PART C

Geotechnical Considerations C4

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 (JC-Sar) 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 a project team under the direction of the Joint Committee (JC-Sar). The project team comprised contributing authors plus a technical review group.

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Version Record

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

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 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 Changes from Version 1

Modifications have been made to Section C4 of Version 1 (July 2017) to enhance the clarity of the guidance and promote greater consistency in its application. These changes include describing when, why, and how allowances should be made for step changes in geotechnical behaviour. Additionally, revisions have been made to emphasize the importance of geotechnical input in the seismic assessment of existing buildings.

To conduct a reliable seismic assessment of an existing building, specialist earthquake foundation engineering input is expected, particularly at the early scoping stage of a project. The scope of this geotechnical input should be focused and limited to factors that could influence the structural assessment. Achieving this requires a collaborative effort between geotechnical and structural engineers. The updated Section C4 provides further guidance on this collaborative approach, with an emphasis on providing appropriate and focused geotechnical input. Structural and foundation engineers are encouraged to review this guidance and assess how they can improve their collaborative efforts.

The main changes from Version 1 July 2017 of Section C4 can be summarised as follows. Table 1 below provides further details of the changes.

- Restructured to improve readability
- New or substantially changed sections:
  - Key principles
  - Selecting geotechnical parameters
  - Guidance on undertaking a desk top study
- Consideration of SSI effects, including:
  - Initial assessment (A filter, does SSI matter?)
  - Guidance on more detailed assessment (If required)
  - Allowing for spatial variation of soils
  - Allowing for beyond peak resistance (sensitive soils)
  - Allowing for degradation of pile side resistance with cyclic loading.
- Identifying and allowing for geotechnical step change, including:
  - Why, when and how to allow for geotechnical step change
- Staged Reporting (To encourage a collaborative approach)
- Appendix of geotechnical step change worked examples
- Appendix of worked examples allowing for uncertainty

Impact on %NBS Assessment

While the changes were not intended to directly impact the assessed %NBS, the enhanced clarity may lead to less conservative interpretations of the guidance in certain cases, potentially resulting in higher assessed %NBS. The geotechnical step change factor has been reduced from 2 to 1.5, unless the potential consequences are extreme. As a result, the assessed %NBS could increase in some instances. These potential adjustments to the assessed %NBS only apply to geotechnically dominated assessments or certain interactive assessments—not all assessments.

Table 1: C4 Geotechnical Considerations - Change Register

Section number	Section title	Changes made relative to version 1 July 2017
All	**Contents**	The document has been restructured to improve readability. Key principles have been collated from throughout the document and brought forward to new section C4.2 Key Principles. The content of section C4.5 Key Principles, of the July 2017 document has been substantially re-written and included within the following new sections: C4.6 Consideration of SSI effects and C4.7 Identifying and allowing for geotechnical step change
C4.1	**General**	
C4.1.1	Scope and outline of this section	Minor changes to bullet points.
C4.1.2	Relevant publications	Reference to Modules updated and slope stability guidance added. Minor referencing updates to ASCE 2016 and 2017 docs, not referencing to latest due to changes in philosophy in some parts. Deleted text that state SSI damping is alternative to this guidance, since the note clarifies appropriate considerations.
C4.1.3	Definitions and acronyms	Updated definitions related to probable capacity (clarified no strength reduction factor applied), added geotechnical step change definition, updated XXX%ULS shaking (demand) to align with section C1, SLSH definition added.
C4.1.4	Notation, symbols and abbreviations	Minor updates. Deleted symbols not used in this document.
C4.2	**Key Principles**	**Key principles brought forward from elsewhere in the document for emphasis and clarity. Also includes updates and additions.**
C4.2.1	Consider geotechnical influences	Repeated and expanded from scope section for emphasis.
C4.2.2	Difference between assessment and design	Taken from C4.5.1 of the July 2017 document and expanded and clarified.
C4.2.3	Probable capacity for geotechnical issues	Taken from C4.5.2 of the July 2017 document and expanded and clarified.
C4.2.4	Selecting geotechnical parameters	A new section to clarify the level of conservatism to be applied in evaluating geotechnical outputs.
C4.2.5	Assess and report uncertainty	A new section to highlight the need to assess and report geotechnical uncertainty
C4.2.6	Geotechnical step change	A new section to introduce and define a geotechnical step change.
C4.2.7	Calculation and reporting of %NBS	Taken from C4.7 of the July 2017 document and clarified.
C4.2.8	Geohazards beyond the building footprint	Repeated for emphasis from C4.5.3.
C4.2.9	Close the loop	Added for emphasis of this important point.

Section number	Section title	Changes made relative to version 1 July 2017
C4.3	**Roles and Responsibilities**	
C4.3.1	General	No change
C4.3.2	Structural engineer's role	Modified sentences to explain the geotechnical input decisions should be jointly made between structural and geotechnical.
C4.3.3	Geotechnical engineer's role and required experience	Minor updates.
C4.3.4	Roles by project category	Minor updates
C4.4	**Assessment Process**	
C4.4.1	General	Minor addition of references back to section C1.
C4.4.2	Stage 1 – Project definition	Updated the text (mainly in the note box) to recommend some geotechnical input in Stage 1. This allows the geotechnical and structural engineer to collaborate in the initial stages to determine the key issues and allow the geotechnical engineer to either: 1. exit the process if limited geotechnical input is needed, or 2. become more involved if additional and detailed input is needed.
C4.4.3	Stage 2 – Assessment	Similar modifications to C4.4.2
	C4.4.3.1 Desktop study	Substantial modifications to text to provide more information to assist geotechnical practitioners in undertaking a desktop study.
	C4.4.3.2 Structural geotechnical meetings	Added further explanation of possible deliverables and outcomes. The objective is to encourage reporting to align with a collaborative approach.
	C4.4.3.3 Investigation, analysis and assessment iterations	Added further explanation of possible deliverables and outcomes. Taken from C4.5.1 of the July 2017 document and expanded and clarified.
C4.4.4	Stage 3 – Reporting and peer review	No change
C4.5	**Site Characterisation**	
C4.5.1	General	No change
C4.5.2	The ground model	Minor changes to text.
C4.5.3	Identifying geohazards	Minor changes to referencing
C4.5.4	Managing uncertainties	Minor change to isolated text.
C4.5.5	Site investigations	Moved detailed information related to desk study into Section 4.4.3.1.
C4.6	**Consideration of SSI Effects**	**This section has been substantially updated including:** **1. Order and structure modified for clarity and flow.** **2. Text from elsewhere in the document which relates to SSI has been shifted to this section as subsections.** **3. Previous text focussed on the vertical mode of SSI. Text modified to consider all 3 modes (Vertical, rotational and lateral).**

Section number	Section title	Changes made relative to version 1 July 2017
		4. Introduced point springs to represent foundation-soil, and separate specific soil-foundation analysis to evaluate these point springs. **5. Proposing an initial assessment to filter if SSI effects are influential.** **6. Outlining more detailed assessments.** **7. Guidance on special circumstances:** **a. Spatial variation of soils** **b. Modelling beyond peak resistance** **c. Degradation of pile side resistance with cyclic loading.**
C4.6.1	Key Principles	Key principles of SSI sub section added at beginning for emphasis and clarity.
C4.6.2	SSI Effects	This is a cut and paste from the July 2017 document of the section titled "C4.6 Consideration of SSI effects".
C4.6.3	Structural model and specific soil-foundation analyses	New section.
C4.6.4	Foundation load-displacement behaviour	New section.
C4.6.5	Initial assessment	New section.
C4.6.6	Detailed assessment	New section.
C4.6.7	Spatial variation of soils	New section.
C4.6.8	Model beyond peak resistance	New section.
C4.6.9	Degradation of pile side capacity with cyclic loading	New section.
C4.7	**Identifying and allowing for geotechnical step change**	**New section.**
C4.7.1	General	New section. Definition of geotechnical step change narrowed to only that triggered by increasing shaking. Change in behaviour with increasing shaking demand.
C4.7.2	Geotechnical step change assessment methodology	New section to provide clarity of why, when and how to allow for geotechnical step change. The step change factor has been reduced from 2 to 1.5, except for where consequences are extreme when 2 is to be applied.
C4.7.3	Geotechnical step change examples	New section.
C4.8	**Reporting and Peer Review**	
C4.8.1	General	No change.
C4.8.2	Level of geotechnical reporting	Clarified.
C4.8.3	Staged reporting and content	Staged reporting proposed to encourage a collaborative approach. Blue box added discouraging use of %NBS by geotechnical and explaining use of %ULS shaking.

Section number	Section title	Changes made relative to version 1 July 2017
C4.8.4	Peer review	Reference to JC-Sar guidance added.
	References and Bibliography	Updated.
Appendix C4A	Modelling of SSI Effects	Edited to improve clarity and detail, including: • Added reference to point springs. • Moved table of SSI modelling options from text to Appendix. Clarified that this table of options and comments only refers to comments on push-over analysis. • Collapsed linear Winkler, Winkler with no-tension, and Winkler with nonlinear springs into a single section on Winkler springs with specific subsections on no-tension and nonlinear springs. • Removed the section "damping approach" for nonlinear modelling since it is incorrect within the framework of the seismic assessment process. • Removed duplication of content related to the influence of SSI on assessment (which is covered in the C4.6).
Appendix C4B	Assessment of Retaining Walls	Information has been deleted that is included in the modules and only information more specific to assessment has been left in place. No additional information has been added.
Appendix C4C	Slope Instability Hazard	Information has been updated to reference NZGS slope stability guidance series and text reduced.
Appendix C4D	Seismic Performance of Foundations	Minor editorial changes made.
Appendix C4E	Liquefaction Assessment	Updated to include reference to MBIE and MFE guidance and the addition of comparison with historic earthquakes.
Appendix C4F	Influence of Shaking Levels on Ground Stability and Liquefaction Triggering	Applicability statement added.
Appendix C4G	Examples of applying geotechnical step change methodology	New Appendix.
Appendix C4H	Allowing for uncertainty in predicting soil deformation near strength capacity	New Appendix.

Contents

C4. Geotechnical Considerations

C4.1 General

C4.1.1 Scope and outline of this section

This section provides guidance on the geotechnical considerations for a Detailed Seismic Assessment (DSA). It provides tools to:

- identify the level of influence that ground behaviour (e.g. soil deformation or specific geotechnical hazards such as slope instability) may have on structural performance during earthquake shaking
- where possible, to quantify these effects and provide an appropriate level of input to the overall assessment

All DSAs are expected to include consideration of geotechnical influences on the building's structural behaviour, and will likely require some geotechnical input to the DSA process. Refer to Steps 1, 2 and 3, outlined in Figure C1.1 of Section C1. However, the level of geotechnical input will be a function of the detail required for the assessment and the likely sensitivity of the building's seismic behaviour to the geotechnical conditions.

The geotechnical assessment of earthquake performance of existing buildings requires a high degree of experience, competence, local knowledge and engineering judgement to properly:

- understand the scope of work required
- understand the likely vulnerabilities of the soil-structure system being assessed
- interpret and act on information acquired during the steps of the assessment process

The geotechnical assessment is to be led by a CPEng (Geotechnical) with appropriate experience and specific training in seismic assessment.

The approach outlined in these guidelines for including the consideration of geotechnical issues in the DSA represents a fundamental change from the traditional approach to considering these issues for new building design. Accordingly, a geotechnical engineer will need to carefully consider the material in this section to make sure this approach is understood.

The lead engineer (who will likely be a structural engineer) will also need to be familiar with this section as significant interaction between the geotechnical and structural engineer during a DSA is considered essential.

This section contains particular guidance on:

- timing and scope of input, including an outline of the respective roles of the geotechnical engineer and structural engineer depending on the nature of the project
- the approach to be taken for the inclusion of geotechnical issues
- development of an appropriate ground model
- identification and screening of common geotechnical hazards (geohazards) related to seismic activity that are relevant to life safety in structures and the manner in which geohazards from outside the site are dealt with in terms of influencing the earthquake rating for the building
- assessment of geotechnical aspects of foundation behaviour
- provision of input to soil-structure interaction (SSI) models and consideration of SSI in seismic assessment
- inputs to the calculation of %NBS (typically in a form relating to geotechnical influences on the assessment of the structure's probable capacity)
- reporting and peer review

Note:

The Canterbury earthquake sequence of 2010-11 triggered widespread liquefaction across much of Christchurch as well as rock slides, rockfalls and cliff collapse and other forms of slope instability in the Port Hills, affecting tens of thousands of buildings. About half of the NZ$40 billion total economic loss from these earthquakes (New Zealand Treasury, 2013) could be attributed to the geotechnical impacts caused by liquefaction and rock mass instability.

However, while seismic assessments may include economic considerations, it should be remembered that the assessment of a building's earthquake rating under these guidelines is focussed on those aspects, including geotechnical influences, which will potentially lead to a life safety issue for building occupants and the public outside the building, and damage to adjacent property.

The assessing engineer should be mindful of the differences between assessment and design. In design the focus is on life safety and serviceability, with the objective of providing a "reliable" solution. Assessment of existing buildings focusses primarily on life safety (damage to adjacent property also requires consideration), and has the objective of developing an understanding of the building's expected behaviour in seismic events. Key principles regarding the differing focus and levels of conservatism ("reliable" for design and "probable" for assessment) are set out in Section C4.2. In general terms, building assessment is not the same as design in reverse as they have different objectives and follow different approaches. This is particularly the case for consideration of geotechnical issues.

As the science and practice of geotechnical earthquake engineering continues to evolve it is intended that these guidelines and the joint New Zealand Geotechnical Society/Ministry of

Business Innovation and Employment modules (described in Section C4.1.2 below) will be updated periodically to incorporate new advances in the field. However, these updates will, naturally, lag behind the very latest advances. It is important that users of this document familiarise themselves with the latest advances and amend this guidance appropriately.

Note:

Additional material can also be found in the appendices to this section. This material is intended to supplement the material in the modules and provide information/discussion on issues that are particularly relevant to assessment rather than design, which is the primary focus of the modules.

A comprehensive bibliography and list of references is provided at the end of this section. Engineers are expected to be familiar with the relevant documents and to know what is important for the seismic assessment of existing buildings, particularly as this relates to life safety aspects.

