} \qquad \text{...C3.4}$$

where:

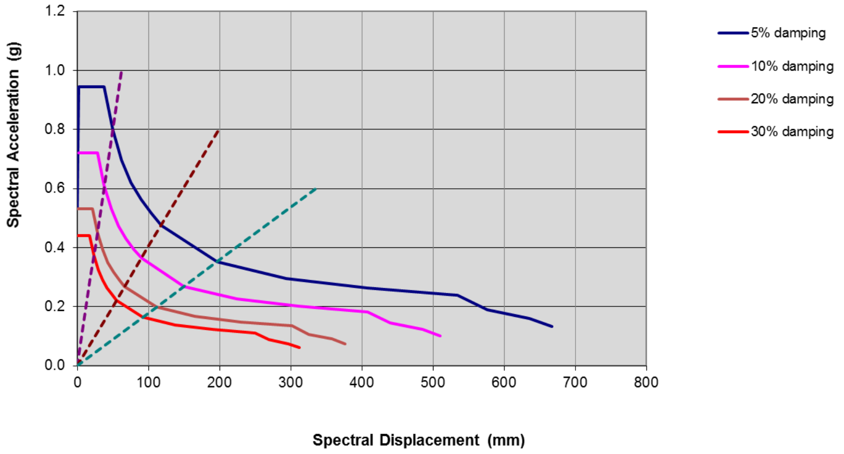
V_{prob} = probable base shear capacity consistent with Δ_{cap} (as calculated in Section C2)

m_{eff} = effective mass of the structure, calculated in accordance with Section C2.4.2.

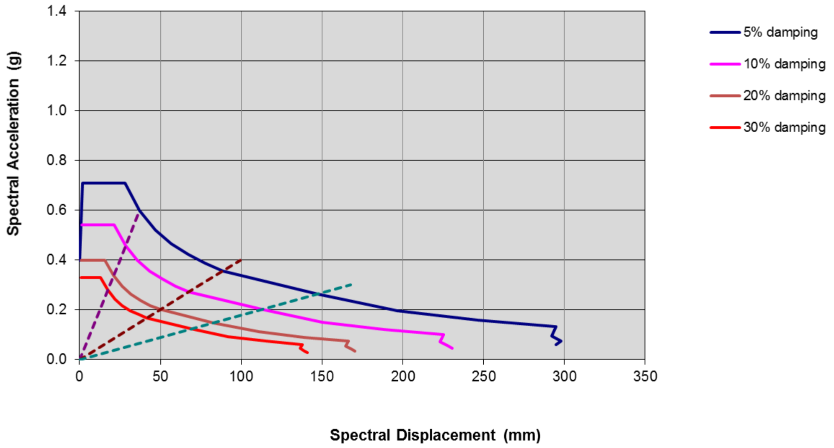
g = acceleration due to gravity.

Δ_{cap} = maximum lateral displacement capacity determined at the effective (equivalent) height (refer to Section C2).

Wellington, Site Subsoil Class C, R =1, S_p=1



Christchurch, Site Subsoil Class C, R =1, S_p=1



Auckland, Site Subsoil Class C, R =1, S_p=1

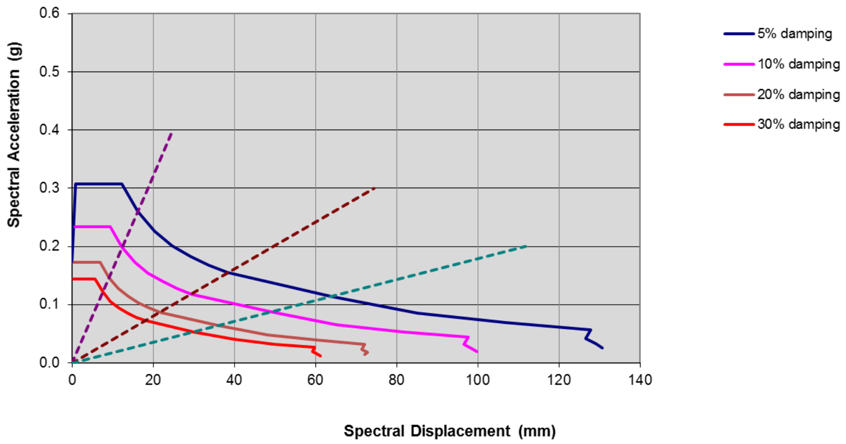


Figure C3.3: Acceleration-displacement spectra for different damping levels for R = 1, S_p = 1 and site subsoil class C

Note that the effective period, T_{eff}, of the equivalent single degree of freedom system can be approximated (assuming predominantly first mode response) from the relationship:

$$T_{eff} = 2\pi\sqrt{(S_d/S_a)} \qquad \text{...C3.5}$$

where:

S_a, S_d are as defined above.