C4.1.2 Relevant publications

C4.1.2.1 New Zealand geotechnical guidance

The New Zealand Geotechnical Society (NZGS) and the Ministry of Business, Innovation and Employment (MBIE) have jointly developed a series of modules for earthquake geotechnical engineering practice ("the NZGS/MBIE modules"). These modules have been published by MBIE as guidance under section 175 of the Building Act 2004 and are summarised in Table C4.1.

While the NZGS/MBIE modules relate primarily to new building design, many of the principles they contain are relevant to the seismic assessment of existing buildings. It is the intent that the requirements set out in these modules are used as the basis for assessment, with appropriate adjustments to reflect the differences between design and assessment outlined in these guidelines (Refer C4.2.2 e.g. strength reduction factors are applied in design but not in assessment).

Although Module 1 (2021) is the current version (at the time of writing) for determining the shaking demand for geotechnical design, Module 1 (2016) is to be used for determining shaking demand for geotechnical assessment of existing buildings in accordance with this guideline.

Note:

The information regarding the status of each NZGS/MBIE module was correct at July 2025. Please check at https://www.building.govt.nz/building-code-compliance/b-stability/b1-structure/geotechnical-guidance and www.nzgs.org for updates.

Table C4.1: Summary of joint NZGS/MBIE modules in the earthquake geotechnical engineering practice series

NZGS/MBIE module (publication date)	Description
Module 1 Overview of the guidelines (March 2016)	• Module 1 dated March 2016 is to be used for determining shaking demand for geotechnical assessment of existing buildings in accordance with this guideline • This module also provides an overview of the module series • Introduces the subject of geotechnical earthquake engineering, provides context within the building regulatory framework, and provides guidance for estimating ground motion parameters for geotechnical design • Includes guidance on a number of geohazards, including fault rupture
Module 2 Geotechnical investigations for earthquake engineering (November 2021)	• Guidance on planning geotechnical site investigations • Detailed description of various techniques available for sub-surface exploration; discussion of advantages and disadvantages of each • Describes that the primary objective is to understand the ground conditions for the project being undertaken
Module 3 Identification, assessment and mitigation of liquefaction hazards (November 2021)	• Introduces the subject of soil liquefaction; describes the various liquefaction phenomena including lateral spreading • Includes discussion on clay soils and volcanic soils
Module 4 Earthquake resistant foundation design (November 2021)	• Discusses foundation performance requirements during earthquakes in the context of New Zealand Building Code requirements • Describes the different types of foundations in common use and includes a strategy for selecting the most suitable type based on necessary site requirements for each Note: Module 4 is an important reference for the assessment of existing structures. However, not all load and resistance factor design (LRFD) requirements for new design are relevant to the assessment of existing buildings. See later in this section for more on this topic.
Module 5 Ground improvement of soils prone to liquefaction (November 2021)	• Considers the use of ground improvement techniques to mitigate the effects of liquefaction, cyclic softening, and lateral spreading at a site, including the effects of partial loss of soil strength through increase in pore water pressure during earthquake shaking • Guidance on assessing both the need for ground improvement and the extent of improvement required to achieve satisfactory performance for new design and for improvement of existing buildings

NZGS/MBIE module (publication date)	Description
Module 5a Specification of ground improvement for residential properties in the Canterbury region (November 2015)	• Guidance on what should be included in a technical specification when designing and constructing ground improvement for liquefaction mitigation purposes. Four ground improvement techniques are covered: densified crust, stabilised crust, stone columns, and driven timber piles. Note: Modules 5 and 5a: The application of ground improvement methods to enhance the safety of existing buildings may be limited, but important principles are covered in these modules that will lead to greater understanding of dynamic soil behaviour and effects on foundation performance.
Module 6 Earthquake resistant retaining wall design (November 2021)	• Seismic considerations for design of retaining walls Note: MBIE's Guidance on the seismic design of retaining structures for residential sites in Greater Christchurch (Nov 2014) is an existing source of information on retaining walls that is informative for existing structures.
Slope Stability Geotechnical Guidance Series – Unit 1, Unit 2, Unit 3, Unit 4, Unit 5, Unit 6, Unit 7A to 7C	Guidance for assessment of slope stability separated into 7 units: • Unit 1: Overarching Document (issued December 2024) • Unit 2: Landslide Recognition, Identification and Field Investigations (not currently available) • Unit 3: Slope Stability Analysis (not currently available) • Unit 4: Mitigation Strategies for Slope Stability (not currently available) • Unit 5: Rockfall Assessment, Analysis and Mitigation (not currently available) • Unit 6: Debris Flow Assessment, Analysis and Mitigation (not currently available) • Unit 7A to 7C: Special Cases and Materials (not currently available)

C4.1.2.2 US geotechnical guidance

ASCE 41-17 (2017) – Foundations and geologic site hazards

ASCE 41-17 (2017) Chapter 8 Foundations and Geologic Site Hazards provides useful additional information with respect to the assessment of existing buildings to supplement that provided in these guidelines and the NZGS/MBIE modules.

Chapter 8 of ASCE 41-17 (2017) presents general requirements for consideration of foundation load-displacement characteristics, seismic evaluation and retrofit of foundations, and mitigation of seismic geologic site hazards. It covers:

- definition of seismic geologic site hazards
- data collection for site characterisation
- procedures for mitigation of seismic geologic site hazards
- soil strength and stiffness parameters for consideration of foundation load-displacement characteristics
- procedures for consideration of SSI effects

- seismic earth pressures on building walls
- requirements for seismic retrofit of foundations

Note:

Care is necessary when applying guidelines from other jurisdictions to ensure that the overarching philosophies are consistent. For example, the New Zealand approach is heavily focused on life safety and uses probable capacities to determine how a building may rate against minimum Building Code (B1) requirements. (Refer C4.2)

Soil-structure interaction (SSI)

There are a number of relevant US references regarding the modelling of SSI effects for the design of new buildings (e.g. NIST GCR 12-917-21, 2012a; FEMA P-1050-1, 2015) and seismic evaluation of existing buildings (ASCE 41-17, 2017).

These documents provide a modelling approach and parameters for foundation flexibility, kinematic effects (i.e. base slab averaging and embedment effects) and foundation damping.

Note:

While the SSI modelling principles are generally applicable to the New Zealand context, the use of SSI to reduce the seismic demand using SSI damping and kinematic effects is not provided for in these guidelines although some aspects of SSI damping could be considered to be included in the NZS 1170.5:2004 structural performance factor, S_p, for the building as a whole. If engineers elect to reduce seismic demand using damping resulting from SSI and kinematic effects, S_p is likely to require amendment accordingly and care will be necessary to reflect the potentially higher level of uncertainty in such assessments.

C4.1.3 Definitions and acronyms

Note:

Definitions and terminology used in Section C4 of the guideline necessarily aligns with that in other sections of the guidelines. Consequently, some definitions and terminology may differ from that used in the MBIE/NZGS Modules.

CPT	Cone penetration test
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Deformation limit (δ_{Limit})	The maximum deformation (δ_{Limit}) of a foundation for which the load deformation behaviour model of that foundation can be relied on without geotechnical review.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines.
Foundation model	The springs that are used in the structural model to represent the load-deformation behaviour at the soil-foundation interface.
FE	Finite element (refer to Section C4A.3.4)
Geohazard	Geotechnical hazards
Geotechnically dominated	One of three defined project categories, in which the structure response is likely to be governed by geohazards and/or ground behaviour. Geotechnical step change is often a characteristic of the ground and foundation performance in a geotechnically dominated project.
Geotechnical step change	A **sudden** and **large** adverse change in geotechnical behaviour, with increasing shaking demand.
Interactive	One of three defined project categories, in which geohazards, soil nonlinearity and SSI may have an influence on the critical structural mechanism(s)
LRFD	Load and resistance factor design
MMI	Modified Mercalli Intensity
PGA	Peak ground acceleration
Probable capacity (of a foundation/soil)	Assumed probable capacity (i.e. resistance/strength) of a foundation/soil The probable capacity is typically taken as the ultimate geotechnical capacity (resistance/strength, R) that would be assumed for design. A strength reduction factor is not applied in assessment.

Project categories	Assessments are categorised as either structurally dominated, geotechnically dominated or interactive depending on the significance of potential geotechnical influences on the structure (refer to Section C1)
Resistance	Restraint that a foundation provides at a specific level of deformation or level of shaking. Resistance increases with deformation to the maximum value R. See "Probable capacity".
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
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 (pushover) relationship for the building as a whole
Serviceability limit state (SLS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
Significant life safety hazard (SLSH)	A hazard resulting from the loss of gravity load support of a member/element of the primary or secondary structure, or of the supporting ground, or of non-structural elements that would reasonably affect a number of people. When shelter under normally expected furniture is available and suitable, mitigation of the hazard below a significant status is assumed.
SPT	Standard penetration test
SSI	Soil- structure interaction
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS*. Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk
Structurally dominated	One of three defined project categories, in which the structural response is unlikely to be significantly influenced by geohazards, foundation soil nonlinearity or SSI
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016

C4.1.4 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as calculated by application of these guidelines
B	Width of foundation
c	Soil cohesion
R	Ultimate geotechnical capacity (strength). (Probable capacity i.e. moderately conservative)
$R_{\mathrm{d}} = \phi_{\mathrm{g}} R$	Reliable geotechnical resistance (strength capacity) used for design, where ϕ_{g} is the geotechnical strength reduction factor and R is as defined above
R_{R}	Probable (moderately conservative) residual resistance (strength capacity) after a change in geotechnical behaviour.
S_p	Structural performance factor associated with the detailing and assessed ductile capability of the system as a whole. Determined in accordance with NZS 1170.5:2004. Refer to Section C3.
δ_{Limit}	The maximum deformation (δ_{Limit}) of a foundation for which the load deformation behaviour model of that foundation can be relied on in structural analysis without review by the geotechnical engineer. (Refer C4.6.4 g))
ϕ	Strength reduction factor
ϕ_{g}	Geotechnical strength reduction factor
γ	Unit weight of the backfill

C4.2 Key Principles

Key principles of the assessment of existing buildings in accordance with these guidelines as they relate to geotechnical considerations are outlined in this subsection. These include the objectives of assessment and the differences between these and those for design, the use of probable capacities and the modelling of the resistance versus deformation behaviour for geotechnical issues. These aspects are discussed below.

C4.2.1 Consider geotechnical influences

All DSAs are expected to include consideration of geotechnical influences on the building's structural behaviour, and will likely require some geotechnical input to the DSA process. Refer to Steps 1, 2 and 3, outlined in Figure C1.1 of Section C1 and C4.4.2. However, the level of geotechnical input will be a function of the detail required for the assessment and the likely sensitivity of the building's seismic behaviour to the geotechnical conditions (assessments are categorised as either "structurally dominated", "interactive" or "geotechnically dominated" for this purpose, as outlined in Section C1 and C4.3.4).

A collaborative and iterative approach to the assessment is proposed as described in Section C4.4. This approach allows geotechnical engineering effort to be targeted on issues and information which can possibly be consequential for the particular structure and life safety. This approach limits the overall geotechnical effort required.

C4.2.2 Difference between assessment and design

C4.2.2.1 Life safety and the behaviour of the structure

Seismic assessment of existing buildings is primarily concerned with life safety. Therefore, it is necessary to understand the mechanisms that may lead to partial or full collapse of the structure, as it is generally the failure of the structure and/or its parts that will lead to casualties. Serviceability issues associated with the onset of general damage are not the focus.

It is not foundation/soil behaviour that determines the seismic rating of a building. It is the behaviour of the structure that determines the seismic rating including the response of the structure to the foundation behaviour.

C4.2.2.2 Earthquake shaking demand for geotechnical considerations

Although NZGS/MBIE Module 1 (2021) is the current version (at the time of writing) for determining the minimum shaking demand for geotechnical design, NZGS/MBIE Module 1 (2016) is to be used for determining shaking demand for geotechnical assessment of existing buildings in accordance with this guideline.

C4.2.2.3 Probable rather than reliable behaviour

For design, the aim is to set limits for geotechnical parameters for which there is a high reliability that support will be achieved without excessive deformation. This is typically a conservative approach, but in new building design this conservatism can be provided for, in most instances, with little cost premium. However, retrofit of foundations in an existing building is typically a disruptive, often difficult and expensive exercise and, as a result, it is not practical to simply adjust the foundation size to meet normal design criteria that are known to be conservative. Therefore, a realistic assessment of the foundation/soil behaviour and how this interacts with the structure becomes very important when establishing how well the foundations, as detailed, are likely to meet the assessment objectives. Refer to Section C4.2.3 and C4.2.4 for further discussion on the use of probable capacity in the context of the geotechnical assessment and the selection of suitable geotechnical parameters. C4.2.3 and C4.2.4 propose that for assessment the probable strength (capacity) be taken as a moderately conservative assessment of ultimate geotechnical capacity, as would be assumed in design (refer to NZGS/MBIE Module 4 - *Earthquake resistant foundation design*), but in assessment a strength reduction factor is not applied.

The intent of the seismic assessment is to establish holistically the probable capacity of the soil, foundation and structural system. This is also different to what may be used for design.

C4.2.2.4 Displacement focus

In design, load and resistance factored design (LRFD) is typically applied. Loads and resistances are factored to provide a low probability of yielding or failure of soil. This also is likely to control deformations. In an assessment this is typically replaced by a displacement-based approach. In this displacement-based approach the geotechnical engineer may need to assess the load displacement behaviour of the foundations/soil, including that at higher demands where nonlinear and yielding behaviour of the soil is expected. The structural engineer would apply this information in soil structure interaction (SSI) analyses and displacement-based assessment of the structure. Typically, large deformations in the soil can be tolerated before life safety in the building becomes an issue. The exception is in the situation where the building structure may not be well tied together. Guidance on assessing foundation/soil load displacement behaviour and applying this to SSI analysis is included in section C4.6

C4.2.2.5 Uncertainties

The uncertainties and unknowns associated with assessment are typically greater than they are in design. Often in assessment the dimensions of the existing foundations are uncertain, and rarely is subgrade verification test data from construction available. Obtaining subsurface information beneath or adjoining an existing building can be disruptive and expensive and therefore the assessment may need to be based on limited information. Section C4.5 provides guidance on investigations and dealing with uncertainties, this includes applying an iterative and collaborative approach, refer C4.2.2.6.

Because assessment does not apply strength reduction factors to capacity (refer C4.2.2.3) and because the focus of assessment is displacement based (refer C4.2.2.4), assessment may require the geotechnical engineer to predict the load-displacement behaviour of a foundation approaching and beyond its geotechnical capacity. Such predictions include very high level of uncertainty. Dynamic loading and possible liquefaction effects result in an even higher level of uncertainty. However as described in C4.6.4 and Appendix C4H, where soil/foundation deformation cannot be reliably predicted, appropriate assessment of a building is still likely to be possible by careful consideration of the following factors. These factors are likely to be more influential to the assessment than the absolute value of soil/foundation deformations and therefore details of the load-displacement behaviour may not be required.

- Ultimate geotechnical capacity
- Vulnerability of the structure to foundation displacement
- How displacement potential changes with increasing intensity of earthquake shaking

In reporting foundation load-displacement behaviour and other geotechnical parameters it is important to also report the uncertainty in these parameters so that this can be jointly considered by the geotechnical and structural engineers in undertaking the building assessment. (Refer 4.2.5).

Where limited information is available it is important that "consistent crudeness" is applied to the assessment, i.e. to avoid reporting analysis to a degree of accuracy that is inconsistent with the uncertainty of the input parameters. However, the assessing engineers need to satisfy themselves that level of information and analysis is appropriate to meet DSA process and that uncertainties in the assessment are reported.

Due to the inherent uncertainty in geotechnical engineering and, in particular, in geotechnical earthquake engineering, engineers need to draw on precedent, empiricism and well-founded engineering judgement to arrive at likely ranges of ground and foundation deformation.

Note:

The precedent referred to above is not a precedent in terms of "this is how we have always done it" (e.g. ignoring SSI) but in terms of observed behaviour (i.e. case studies with comparable earthquake demand, structural system, loads and ground conditions). In this regard, the experiences of the Canterbury earthquake sequence of 2010-11 (and other well-documented international earthquakes) can be of benefit to the assessment process.

C4.2.2.6 An iterative and collaborative approach

In design and assessment, a collaborative approach between the structural and geotechnical engineers is important, but particularly so for assessment.

Section C4.4 describes the assessment approach. This approach is necessarily iterative and collaborative to allow the assessment to focus on what could be consequential for the structure in terms of life safety, as described by the examples below:

- **Dealing with geotechnical uncertainty**

  Geotechnical data is often limited and expensive to obtain. As described in section C4.5.4 sensitivity analyses are proposed to test if the associated geotechnical parameters are consequential and therefore whether a specific investigation programme is warranted.

- **Is soil structure interaction consequential for a particular structure?**

  As described in section C4.6, before making detailed assessment of the load displacement behaviour for a foundation/soil it is proposed that the structure be initially assessed assuming the two extremes of pinned base with relatively soft vertical stiffness and fixed base. This may indicate that more detailed assessment of foundation load/ deformation behaviour is not warranted.

Sensitivity checks on the assumptions made will be an essential part of most seismic assessments. Depending on the sensitivity on the structural performance these checks might include the consideration of both upper and lower range soil strength/stiffness, the effect of different analysis methods, and soil behavioural models and their uncertainties. Depending on the outcome of these sensitivity checks a further iteration of more detailed assessment may or may not be warranted.

C4.2.3 Probable capacity for geotechnical issues

These guidelines are based on assessing the structural capacity of the building at a probable level. “Probable” for structure is considered as being at the expected or mean level. It is typically evaluated by using the determined/estimated mean (structural) material properties and setting the capacity reduction factors, applied for the purposes of design, to 1.

The concept of mean soil properties presents some difficulties in the geotechnical field. It may not be possible or appropriate to work with mean soil properties, for example, given the uncertainty and variance possible. At the same time, undue conservatism and the level of reliable behaviour aimed for in design, particularly around deformation capacity, is likely to be inappropriate for seismic assessment.

To recognise this situation the following approach has been adopted in these guidelines for assessing the probable capacity/resistance and deformation for geotechnical issues.

The probable strength (capacity) is taken as the ultimate geotechnical capacity as would be assumed in design (refer to NZGS/MBIE Module 4 - Earthquake resistant foundation design). In assessment a strength reduction factor is not applied, and the resistance deformation behaviour is assessed and modelled. Section C4.2.4 provides guidance on selecting geotechnical parameters and modelling geotechnical behaviour.

C4.2.4 Selecting geotechnical parameters

Geotechnical parameters routinely selected and reported as part of assessment include the capacity of a foundation, foundation load-displacement behaviour, trigger for liquefaction or slope instability, and magnitude of ground or foundation deformation. As outlined in section C4.2.3 these parameters are to be "*as would be assumed in design*" and "*a strength reduction factor is not applied*".

Geotechnical parameters used in design could be described as "moderately conservative". Expectation for these "moderately conservative" parameters is that there is a 5% to 30% probability that the actual value will be more adverse (Probability of exceedance 5 to 30%, typical expectation of 15%). These values of probability of exceedance are suggested to indicate an expected level of conservatism. Typically, insufficient information is available to allow a statistical assessment and undertaking a statistical assessment is not the expectation. The available information and engineering judgement is typically applied in selecting geotechnical parameters. "Moderately conservative" parameters are also to be selected for assessment.

In design and assessment, the evaluation of each output geotechnical parameter reported to the structural engineer (e.g. capacity of a foundation) will require several input parameters. These inputs may include:

- Foundation geometry
- Soil model / spatial variability
- Soil strength and stiffness
- Analysis methods

Each of these inputs will have associated uncertainty that is normally higher for assessment than it is in design. In assessment the geometry (dimensions) of a foundation is often not confirmed and geotechnical data may be limited because of the difficulty and cost of obtaining it beneath or adjoining an existing building. This uncertainty needs to be considered and allowed for in selecting the "moderately conservative" parameters.

Care needs to be taken in design or assessment not to put conservatism on conservatism by selecting a conservative value for each input. Rather, the evaluation should include sensitivity assessment of each input and judgement leading to selection of an output parameter which reflects the overall uncertainty. The aim should be to derive an output that is on whole "moderately conservative".

For example, inputs to the bearing capacity of a foundation include: foundation geometry, soil and groundwater model, soil strength and the analysis method. Making a conservative assumption for each of these inputs would be expected to result in an overly conservative bearing capacity estimation. Sensitivity analysis and judgement should be applied in evaluating the parameter.

Some parameters e.g. foundation stiffness should be reported as a range, recognising that in some cases a stiffer response may be more onerous. That range should be from a moderately conservative estimate of how stiff it could possibly be to a moderately conservative estimate of how soft it could be, i.e. an expectation that there is a 15% probability that the actual stiffness could be stiffer than the upper end of the range and a 15% probability that the actual stiffness could be softer than the soft end of the range. This estimate of the stiffness range should take into consideration soil variability and the range of loading (demand). NB: The 15% probability is included here to give an indication of expected level of conservatism. A statistical assessment is not expected.

A common approach is to make a best estimate of the stiffness and report the range as from a half to double that value. In some situations the range could be greater than indicated by half to double. It is recommended that the geotechnical engineer makes a specific assessment of the moderately conservative possible range of stiffness and report accordingly. Section C4.6 provides guidance on evaluating load-displacement behaviour of foundations/soil including the range/bounds to be considered for assessment and how these can be modelled in structural analysis.

In summary it is proposed that the geotechnical parameters reported to the structural engineer for assessment should be "moderately conservative", with due consideration of the uncertainty of each input and how the parameter is applied by the structural engineer in their sensitivity checks.

Note:

Moderately conservative parameters in section C4 are parameters that would be reported to a structural engineer e.g. the bearing capacity of a shallow foundation. The overall assessment of the bearing capacity is to be moderately conservative.

C4.2.5 Assess and report uncertainty

There can be high uncertainty in reported geotechnical outputs provided as inputs to the structural assessment. Situations where uncertainties can be very high (possibly a factor of 5 or more) include:

- Load-deformation behaviour of foundations where the demand exceeds 70% of the foundation's ultimate geotechnical capacity
- Load deformation behaviour of foundations subject to liquefaction effects
- The magnitude of cyclic displacement or lateral spread
- The magnitude of slope or retaining wall displacements due to earthquake shaking

In addition to reporting the assessed moderately conservative value of any geotechnical output, the expected range of that parameter should also be reported. The structural engineer should consider this range in the assessment and report back to the geotechnical engineer the significance of that range to the structural assessment (refer C4.2.9). In some situations, uncertainty in a geotechnical parameter and the vulnerability of the structure to that geotechnical parameter may drive strengthening of the building to reduce vulnerability of the structure, e.g. improve tying of the foundations together.

C4.2.6 Geotechnical step change

With increasing intensity of shaking demand soils can exhibit a "geotechnical step change" in behaviour. Examples of "geotechnical step change" behaviour include:

- Reduced support to a foundation due to liquefaction
- Ground displacement due to liquefaction and lateral spread or cyclic displacement
- Heave of basements due to liquefaction
- Ground displacement due to slope movement triggered by earthquake shaking

A key principle of assessment is to identify these potential geotechnical step changes and where they could lead to a significant life safety hazard (SLSH) in the structure, make appropriate allowance for them. Section C4.7 provides guidance on identifying and allowing for geotechnical step change in the assessment of existing buildings.

C4.2.7 Calculation and reporting of %NBS

The basis for the earthquake rating for the structure is %NBS, which is the ratio of the ultimate (probable) capacity of the structure's lowest scoring mechanism leading to a significant life safety hazard or damage to neighbouring property, to the actions expected when the structure is subjected to the demands resulting from the ULS defined loads/deformations for new buildings (refer to Part A and Section C1).

It is not foundation/soil/ground behaviour that determines the seismic rating (%NBS) of a building. It is the behaviour of the structure that determines the seismic rating including the response of the structure to the foundation behaviour. For this reason, the %NBS is calculated and reported by structural engineer and not the geotechnical engineer. The geotechnical engineer provides information on behaviour of the ground/soil which the structural engineer applies in assessing the structure and its %NBS.

Soil/ground behaviour can be modified by the intensity of earthquake shaking, for example triggering of liquefaction or slope instability. This intensity of earthquake shaking can be reported in terms of a XX%ULS shaking demand. 100%ULS shaking for geotechnical issues is the PGA and Magnitude as determined in accordance with NZGS/MBIE Module 1 (2016). Lesser intensity can be expressed as a percentage of this 100%ULS PGA for the same Magnitude. (Refer Section C4.1.3).

Where the mechanism for a structural element/member exceeds the structure's probable capacity and is directly as a consequence of a change in the behaviour of the soil/ground with %ULS shaking, the %NBS score is calculated as a ratio of the PGA triggering the change resulting in exceedance of the structures probable capacity (capacity) to the 100%ULS shaking demand.

It is the %ULS shaking resulting in the change in soil/ground behaviour leading to exceedance of the structure's probable capacity and a significant life safety hazard (SLSH), which is applied in calculating the %NBS score for that issue. For example:

- The %ULS shaking triggering the SLSH could be greater than that triggering liquefaction where the mechanism of concern is lateral spread i.e. a higher level of shaking may be required to cause sufficient lateral spread to exceed the structures probable capacity leading to a SLSH. It is this higher level of shaking that is applied in calculating the %NBS.

Where a geotechnical step change is identified which leads to a significant life safety hazard modification to the calculated %NBS score may be required as described in section C4.7.

Note:

%NBS score or rating is a measure of the assessed performance of the structure which will be reported by the structural engineer. Geotechnical reporting should not include %NBS. Where reporting of geotechnical capacity in terms of resistance to shaking is required (e.g. triggering of liquefaction or slope movement) this is to be reported in terms of "%ULS shaking"

C4.2.8 Geohazards beyond the building footprint

As outlined in Part A and Section C1 the earthquake rating is not intended to cover issues that arise from outside the site. This includes the effect of adjacent buildings and geohazards. Therefore, while aspects such as fault movement away from the site, slope failure onto a building, rockfall from above, and tsunami are important to note (where known) from a holistic hazard point of view, they should not be included in the assessment of the earthquake rating for the building. This is similar to the approach taken when rating a building when the neighbouring buildings could present a hazard to the building being assessed.

Land instability causing loss of support or deformation of the building is to be included in assessing the earthquake rating – lateral spread, slope instability, and instability of retaining walls affecting the support of the building.

C4.2.9 Close the loop

On the completion of each iteration of structural analysis based on any geotechnical information (models, parameters) it is important for the structural and geotechnical engineers to discuss the conclusions. This importantly includes the final conclusions of the assessment. This offers the opportunity to check that the information has been applied as intended and if the information proves to be critical to the assessment to allow the information and level of conservatism associated with it to be reviewed. This essential part of collaboration equally applies to design as it does to assessment.

C4.3 Roles and Responsibilities

C4.3.1 General

The roles and responsibilities for structural and geotechnical engineers are outlined in the following sections, together with suggestions on the suitable level of experience for geotechnical engineers involved in DSAs. This is followed by a summary of the roles and responsibilities that can be considered to apply based on the project categorisation, i.e. considering the potential impact of the geotechnical hazards on the building structure behaviour.

The effective assessment of structures starts with effective communication between the client/owner/tenant, the structural engineer and the geotechnical engineer (Oliver et al., 2013). A collaborative approach between all parties is essential so that the scope of work undertaken and the final assessment is appropriate for its intended purpose.

A common understanding of the expectations, roles and requirements of each team member at the outset of an assessment is important. Developing an appropriate brief that recognises the potential impact of geotechnical issues will likely require collaboration between the geotechnical engineer and the structural engineer and is an important step in the assessment process (refer to Section C1, DSA process Step 1).