Thus the stiffness of the building (T) can be represented by radiating lines from the origin of the acceleration-displacement spectrum. These lines, for example periods of 0.5, 1.0 and 1.5 seconds, are shown in Figure C3.3.

Note:

ATC 40 (1996) presents an excellent discussion on the way in which the acceleration-displacement spectrum can be derived and used to assess the performance of buildings. Refer to Section C2 for the use of ADRS with nonlinear static pushover analysis and in particular with SLaMA.

C3.6 Vertical Acceleration Response Spectra

When a vertical response spectrum is required to establish the ULS seismic demand, the spectrum shall be derived from NZS 1170.5:2004, Clause 5.4.

Note:

The vertical response spectrum found in the 2016 Amendment ot NZS 1170.5 differs significantly from that found in the 2004 edition. It is intended that the earlier version from NZS 1170.5:2004 be used because that was the version cited in Verification Method B1/VM1 on 1 July 2017 when the EPB methodology was enacted.

C3.7 Acceleration Ground Motion Records

When acceleration ground motion records are required, their selection and scaling shall meet the requirements of NZS 1170.5:2004, Clause 5.5. Alternative scaling procedures may also be employed provided their application is consistent with the intent of these guidelines.

The input earthquake records shall either contain at least 15 seconds of strong motion shaking or have a strong shaking duration of at least five times the fundamental period of the structure, whichever is greater.

All three components of any ground motion records should be scaled by the same factor which is determined separately for each direction of application of the principal component. The two horizontal components should be applied simultaneously. The vertical ground motion component should additionally be applied if it is expected to significantly affect the analysis outcome. When scaled ground motion records are used to establish a *%NBS* other than 100*%NBS*, only the acceleration ordinates should be scaled. The duration of shaking established for the ULS seismic demand should not be changed.

Note:

If the vertical ground motion component is applied, care should be taken to ensure that the analysis model used provides a realistic representation of the vertical dynamic characteristics of the structure. This will often not be the case for analysis models that have been developed following approaches that are commonly adopted when analysis is focussed on response to lateral actions.

Further guidance on inclusion of vertical ground motion in NLRHA can be found in SESOC et al. (2024).

C3.8 Demands on Elements Not Part of the Primary Lateral Structure

The ULS seismic demand on elements not part of the primary lateral structure should be determined in accordance with Section 8 of NZS 1170.5:2004. The demand may be in the form of applied loads/forces or deformations. Further guidance is provided in Sections C2 and C10.

C3.9 Representations for Geotechnical Considerations

The ULS seismic demand for geotechnical considerations, including PGA, representative (effective) earthquake magnitude and number of cycles, should be derived in accordance with the requirements of Module 1 of the Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016).

C3.10 Other Issues

C3.10.1 Site-specific probabilistic seismic hazard analysis

Site-specific probabilistic seismic hazard analyses should be completed in accordance with the requirements of NZS 1170.5:2004 and Module 1 of the Earthquake Geotechnical Engineering Practice series (NZGS/MBIE, 2016) as appropriate. The constraints noted in the Verification Method B1/VM1 (for New Zealand Building Code Clause B1 Structure) regarding the results from a site specific hazard analysis apply.

C3.10.2 Incorporation of the structural performance factor, S_p

The appropriate value of the structural performance factor, S_p, needs to be used when assessing the ULS seismic demand for structural considerations. This may require different values for S_p depending on the level of nonlinear deformation possible from the aspect under consideration, as determined in accordance with NZS 1170.5:2004 and this section.

For non-linear static pushover analysis (NLSPA) the value of S_p should be taken as the value specified by NZS 1170.5:2004 for equivalent static or modal response spectrum analysis.

Note:

Other approaches for including S_p in NLSPA (e.g. Marriott 2018) are inconsistent with the approach adopted in NZS 1170.5:2004 for the application of S_p and are therefore not appropriate for use in seismic assessments following these guidelines.

As S_p is dependent on the structural ductility available it is likely that this factor will only be able to be set once the available global ductility has been determined from the global deformation capacity of the building.

S_p is not used for geotechnical considerations.

C3.10.3 Application of ULS loading (actions)

The direction of application of the specified actions should meet the requirements of NZS 1170.5:2004, Clause 5.3. Allowances for accidental eccentricity should be in accordance with Section C2.5.7.

Where the actions for an element are influenced by more than one direction of loading (e.g. a corner column in a moment resisting frame building) and the load on the element cannot be limited by a yielding mechanism, the application of the ULS actions may be as for a nominally ductile structure.

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