While in some cases the geotechnical input to an assessment may be limited (i.e. structurally dominated), in many instances the ground and its interactions with the structure at increasing levels of shaking intensity can be complex and nonlinear. In these situations, specialist geotechnical advice and close collaboration between the structural and geotechnical engineer during the entire assessment process will be required (i.e. geotechnically dominated or interactive). Some projects may also warrant special studies, e.g. a site-specific seismic hazard assessment and/or site response, which will require specialist input.

The early decisions regarding the potential impact of geotechnical issues and the complexity of the geotechnical assessment that is warranted to address these will be under the influence of the lead engineer, who will more than likely be a structural engineer. If there is any question regarding whether ground conditions may influence the behaviour of the structure, the lead engineer should seek geotechnical advice, at least as part of formulating the scope of the assessment. This is important as there are a number of geohazards that can have a significant effect on a building's performance but may not be readily apparent to a non-geotechnical engineer.

Note:

All structural assessments are expected to include some consideration of the influences the ground behaviour and foundation systems can have on structural performance. Hence, geotechnical considerations are integral to the DSA process and in particular Steps 1 to 3 (refer to Section C1). Depending on the ground conditions, foundation types and the level of detail of the assessment, the geotechnical input to an assessment may vary significantly.

As this will potentially influence the project briefing, the assessing engineer liaising with the client at the outset should be experienced and aware of the range of interaction that may be required between the structural and geotechnical engineering disciplines.

C4.3.2 Structural engineer's role

The structural engineer:

- is typically the lead consultant for the assessment
- will assess if specialist geotechnical input is required (in most instances in consultation with a geotechnical engineer). (Refer C4.4.2)
- is responsible for liaison and reporting between the assessment team (structural and geotechnical) and the client. This should include involving the geotechnical engineer with client meetings when appropriate. For example:
  – at briefing meetings so the geotechnical engineer can hear and understand the client's needs and drivers
  – at other meetings so the geotechnical engineer can present conclusions, describe uncertainties, respond to questions on geotechnical aspects, and allow for the structural-geotechnical interaction required
- works collaboratively with the geotechnical engineer
- identifies structural forms and details which could potentially make the structure sensitive to soil and/or foundation performance

Note:

At the outset of a project it is important that the structural engineer is aware of potential geotechnical influences and makes the client aware of the potential need for, and value of, the input of a geotechnical engineer at various stages of the project.

C4.3.3 Geotechnical engineer's role and required experience

The geotechnical engineer:

- provides advice relating to geohazards, soils and SSI effects, as they relate to foundation behaviour
- provides advice relating to geotechnical uncertainties
- recognises when the project would benefit from the geotechnical engineer's involvement with client communication (meetings) and discusses this with the structural engineer if so, and
- works collaboratively with the structural engineer

The level of advice and judgement that will often be necessary in this role requires knowledge of:

- local ground conditions and geohazards
- the earthquake behaviour of soil and rock
- the interactions and behaviour of building/foundation/soil systems and how these may influence the performance of structures in earthquakes
- soil-spring characterisation

The advising CPEng geotechnical engineer must have relevant experience in geotechnical foundation and earthquake engineering (refer also to the NZGS/MBIE modules) and must have completed training in the assessment of existing buildings in accordance with these guidelines so there is confidence that the underlying principles and approach to assessment taken in these guidelines are understood.

Alternatively, the work may be undertaken by a geotechnical engineer with guidance and appropriate review from a CPEng geotechnical engineer with the experience and training described above.

C4.3.4 Roles by project category

C4.3.4.1 General

Either in Step 2 (desk study) or Step 3 (site inspection and investigations) of the DSA process (refer to Section C1 for process steps and additional details in C4.4) it is expected that the significance of geotechnical influences will be understood such that project can be categorised as either structurally dominated, interactive or geotechnically dominated as indicated in Figure C4.1 (refer to Section C1 for a description of the project categories, examples, and the process).

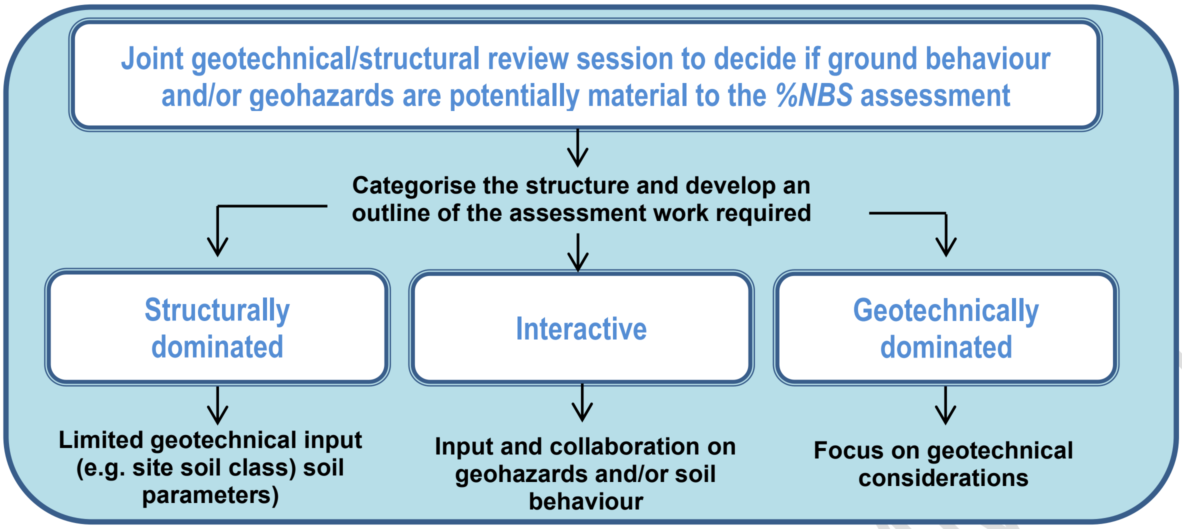


Figure C4.1: Project categorisation to reflect potential impact on the assessment of geotechnical issues

The guidance given below conveys the expected differences in scope for each project category. Specific project requirements will be determined at the outset and may vary as the project progresses.

C4.3.4.2 Structurally dominated

For structurally dominated projects, the structural/geotechnical collaboration should be sufficient to convey the general characteristics of the ground model and to develop an understanding and agreement that the probable range of geotechnical parameters are unlikely to significantly influence the behaviour of the structure.

The geotechnical parameters to be provided include:

- site seismic subsoil class
- near fault (as defined in NZS 1170.5:2004) assessment, and
- probable resistance available/strength (capacity) and possibly soil foundation stiffness (reported as a range).

The structural analysis is to include:

- sensitivity analysis across the range of parameters provided. To be "structurally dominated" it will be necessary to conclude that the structural analysis is not likely to be sensitive to the choice of parameters across this range and demand on foundation soils does not exceed their capacity
- a feedback loop to the geotechnical engineer, i.e. discussion of the results and conclusions of the analysis with the aim of verifying that geotechnical parameters have been interpreted and applied as intended and expected

C4.3.4.3 Interactive

An interactive project is when the geotechnical behaviour modifies the structural behaviour. Interactive projects generally require substantially more detailed geotechnical input. Significant interaction is expected between the geotechnical and structural engineering disciplines.

A staged approach should be employed, with structural/geotechnical collaboration and re-evaluation on completion of each stage to check that:

- geotechnical parameters have been applied as intended, with results as expected
- investigation and analysis is targeted and appropriate for specific building vulnerabilities

C4.3.4.4 Geotechnically dominated

For geotechnically dominated projects geotechnical behaviour triggers structural issues limiting the assessed %NBS rating. To be geotechnically dominated the lowest scoring element/member/issue has this score as a consequence of a geotechnical issue. It is the consequences of the geotechnical behaviour for the structure that will be critical to the assessment, e.g. triggering of liquefaction and lateral spread only determines the %NBS if as a consequence of that liquefaction and lateral spread the structures probable capacity is exceeded and this leads to a significant life safety hazard (SLSH).

Collaborative work between geotechnical and structural will be required to assess the ability of the structure to tolerate the geotechnical behaviour. The geotechnical engineer will describe scenarios of geotechnical behaviour and the structural engineer in collaboration with the geotechnical engineer will assess the consequences of that geotechnical behaviour for the structure.

A staged approach can be employed, with re-evaluation on completion of each stage so that investigation is targeted at valid vulnerabilities and gaps in knowledge, as appropriate.

C4.4 Assessment Process

C4.4.1 General

As the seismic assessment of a building should consider the interaction of the soil, foundation and structure, this requires collaboration between the geotechnical and the structural disciplines (as outlined in the previous section).

A collaborative and iterative approach is proposed here. This approach allows geotechnical engineering effort to be targeted on issues and information which can possibly be consequential for the particular structure and life safety.

Figure C4.2 illustrates the three key stages in this process:

- Stage 1 – project definition (aligned with Step 1 in Section C1)
- Stage 2 – assessment (including the geotechnical desktop study and geotechnical analysis and assessment, (this is aligned with Step 2 to Step 11 in Section C1)
- Stage 3 – reporting within the DSA.

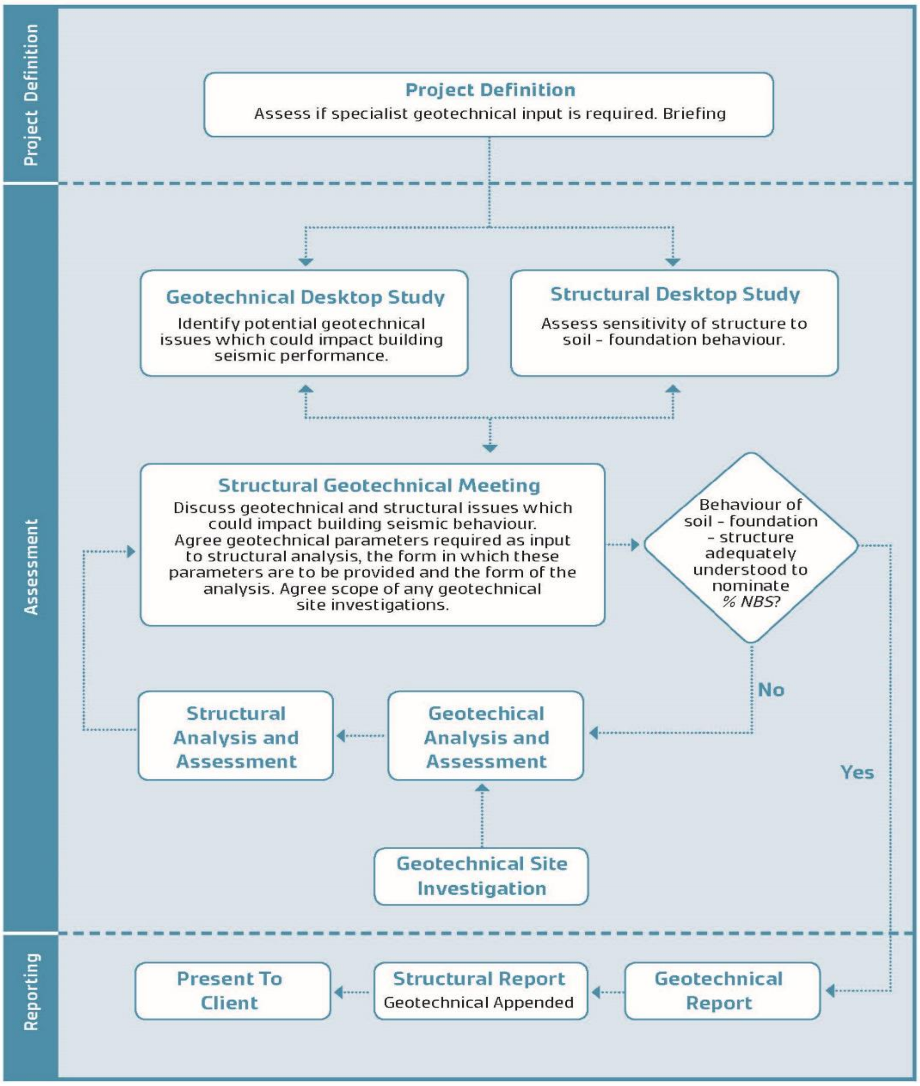


Figure C4.2: Project definition, assessment and reporting stages

These stages are outlined below and discussed in more detail in later sections.

C4.4.2 Stage 1 – Project definition

This first stage of the process outlined in Figure C4.2 is the initial review by the structural engineer in collaboration with the geotechnical engineer, to assess whether specialist geotechnical input is required and the likely scope of that work. From a geotechnical perspective it may identify the key geotechnical conditions that may be influential to the assessment. From a structural perspective it may identify key features of the structure which may make the structure sensitive or tolerant to these geotechnical conditions. These geotechnical and structural conditions should be discussed and the scope of the next stage of geotechnical work jointly agreed.

This stage involves:

- preliminary review of historic drawings and building records
- consideration of the ISA report, where available
- local knowledge of the site, ground conditions and groundwater regime
- judgement/experience
- the client's requirements
- broad and initial consideration of potential geohazards and SSI effects, reliability of soil-foundation support and associated uncertainties in the ground model, and the level of sensitivity of the structure to the soil-foundation behaviour

Examples of Possible deliverable

Examples of possible outcomes and deliverables from this stage include:

Example 1:

Circumstance:

The geotechnical and structural engineers agree limited geotechnical input is required and there are no geotechnical hazards that will influence the outcome of the assessment. The building is expected to be structurally dominated. The geotechnical engineer may conclude involvement in the project at this stage.

Possible deliverable:

A letter from the geotechnical engineer advising the site's seismic subsoil class and that the geotechnical engineer and structural engineer have jointly agreed that further specialist geotechnical input to the assessment is not warranted. The geotechnical and structural conditions which led to this conclusion should be listed along with the sources of this information.

Example 2:

Circumstance:

It is unclear what geotechnical hazards exist and whether they influence the outcome of the assessment, or geotechnical hazards may exist which could influence the assessment outcome. In this scenario it would be appropriate to advance assessment to Stage 2 and complete a more detailed desktop study.

Note:

The decision of what if any geotechnical input is required necessarily needs to be a joint decision between the structural and geotechnical engineers because that decision is influenced by the behaviour of the ground and the structure's ability to tolerate that behaviour.

Situations where no specialist geotechnical input may be required are where geohazards are absent or are not potentially influential or governing for structural life safety, soil-foundation (SSI) behaviour is well understood and is reliable, and the assessment is expected to be "structurally dominated". However, in this scenario it is expected that some degree of specialist geotechnical input will be required to confirm that geotechnical issues are not influential. The scope of work for the geotechnical engineer may vary as the assessment proceeds and potential influences on the building behaviour become clearer. If the decision is made at any stage that further specialist geotechnical input is not warranted, the structural engineer must review this decision as subsequent work progresses and if necessary seek geotechnical advice. For example if foundation demands are found to exceed ultimate geotechnical capacity, contrary to earlier assumptions, further advice on foundation/soil load-displacement behaviour is likely to be required from the geotechnical engineer.

C4.4.3 Stage 2 – Assessment

C4.4.3.1 Desktop study

If stage 1 determines that there is influential geotechnical information or hazards that may impact the assessment outcome for the building, or if it is unclear what geotechnical hazards may or may not exist, then it is appropriate for the geotechnical engineer to continue with further desktop study (following the initial project definition and scoping).

A detailed desktop study to collate available information relating to existing foundations, underground facilities and soil and groundwater conditions is highly valuable for assessments because:

- This information can be influential to the outcome of geotechnical assessment of existing buildings.

- Obtaining information by physical investigation in the environment of an existing building can be constrained, disruptive and very expensive. Therefore, it is valuable to research in detail what existing information may be available.
- The information from the desktop study can be applied to identify consequential geotechnical issues for the structure and if necessary, target any physical investigations at these issues.

Considerations

Information of particular interest to geotechnical assessment of existing buildings includes:

- Geometry and structural details of existing foundation elements.
- Geotechnical data informing soil and groundwater conditions.
- Seismic hazard information.
- Headroom and access widths for investigation and construction plant.
- Locations of underground services.
- Constraints imposed by any continued building operation including, access noise and vibration limits.
- Information to inform potential for soil contamination.

Researching the last four bullet points can be delayed and only undertaken if assessment concludes that physical investigation and/or construction of works penetrating the ground is required.

Possible sources of information include:

- Seismic assessment reports.
- Design or as-built drawings and specifications of the existing building. Existing foundation dimensions are important for geotechnical assessment. In addition to drawings and specifications, it can be beneficial to obtain information through discussions with those involved in the original construction or subsequent alterations and with the building maintenance personnel.
- Geotechnical reports from the existing or neighbouring building design and construction.
- Soil contamination reports.
- Council property files.
- Council liquefaction hazard maps and subsoil class maps.
- Geological maps such as QMAP geological map series.
- Geotechnical investigation databases such as the New Zealand Geotechnical Database (NZGD).
- Records of shaking intensity and magnitude of historic earthquakes and associated damage in the vicinity. The GeoNet strong motion data tool can provide this information along with various historic papers.

- Historic records of liquefaction such QuakeCore online map[1]
- “beforeUdig” service and direct contact of utility authorities.
- Existing and historic aerial photographs (such as Retrolens).
- Library archive photographs (e.g. National Library online).
- Information held by property owner or maintenance officer.
- LiDAR data to understand existing and historic ground elevation differences (e.g. LINZ)
- Local community pages on social media platforms.
- Site walkover inspection. Site observations and measurements can provide geomorphological information and can clarify historic information and as far as possible confirm or discount the accuracy of that historic information.
- Specialist engineering geologist interpretation of available information and site observations.

NZGS/MBIE Module 2 - Geotechnical investigations for earthquake engineering also provides guidance on undertaking a desktop study to inform likely site ground conditions and geohazards.

Possible deliverable:

Preparing a desktop study report at this stage is suggested:

- To record the work completed
- To collate the information in a readily available form for reference during later stages of the project, and
- To allow the desktop study report to be available to append to reports at a later stage of the project rather than repeating the work.

The content of the geotechnical desktop study should include:

- An outline of the purpose, scope and limitation of the assessment
- A list of information that has been reviewed
- A brief summary and associated conclusions of relevant information
- A list of geotechnical issues (including geohazards) that could influence the seismic assessment of the building. This could be in a risk register form
- An outline of uncertainties and further work that may be required to inform consequential uncertainties.
- A site plan showing key investigation information and a sketch (cross section) to describe the inferred ground model, including the soil profile and existing foundations, and
- Existing foundation drawing information and relevant geotechnical data should be appended to the report. Any information appended to the report should comply with copyright requirements, i.e., be publicly available information or be information for which the owner of that information has given written approval for it to be reproduced.

[1] QuakeCoRE (date unknown), Historical Earthquake Events, accessed from website: https://projectorbit.maps.arcgis.com/apps/webappviewer/index.html?id=140265d6f8754f28851c92dee5491c9a

Note:

Generally, the geotechnical desktop study would provide sufficient information to inform the structural/geotechnical meeting and would not include specific quantitative information related to soil parameters. That meeting would be focussed on understanding the geotechnical issues and the consequences of those issues for the particular structure in qualitative terms. This qualitative discussion is to inform identifying the scope of subsequent work including possible quantitative assessments. This staged and collaborative approach is proposed to allow geotechnical engineering effort to be targeted only on issues and information which could possibly be consequential for the particular structure and life safety.

C4.4.3.2 Structural geotechnical meetings

Once the structural and geotechnical engineers have carried out their desktop studies, they then need to meet to share understanding from these and to explore the scope of subsequent investigation and analysis work (refer to Figure C4.2). An outline of these meetings and collaboration follows:

- **Inputs:**
  - conclusions of geotechnical desktop study (refer to Section C4.4.3.1)
  - results of geotechnical and structural review and analysis, and assessment to date.
- **Initial assessment:**
  - Consider the identified geotechnical issues in conjunction with understanding of structure. Discuss any potential geotechnical step change behaviours. Assess each issue with regard to its impact on %NBS and identify those issues which could be material to the assessment
  - Consider what further analysis and assessment is required and how best to undertake this, focussing on those issues which could be material to the assessment, and
  - Consider the current uncertainties associated with issues which could be material to the assessment. Consider how they are likely to impact on the reliability of the assessment of %NBS rating and, if appropriate, the cost/benefit of further investigations to reduce these uncertainties (refer to Section C4.5).
- **Output:**
  - agreement of updated list of geotechnical issues identified. Categorise these as:
    a) originating from outside the building footprint and thus not influencing the %NBS rating
    b) jointly agreed with the structural engineer as not being critical to the assessment of the %NBS rating
    c) to be specifically assessed
  - agreement on the project categorisation that best describes the potential behaviour of the building and therefore the type of assessment expected; i.e. structurally dominated, interactive, or geotechnically dominated

- agreement of the analyses that will be carried out
- agreement of what, if any, site investigations will be undertaken
- agreement of the geotechnical parameters required as input to the structural analysis and the form in which these parameters will be provided

Several meetings may be required before an output acceptable to all is achieved, as outlined below.

Possible deliverable

Examples of possible outcomes and deliverables from this stage include:

General - Update of desktop study report

An update of the desktop study report may be required to record any further information from the structural engineer and to reflect the understanding of the structural issues and interaction. The list of geotechnical issues (including geohazards) that could influence the seismic assessment of the building may require updating.

Example 1:

Circumstance:

Based on the desktop study and geotechnical/structural discussions it is agreed that further geotechnical input is not expected to be required. The building is likely to be structurally dominated. Foundation demands are expected to be less than the foundations ultimate geotechnical capacity. The geotechnical engineer may conclude involvement in the project at this stage.

Possible deliverable:

A letter from the geotechnical engineer advising the site's seismic subsoil class and possibly the ultimate bearing capacity of foundations, and that the geotechnical engineer and structural engineer have jointly agreed that further specialist geotechnical input to the assessment is not warranted. The geotechnical and structural conditions which led to this conclusion should be listed along with the sources of this information. The desktop study report would be appended to the letter.

Example 2:

Circumstance:

It is agreed by the structural and geotechnical engineers that further targeted geotechnical assessment and possibly investigations are required. Refer section C4.4.3.3.

C4.4.3.3 Investigation, analysis and assessment iterations

As indicated in Figure C4.2 a series of iterations of investigation, analysis and assessment, with collaboration, may follow the initial meeting.

- The geotechnical engineer undertakes investigation, analysis and assessment, and reports the parameters required to the structural engineer.
- The structural engineer applies these parameters to the structural analysis and assessment.
- The structural and geotechnical engineers discuss the results of the analysis and assessment, and consider what further investigation and analysis is required to complete the assessment of *%NBS* rating.

This is an iterative process of reducing uncertainties and increasing understanding of potential building behaviour and, therefore, the %NBS earthquake rating. Each stage of the iteration is purposely targeted at those issues which could be material to the %NBS rating.

Possible deliverable

Possible deliverables from this iterative process are listed below. These reports could be issued in draft and updated with iterations of the assessment process, or interim advice could be provided by email and collated in these reports toward the end of the assessment.

Desktop study report

Factual geotechnical report

- Recording the scope, methods, locations and results of any geotechnical investigations.

Interpretive geotechnical report

- Recording proposed geotechnical parameters to be applied to the structural assessment and supporting information.
- For further information on the content of the interpretive report refer to Section C4.8.

C4.4.4 Stage 3 – Reporting and peer review

As the assessment process (Stage 2) is collaborative and iterative, the geotechnical report cannot be finalised until the assessment is finished. As outlined above, the geotechnical engineer will provide inputs during this process.

Refer to Section C4.8 for guidance on reporting and peer review.

C4.5 Site Characterisation

C4.5.1 General

Understanding the site's ground conditions and how these relate to the foundations, and communicating this adequately, is fundamental to the assessment of an existing building.

C4.5.2 The ground model

The geotechnical engineer should develop the ground model from information collated in the desktop study and site investigations, and update this throughout the investigation and assessment process as more information becomes available. However, the ground model only needs to be of sufficient detail to meet the overall needs of the assessment.

The ground model can be a site plan, cross section, and possibly a table, clearly summarising the inferred soil profile, groundwater level and foundation details, and presence of geohazards. As part of the ground model, it is important to also highlight the uncertainties. Refer to Section C4.5.4.

Variation of soil conditions across the building footprint can be expected. These variations and associated differential effects can be averse to the structure. As part of the ground model, it is important to consider the likely range of ground conditions and how these could vary across the site. Where possible (i.e. informed by available information) delineate the site with different conditions and record these on the site plan and cross section. Where the available information does not allow conditions to be delineated consider how the variability could be spatially characterised, e.g. as local soft or hard spots, or as a gradual transition from softer to harder conditions across the width of the site. Application of this spatial variability of ground conditions is discussed in Section C4.6.7.

This ground model then becomes the basis for discussions between the geotechnical engineer and the structural engineer. Its clarity will also aid in discussions with non-technical personnel (e.g. a building owner or tenant). As part of the ground model it is important to highlight the uncertainties.

C4.5.3 Identifying geohazards

Geohazards are to be identified as part of developing the ground model. The NZGS/MBIE Modules provide guidance on evaluating seismic geohazards as indicated in Table C4.1, Section C4.1.2, including an overview of these in NZGS/MBIE Module 1 - *Overview of the Guidelines*. Sources of information are also described in Section C4.4.3.1.

Geohazards which could potentially affect the earthquake rating of a building include the following (NZGS/MBIE Modules and appendices to this section that will aid the assessment are identified in brackets):

- soil/foundation compression/tension/lateral deformations with loading and the associated effects of deformation of the building (Module 4 and C4.6)

- loss of ground strength and stiffness under the building – liquefaction (sandy soils) and cyclic softening (clayey soils), post liquefaction settlement (Module 3 and Appendices Appendix C4E
- land instability causing loss of support for the building – lateral spread, slope instability, and instability of retaining walls affecting the support of the structure (Module 1 and Appendices Appendix C4B and Appendix C4C), and
- fault rupture under the building and complexities of near-fault effects.

The assessing engineer should consider if and how the relevant seismic geohazards could affect the building. The full range of earthquake demand (%ULS shaking, refer note within Section 4.2.7) relevant to the assessment needs to be considered and reported.

Note:

NZGS Slope Stability Geotechnical Guidance Series provides valuable information related to slope stability. Currently Unit 1 is available, and future Units will be made available in future (Refer Table C4.1 within Section C4.1.2.1).

NZGS/MBIE Module 6 - Earthquake resistant retaining wall design provides valuable information for both design and assessment. Appendix C4B provides supplementary information to be considered in assessment of existing retaining walls and buildings. There is good coverage of retaining wall design in the literature (e.g. Kramer, 1996 and MBIE, 2014), and also insightful coverage of their seismic performance (Wood, 2014).

The location of the surface expression of any future fault movement may not be known with any certainty. It is important that the DSA appropriately discusses the uncertainties involved and the effect these have on the hazard and risks associated with future fault movements on the site.

Geohazards originating beyond the building footprint are not intended to be included in assessment of the earthquake rating. Nevertheless, they may be important considerations if a holistic seismic assessment is to be achieved. This principle is discussed above and in Part A and Section C1. Such geohazards include:

- tsunami or dam break and associated impact and inundation
- tectonic movement leading to flood inundation, and
- rockfall and slope or retaining wall instability from above leading to inundation.

Note:

NZGS/MBIE Module 1 provides general comments on Tsunami: it is not currently planned to include information about the assessment of tsunami hazard within this module series.

C4.5.4 Managing uncertainties

Any investigation of geotechnical issues will involve uncertainties. These should be evaluated and where necessary and appropriate, a targeted investigation programme developed to address them.

These uncertainties could relate to:

- ground conditions
- type and geometry of foundations (shallow, deep or mixed; size; founding level; beam connections and condition, etc.)
- condition of foundations, and
- nature of foundation subgrade (while new builds can include verification testing of foundation subgrades, such information is rarely available for existing buildings).

It is often not economically or technically viable to undertake investigations to resolve all these uncertainties in the assessment process. Due to access constraints these investigations can be considerably more expensive than equivalent investigations for a new build. Therefore, the geotechnical engineer and the structural engineer should collaborate to identify which of these uncertainties could have a material impact on the assessed seismic behaviour and earthquake rating of the building, and develop a targeted investigation in response. Identified critical uncertainties related to geotechnical step change, the critical structural weakness (CSW), severe structural weaknesses (SSWs) and other low scoring structural weaknesses (SWs) are likely to require specific investigation.

Identifying critical uncertainties could include the geotechnical engineer identifying a number of possible scenarios for critical soil and foundation properties (and combinations of these), and the structural engineer testing these scenarios for their impact on the structural seismic assessment.

The geotechnical engineer's description of a scenario could include:

- assumed foundation type, size, depth and founding conditions
- assessed behaviour of this foundation (e.g. soil/foundation stiffness, probable strength (capacity), probable deformation limit)
- likelihood of these assumed conditions or worse/better existing, and
- the scope of investigations considered necessary to verify assumed conditions (i.e. if this scenario is based on conservative assumptions no investigation may be required to verify. If this scenario is based on optimistic assumptions, specific investigations will be required to confirm or modify these assumptions.).

In the first round of the process described above it would be appropriate to assume a scenario with geotechnical parameters which can be relied on without further site investigation (necessarily pessimistic), i.e. to test if these conditions are critical to the structure and if investigation is necessary.

C4.5.5 Site investigations

C4.5.5.1 General

NZGS/MBIE Module 2 - Geotechnical investigations for earthquake engineering provides guidance on desktop studies and physical investigations. This section of these guidelines should be read in conjunction with Module 2 as it provides additional guidance relating to existing buildings.

The first phase of the investigation, the desktop study (refer Section C4.4.3.1), allows an initial ground model to be developed and likely issues and uncertainties to be identified. If potential issues or uncertainties are identified which could be critical to the assessment of the building targeted physical investigations are likely to be required.

The purpose of the geotechnical investigation of an existing structure is to characterise the ground conditions and foundations that the building is supported on. This includes:

- seismic subsoil class (refer to NZS 1170.5:2004)
- ground conditions and liquefaction potential (refer to NZGS/MBIE Module 2 and 3)
- dimensions of existing foundations (refer to Section C4.5.5.2 below)
- foundation load/deformation behaviour (refer to Section C4.5.5.3).

C4.5.5.2 Dimensions of existing foundations

Physical investigation of foundations is sometimes necessary to confirm foundation dimensions and geometry. This may include local excavation around foundations or piles/pile caps by hydro-excavation or other excavation technique. Coring may be used to drill through foundations to confirm foundation dimensions, concrete condition and founding depth, and if extended below the foundation the condition of foundation soils. There are a number of non-intrusive investigation techniques which may provide alternative options or be used in conjunction with intrusive methods. These include the use of:

- a cover meter to check for reinforcement in foundations
- a magnetometer in an adjacent borehole or cone penetration test (CPT) to detect the toe level (or at least the base of reinforcement) in an adjacent pile
- down-hole or cross-hole seismic testing performed adjacent to a pile to detect the toe level (refer to FHA, 1998), and
- pile integrity test methods to estimate the length and condition of a pile.

These can offer relatively convenient and cost-effective investigation methods. However, calibration against independent (preferably physical) methods is recommended, particularly where structure performance is sensitive to results.

C4.5.5.3 Foundation load-displacement behaviour

NZGS/MBIE Module 4 and section C4.6 provides guidance on evaluating the capacity and load-displacement relationship for foundations/soils. Where sensitivity assessment based on this evaluation identifies these parameters to be critical to the assessment of the building targeted physical investigations may be justified to refine these parameters. This could include subsurface investigations (refer NZGS/MBIE Module 2) and/or load testing of a foundation. Load testing of an existing foundation can be undertaken by physically separating the building from the foundation by cutting through the pile and inserting a jack which then loads the pile against the building. There are published examples of this approach (e.g. Jury, 1993). Because of the cost and disruption associated with this load testing delaying this work to be part of a retrofit rather than a DSA may be appropriate.

C4.6 Consideration of SSI Effects

C4.6.1 Key principles

Key principles of considering SSI effects in assessment of existing buildings are described in this section (C4.6) and are listed below:

- Soil-foundation load-deformation behaviour is non-linear, and particularly so at higher demands. The assessment of an existing building may need to consider demand on foundations well into this non-linear behaviour resulting in relatively large and possibly adverse deformations. In contrast, new building foundation design purposely sizes foundations so that the foundations have a predominantly elastic behaviour under design demands. (i.e. SSI effects may be more important in assessment than they are in new design.)
- There can be considerable uncertainty in predicting soil-foundation load-deformation behaviour, particularly where demand exceeds 70% of the geotechnical ultimate capacity of the foundation. When reporting a foundation model and associated parameters the uncertainties in these parameters should also be reported. The structural analysis should include sensitivity analyses to explore the significance of this uncertainty to the assessment (refer C4.2.5).
- SSI effects can be favourable or adverse to the performance of a building. (Refer section C4.6.2).
- The springs that are used in the structural model to represent the load-displacement behaviour at the soil-foundation interface, referred to as the “foundation model”, should capture the interface behaviour across the load-displacement range of interest.
- It is recommended that detailed analysis to quantify soil-foundation load-displacement behaviour be performed separately from the structural model, and the quantified behaviour is then used to establish a simple foundation model in the structural model (e.g. using point springs). This allows soil-foundation behaviour to be separately analysed and understood, including application of specialist geotechnical software e.g. LPILE. (Refer section C4.6.3)

- The recommended steps in considering SSI are indicated on Figure C4.3 and listed below. Not all steps are required depending on whether SSI is influential to the performance of the structure. Each of these steps require collaborative input by the geotechnical and structural engineers. A qualitative step should be completed before any numerical assessment, and this should involve discussion between the structural and geotechnical engineers to inform the approach to subsequent analysis and assessment. Section C4.6.2 discusses qualitative assessments of structures allowing for SSI effects. Section C4.6.4 includes guidance on qualitative assessment of soil-foundation behaviour.

  1. **An initial assessment** of the entire structure to consider if SSI effects could be material to the assessment and if further SSI assessment is warranted. This initial assessment is described in Section C4.6.5 and includes analysis considering the extremes of rigid and flexible soil-foundation behaviour.
  2. **Specific soil-foundation analysis** to define soil-foundation load-displacement behaviour for establishing the "foundation model". Refer 4.6.4.
  3. **Define the foundation model** in the form of point springs or a bed of springs which can be incorporated into the model of the entire structure. Refer 4.6.4.
  4. **Detailed assessment** of the entire structure allowing for SSI effects. The foundation model is represented by the point springs or bed of springs. Refer section C4.6.6. This assessment is likely to be undertaken in stages with the first stage applying relatively simple models and methods. Subsequent and more complex stages of analysis would only be undertaken if the earlier stage indicated this to be warranted. Each stage of analysis is likely to be iterative and collaborative between the geotechnical and structural engineers with refinement of the input parameters, i.e. iterations of steps 2, 3 and 4.

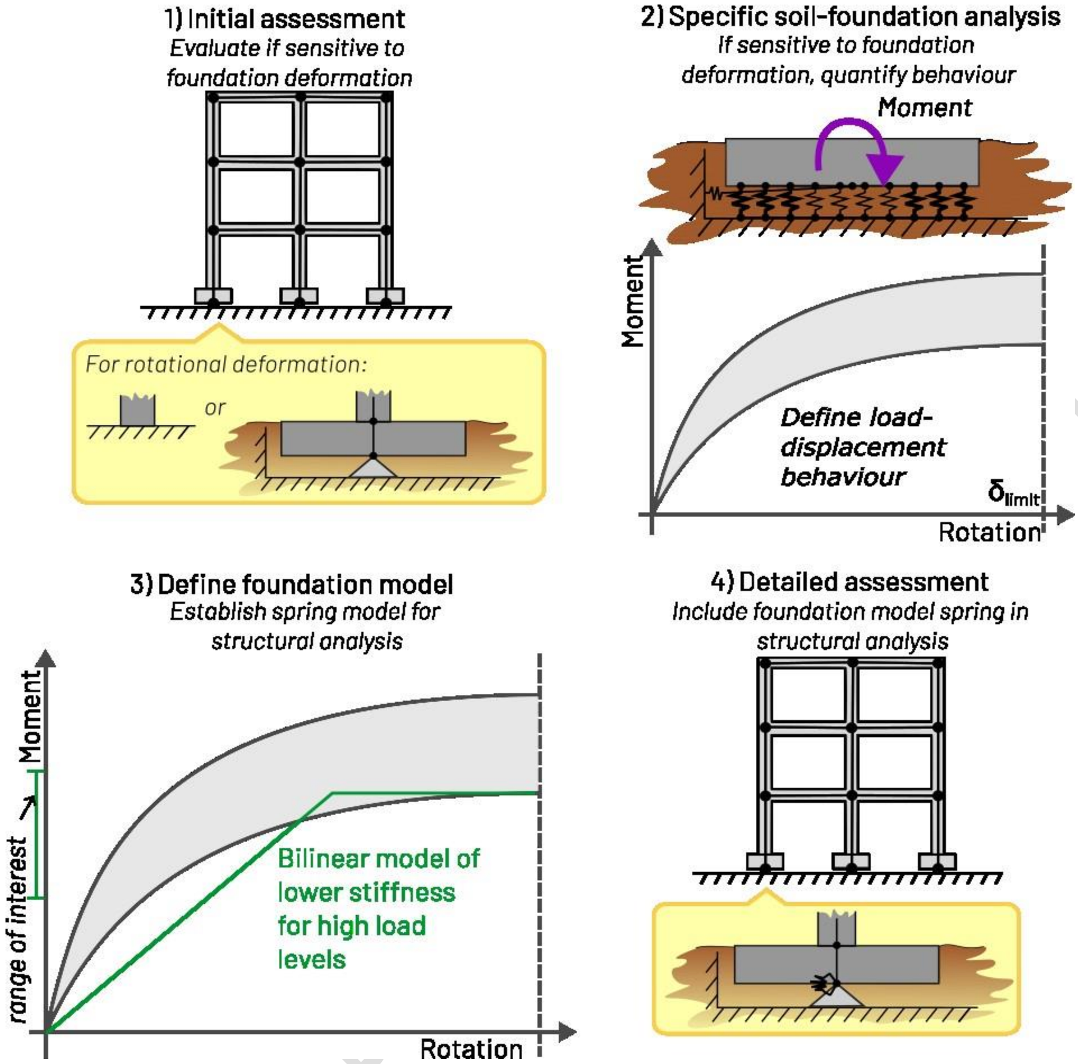


Figure C4.3: Steps in assessing SSI effects

- Vertical and rotational foundation models could be material to the assessment of a building. Lateral foundation models are only likely to be a consideration for structures which are poorly tied together and/or for foundations with low lateral capacity and/or subject to lateral ground movement and/or for assessing foundation elements (e.g. piles) under lateral loading.
- In developing foundation models there are some conditions requiring special consideration. These include:

1. Modelling spatial variation of soils. (Refer C4.6.7)
2. Modelling beyond peak soil resistance. (Refer C4.6.8)
3. Degradation of pile side capacity with cyclic loading. (Refer C4.6.9)
4. Allowing for pore pressure build up, liquefaction, cyclic softening, lateral spread and other effects on soils from earthquake shaking

C4.6.2 SSI effects

Close collaboration is required between structural and geotechnical engineers to understand soil-structure interaction (SSI) behaviour. SSI effects may have a significant influence on the seismic behaviour of a building and the way in which some mechanisms might develop in the structure. Accordingly, possible SSI effects should be considered as part of an assessment and a decision made on how detailed and complex the inclusion needs to be. SSI effects are complex but can often be simplified for assessment; particularly initial screening to assess sensitivity of behaviour.

For example, this could be as simple as recognising that the soil support for a footing may not be rigid and reflecting on what this means for the rigidity of a supported column and its ability to receive flexural resistance/restraint at the base. This may influence the possible actions in the column and mechanisms that are possible in the structure. For this example it may be appropriate to at least consider the possibility of varying restraint, within appropriate bounds, when assessing the structure.

Simple hand checks can be undertaken collaboratively with the structural engineer to assess if the building is likely to be sensitive to the deformation demands from foundation flexibility (e.g. Millen et al., 2020). The extent of acceptable deformations for foundations generally depends primarily on the effect of the ground-induced lateral deformation on the structure and ultimately on the life safety hazard that can develop.

Engineers should note that it is important to consider the potential for the soil to be stronger/stiffer or weaker/softer and for this variability to be non-uniform in distribution. Similarly, imposed displacements or loads may be uniform or differential. Figure C4.4 illustrates a simple example of the range of structural responses as a consequence of the soil strength/stiffness adopted.

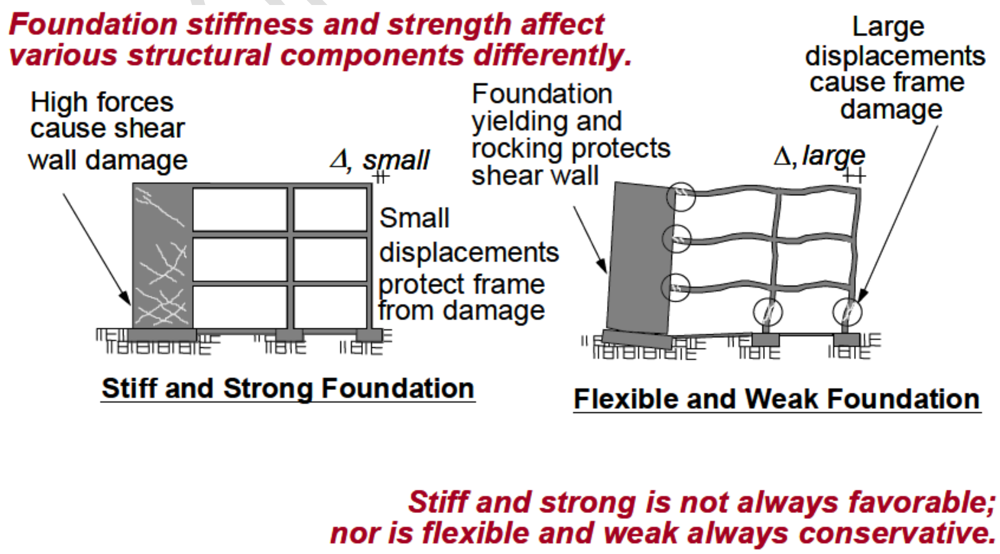


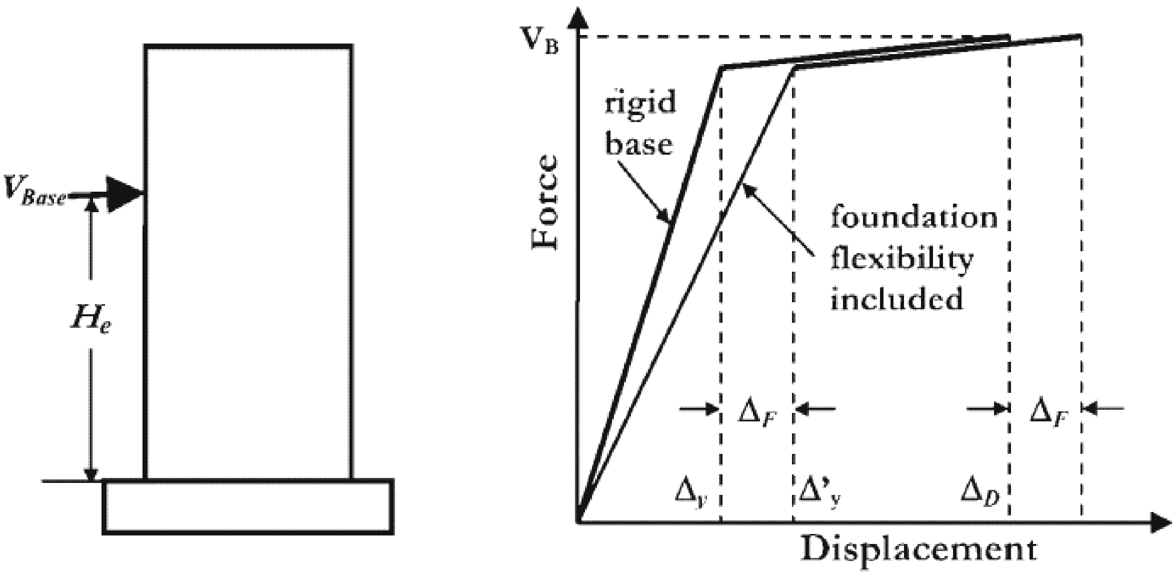
Figure C4.4: Influence of SSI on structural performance (figure adapted from Mahoney, 2005)

Assuming unrealistically stiff soil/foundations (e.g. fixed base assumptions) could result in an unrealistically short natural period of shaking for the structure (unrealistically high seismic loads) or underestimation of structural deformations. The converse also applies.

Note:

Foundation flexibility may increase the deformation at the soil-foundation interface which could affect the behaviour of the building through additional imposed inter-storey drifts on the gravity framing system. The foundation flexibility may also increase the yielding displacement and effectively reduce the achievable ductility of the system. Refer to Figure C4.5.

While the local effect of SSI should be considered (e.g. effect of soil flexibility on the support to the structure), any beneficial effects of foundation radiation damping and kinematic interaction should only be included in the SSI modelling, if there, with detailed consideration (see Section 4.6.6).



(a) Structure **(b) Force-Displacement**

Figure C4.5: Influence of foundation flexibility on displacement and ductility

C4.6.3 Structural model and specific soil-foundation analyses

The foundation model in the structural model should be developed with simple spring models (either point springs or spring bed models, see Figure C4.6), if the initial assessment shows that the structure is sensitive to the foundation flexibility. Point springs are recommended due to the ease of interpreting their behaviour, however, the modelling of flexible mat foundations may require spring bed models.

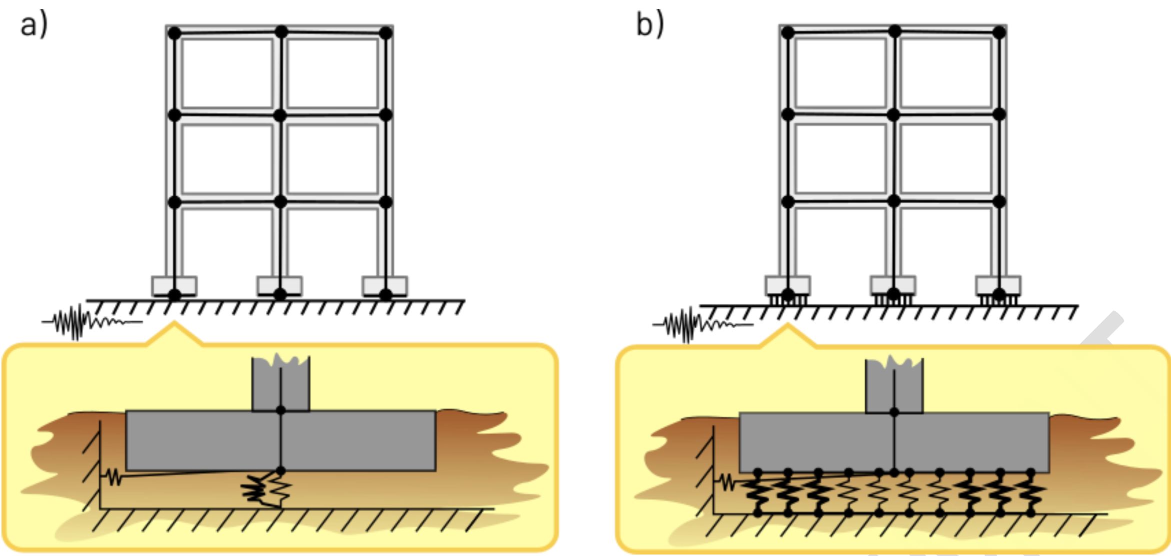


Figure C4.6: a) point springs foundation model, b) spring-bed (Winkler) foundation model

Separate geotechnical analysis and assessment is likely to be required to define the load-displacement behaviour of the soil and foundation and to represent that behaviour in the point springs or bed of springs. Section C4.6.4 provides guidance on this geotechnical analysis and assessment (modelling the soil and foundations).

This separate modelling of the soil and foundation behaviour is recommended because:

- By modelling the foundation element plus supporting soil separately specialist geotechnical knowledge and software can be applied (e.g. LPILE to model a pile and supporting soil) to develop this model. An understanding of the load-displacement behaviour of the foundation-soil can be developed and applied to the assessment.
- Structural models may not realistically represent load-displacement behaviour of soils.
- If the modelling of soils is combined with modelling of the entire structure this can lead to an overly complex model that could produce erroneous results which are not easy to recognise.

Note:

When defining the foundation model, it is recommended to define the behaviour at the base of a shallow foundation or pile cap and model the shallow foundation or pile cap within the structural model along with springs to represent the soil and pile deformation. However, when there is significant deformation within a footing, a more sophisticated soil-foundation analysis may capture the footing deformation behaviour in the load-displacement behaviour curves, therefore the foundation model should be implemented in the structural model at the location where the footing is attached to the structure to avoid double counting this deformation. It is important to report this location.

C4.6.4 Foundation load-displacement behaviour

The load-displacement behaviour of a foundation should be defined with an appropriate level of detail for each mode of interest (e.g. vertical, rotational). Behaviour for modes which are not likely to be material to the structural assessment need not be defined. The behaviour does not need to be defined in detail and should only capture the details appropriate for the level of analysis (e.g. if a footing has loads that are near the ultimate capacity and the building is sensitive to changes in the capacity, focus should be on quantifying the capacity, as the initial stiffness may have limited influence on the response). The evaluation of the load-displacement behaviour can be initially with a wide range and then refined, if necessary, in subsequent iterations of analysis between structural and geotechnical.

Note:

Vertical and rotational modes are often the primary concern for the assessment of a building. The lateral mode is only likely to be a consideration for structures which are poorly tied together and/or for foundations with low lateral capacity and/or subject to lateral ground movement and/or for assessing foundation elements (e.g. piles) under lateral loading.

NZGS/MBIE Module 4 provides some guidance on load deformation behaviour of foundations in the vertical mode. Figure C4.7 and Figure C4.8 have been taken from NZGS/MBIE Module 4. They present examples of vertical load-displacement behaviour of shallow foundations and pile foundations respectively.

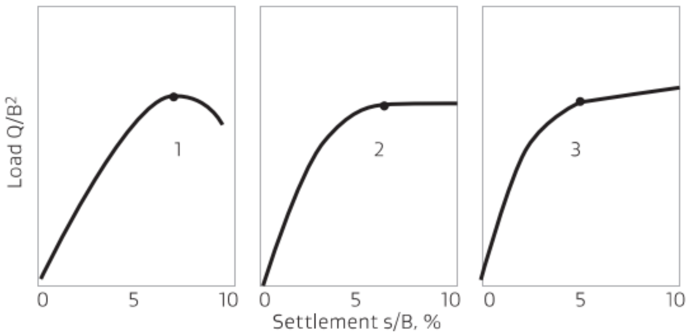


Figure C4.7: Load-displacement shallow foundations [Source: Vesic, 1975]

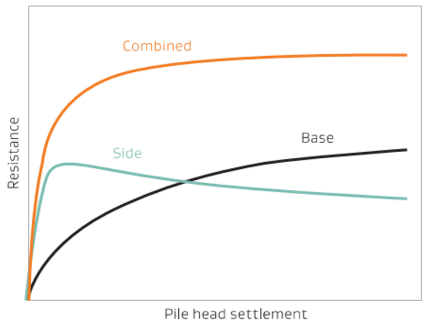


Figure C4.8: Load-displacement deep foundations (Adapted from FHWA 2010)

Note:

The foundation models should include a range of stiffness because of the uncertainty in soil stiffness and its nonlinear nature. In many cases the stiff end of the range can be taken as fixed base, except for when evaluating the structural performance of a foundation element. The soft end of the range is important for understanding the deformation demand on the building. Refer section C4.2.4 for further discussion on selection of geotechnical parameters.

The following general notes and specific notes based on foundation type can inform defining the load-displacement behaviour of a foundation in both qualitative and quantitative terms.

General

a) Identify and consider the mechanism of load transfer to the soil (e.g. Sliding/shearing or bearing or a combination of these) and the soil type and associated load deformation behaviours, e.g. refer Figures C4.7 and C4.8. This will help inform likely behaviour in qualitative terms including beyond the ultimate capacity (beyond the black dots on Figure C4.7).

b) Where foundation models are represented as linear these models are only to be relied on for the load range they were developed to represent. When reporting the linear behaviour, the relevant load range should also be reported.

c) Stiffness at low demand (less than 70% of the ultimate geotechnical capacity) can be assessed by elastic analysis and reported as linear.

d) There can be considerable uncertainty in predicting soil-foundation load-deformation behaviour, particularly where demand exceeds 70% of the geotechnical ultimate capacity of the foundation.

e) When reporting a foundation model and associated parameters the uncertainties in these parameters should also be reported. The structural analysis should include sensitivity analyses to explore the significance of this uncertainty to the assessment (refer C4.2.5).

f) When reporting a foundation model, the maximum deformation for which this model can be relied on, δ_{Limit}, should also be reported. Structural analysis should not extend beyond δ_{Limit}, without review of the model by the geotechnical engineer. δ_{Limit},, is shown on Figure C4.9.

g) When applying software to aid modelling, parallel hand calculations are recommended to challenge the outputs.

h) In developing foundation models there are some conditions requiring special consideration. These include:
   - Modelling spatial variation of soils. (Refer C4.6.7).
   - Modelling beyond peak soil resistance. (Refer C4.6.8).
   - Allowing for pore pressure build up, liquefaction, cyclic softening and other effects on soils from earthquake shaking.

Shallow foundations

a) NZGS/MBIE Module 4 provides some guidance for assessing vertical deformation to mobilise the bearing ultimate capacity of shallow foundations (e.g. 5 to 10% of foundation width).

b) For rotational behaviour the moment capacity can be defined using the simple relationships in Millen et al. (2020), or more sophisticated analyses (see Appendix C4A).

c) Lateral resistance of a shallow foundation is likely to be a combination of friction (sliding resistance) and passive resistance. The sliding resistance is likely to be mobilised with approximately 10mm displacement. The load deformation behaviour due to the passive resistance can be assessed with backbone curves (e.g. Harden et al. 2005).

d) The foundation model for raft or mat foundation is not reliably represented by a uniform bed of linear springs. This is because the pressure distribution through soils beneath a raft foundation influences the equivalent spring stiffness; i.e. a larger area of loading results in a greater depth of influence and greater settlement (softer springs). This can be addressed by iterations between geotechnical and structural analysis and local modification of spring stiffnesses as described in Appendix C4A.

Deep foundations

a) The load-displacement behaviour for lateral or rotational modes of a pile head can be evaluated by applying geotechnical software such as LPILE.

b) The load-displacement behaviour for the vertical mode of a pile should consider the degradation of pile side resistance due to cyclic loading (Refer Section C4.6.9).

c) Refer to NZGS/MBIE Module 4 (including Figure C4.8 above) for guidance on assessing vertical displacement to mobilise pile side resistance ultimate capacity (approximately 12mm) and to mobilise end resistance (5 to 10% of base width). Elastic shortening of the pile shaft also needs to be considered, and in reporting, make it clear that it is pile head behaviour which is reported (or otherwise).

C4.6.4.1 Example foundation models

Figure C4.9 illustrates the expected non-linear load-displacement behaviour of a foundation under vertical, lateral or rotational loading (grey zone). This grey zone represents “expected behaviour”, but actual performance may fall outside this range.

Figure C4.9 and the following commentary outlines possible (simplified) models of a shallow foundation under vertical loading. The purpose of these simplified models is for application to structural analysis. Each model is only relevant to a specific range of load demand, referred to as the “range of interest” in Figure C4.9. It is essential to specify the applicable load range when reporting a model.

a) **Load demand < 40% of the ultimate geotechnical capacity**

Behaviour is predominantly elastic and may be modelled as linear (blue line in Figure C4.9).

b) **Load demand < 70% of the ultimate geotechnical capacity**

Above 40% of the ultimate geotechnical capacity, some nonlinear behaviour can be expected. This behaviour could be simplified as linear up to 70% of the ultimate geotechnical capacity (<70%R), as indicated by the blue line on Figure C4.9.

c) **Load demand > 70% of the ultimate geotechnical capacity**

Significant uncertainty arises. A bi-linear model (green line) may be applied, assuming plastic deformation at ultimate capacity. Loads beyond this cannot be resisted. The "knee" of the model—typically at the ultimate capacity (R) —should be defined by the geotechnical engineer, with a moderately conservative (high or soft) estimate of deformation (e.g., 10% of foundation width, refer NZGS/MBIE Module 4). Due to uncertainty, initial structural assessments for demands between 70–99% should consider deformations of double this moderately conservative estimate. (refer C4.6.5). If this 2x deformation critically affects the assessment, further discussion between structural and geotechnical engineers is advised. In some cases, the upper estimate (2× deformation) may need to be adopted.

This model considers the soft (high deformation) end of the estimated behaviour. Separately the structural assessment should consider the stiff end of the estimated behaviour. For initial assessment rigid behaviour could be considered (refer C4.6.5).

Refer to Appendix C4H.2 for further discussion of assessment with foundation demand > 70% of the foundation's ultimate geotechnical capacity.

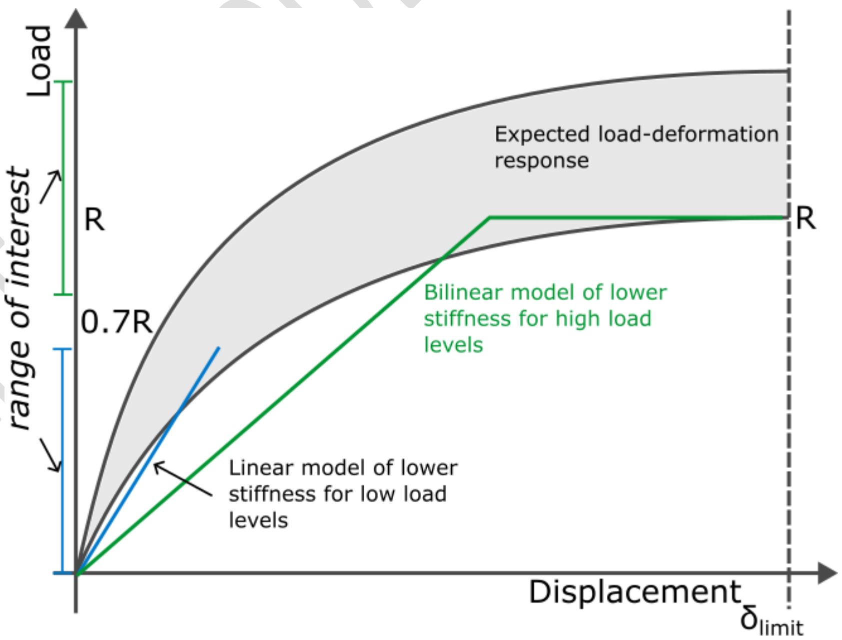


Figure C4.9: Expected load-displacement behaviour and possible foundation models. Blue line: foundation model for low demand (load) levels. Green line: foundation model for high demand (load) levels.

C4.6.4.2 Model Use and Limitations

The linear and bi-linear models are simplifications of complex, nonlinear behaviour, which includes considerable uncertainty. When used with sensitivity analysis and a clear understanding of their limitations, they are suitable for most cases. They help assess structural vulnerability to large deformations. More advanced methods (e.g., dynamic finite element analysis) may be appropriate for complex SSI problems or high-consequence projects, typically as a second-stage refinement. The second stage, more advanced methodology is typically reserved for complex soil–structure interaction (SSI) problems, understanding failure mechanisms, and refining assumptions or limitations of simpler analyses, particularly for high-value or high-consequence projects.

Note:

Most structural engineering software cannot fully capture the nonlinear behaviour of soil–foundation systems and must therefore idealise it. Comparing the expected nonlinear response with the modelled behaviour helps structural engineers understand the impact of these simplifications. To support communication, geotechnical findings should be illustrated, e.g., using sketches similar to Figure C4.9.

C4.6.5 Initial assessment

Due to the additional complexities of SSI modelling, the purpose of the initial assessment is to establish whether SSI analysis is required.

The following is suggested for the initial assessment. This initial assessment should be undertaken prior to undertaking the modelling of foundations described in C4.6.4. A qualitative assessment should be undertaken to identify which modes of deformation (vertical, rotation and lateral) could be material to the assessment of the structure. For the modes which are not likely to be material to the assessment these may be modelled as fixed for this initial assessment. For the other modes analyse the structure assuming the extremes of foundation models as follows:

- For the rotation mode assess the structure for a fixed foundation and then for a pinned foundation.
- For the vertical mode assess the structure for a fixed foundation and then for the flexible (soft) foundation model indicated by the green line on Figure C4.9. Sensitivity check should consider double this deformation. (Refer C4.6.4.1).
- For the lateral mode assess the structure for a fixed base and then for the flexible foundation model indicated by the green line on Figure C4.9. Sensitivity check should consider double this deformation. (Refer C4.6.4.1).

If this initial assessment indicates that SSI effects are not likely to be material to the structural assessment, more detailed SSI analysis is usually not warranted. If this initial assessment indicates that SSI effects are likely to be material to the structural assessment, then more detailed SSI analysis may be warranted including refining the foundation models as described in Section C4.6.6.

For the described initial assessment, the only input required from the geotechnical engineer is assessment of ultimate capacity and the displacement to mobilise this ultimate capacity (i.e. the location of the knee in the green line on Figure C4.9) for vertical and lateral modes, and then only if these modes have been identified as possibly being material to the structural assessment. Assessment of these capacity and deformation parameters is to be moderately conservative, refer C4.2.4. Assessment of these parameters is to make allowance for the following where appropriate:

- Uncertainty. (Refer C4.6.4.1 c))
- Modelling spatial variation of soils. (Refer C4.6.7).
- Modelling beyond peak soil resistance. (Refer C4.6.8).
- Degradation of pile side capacity with cyclic loading. (Refer C4.6.9).
- Allowing for pore pressure build up, liquefaction, cyclic softening and other effects on soils from earthquake shaking.

C4.6.6 Detailed assessment

If the initial assessment indicated that SSI effects could be material to the assessment more detailed assessment may be warranted. In this more detailed assessment, the soil-foundation behaviour can be represented by point springs or bed of springs as described in section C4.6.4. The assessment is likely to be undertaken in stages with the first stage applying relatively simple models and methods. Subsequent and more complex stages of analysis would only be undertaken if the earlier stage indicated this to be warranted. Each stage of analysis is likely to be iterative and collaborative between the geotechnical and structural engineers with refinement of the input parameters, i.e. iterations of refining the foundation model and how it is represented in the structural model (Section C4.6.4) and analysis of the structure with these refinements.

These iterations could include the following:

1. Considering whether the foundation model reflects the load-displacement behaviour across the range of interest (i.e. if the loads are small then adopting a stiffness that is secant-to-failure may be overly flexible), this is typically an iterative process.
2. Using a more advanced foundation model, e.g. adopting a bilinear spring instead of a linear spring.
3. Improving the understanding of the load-displacement behaviour and adjusting the model to reflect this. This refinement could be from better understanding of ground conditions through site investigation and/or specific soil-foundation analysis.

Moderately conservative, soft, foundation models should be assumed to assess deformation of the structure. In addition, where applicable the structure should be tested for possible scenarios of variation of stiffness across the building, e.g. one pile being relatively soft and all adjoining piles being relatively stiff, refer C4.6.7. Appendix C4A provides details on SSI analysis.

Note:

There can be some beneficial influence of SSI on a building's life safety performance (e.g. elongation of building period, concentration of displacement demands in "ductile" foundation rotation or rocking, damping resulting from plastic soil behaviour, etc.). When relying on these beneficial effects moderately conservative properties should be adopted, in this case probable upper stiffness and strength. The upper probable properties should consider both the uncertainty in soil properties, as well as additional mechanistic resistance, e.g. the role of soil-foundation contact along the sidewalls, peak strength response of the soil, the potential for increased vertical load (and therefore increased moment capacity) due to restraint against uplift, resistance from the floor slab and foundation tie elements. Additionally close collaboration between the geotechnical and structural engineer is imperative to evaluate the role of soil variability on the improved performance, as well as cross-check the validity of the SSI model. Where the assessment is complex or requiring significant judgement appropriate peer review should be considered, refer C4.8.

C4.6.7 Spatial variation of soils

As discussed in Section C4.5.2, variation of soil conditions across the building footprint can be expected. These variations and associated differential effects during or following seismic loading can be adverse to the structure.

The structural and geotechnical engineers should discuss the expected variability of ground conditions and the expected consequences of this for the structure and, if considered necessary, jointly develop foundation models and their distribution across the site to represent the spatial variation, and to be applied in the structural analysis. To avoid being overly conservative and to avoid excessive structural analysis, these representations of the spatial variation are to be limited to a few scenarios which are considered realistic and are chosen because they could be critical for the structure.

Where sufficient information is available to delineate the site with different ground conditions this should be indicated on a plan and different foundation models provided accordingly. Where local variations are expected but the locations of these variations are not known, realistic scenarios should be developed to model this, e.g. Assume one pile (or footing) is adversely affected by local liquefaction effects as represented by foundation model A, while all other piles (footings) are not subject to liquefaction effects and are represented by model B. Select the pile or footing for model A to be that which would result in the most adverse effect on the structure.

To help communication it is recommended that the geotechnical conclusions be reported with the aid of sketches, e.g. simple cross sections and plans and models in the form of Figure C4.5.

C4.6.8 Modelling beyond peak soil resistance

Some soil-foundation behaviour can exhibit reducing resistance with deformation beyond a peak resistance. Figure C4.10 represents this behaviour. Where this loss of resistance beyond the peak could be more than 20% over the deformations of interest for the structural assessment, special allowance for this should be made in the modelling. An example could be a shallow foundation bearing on sensitive volcanic soils. This reducing resistance with deformation could be specifically modelled in the structural analysis. Alternatively, either of the two following simplified models could be applied (whichever gives the most favourable outcome):

- Option 1: Model as elastic plastic as indicated by the green line on Figure C4., with RR being taken as the residual capacity (the resistance after deformation).
- Option 2: Model as elastic up to a limiting resistance and displacement as indicated by the blue line on Figure C4.10. The limiting resistance is to be taken as 75% of the peak resistance (0.75R). The structural analysis is not to go beyond this limit. This provides a buffer away from the unfavourable, beyond peak behaviour.

Note that these simplified models are not proposed for pile side resistance subject to cyclic loading. Refer C4.6.9 for this situation.

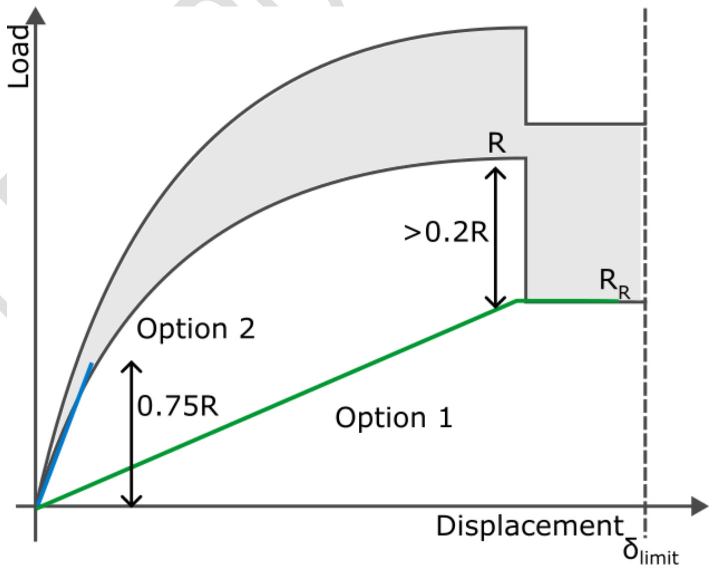


Figure C4.10: Modelling options for peak resistance

C4.6.9 Degradation of pile side capacity with cyclic loading

C4.6.9.1 Introduction

Cyclic axial loading of piles causes degradation of the side resistance mechanism resulting in a reduction of pile axial load capacity. Degradation can be significant in cases where the direction of loading alternates between compression and tension during an earthquake. For design, strength reduction factors are normally applied such that the cyclic load levels are too low to cause a significant problem and cyclic degradation is usually ignored. Exceptions would be piles carrying a small amount of building dead load compared to earthquake overturning loads. But for assessment, strength reduction factors are not applied, and cyclic load levels may be high enough to cause significant degradation requiring consideration. Degradation of side resistance may be an important issue in the assessment of micropiles, other friction piles, and end bearing piles subject to uplift from earthquake overturning loads.

In considering potential degradation of pile side resistance due to cyclic loading the following should be noted:

- The pile side resistance is potentially degraded by cyclic loading, however the end bearing capacity is likely to be maintained (Poulos, 1988).
- Side resistance degradation is particularly an issue for micropiles, friction piles or end bearing piles in tension, i.e. piles that rely on side resistance and piles that have a high aspect ratio (Poulos, 1988)
- Reverse cyclic loading (tension and compression) can be particularly damaging to pile side resistance, but cyclic loading without reversal (tension or compression cycles but not both) can also be damaging. The effect of the loading regime is discussed in the references.
- The test results discussed in the references and the typical values reported above relate to model and full-scale bored and driven piles in sand and clay. Piles in sensitive soils, volcanic soils, and rock will require special consideration. Note that the residual capacity of pile side resistance after cyclic loading in sensitive soils and weak rock (carbonate soils, chalk) is likely to be low. No relevant published test data for this uncommon situation has been found.
- Liquefaction or increases in pore water pressures also degrade pile capacity. These effects need to be separately considered for susceptible soils.

C4.6.9.2 Background

When piles are subjected to vertical cyclic loading this can lead to degradation of the pile's side resistance as indicated by the example test result in Figure C4.. If the cyclic loading does not exceed the pile side "stability limit" cycles of load can be applied without degradation of the side resistance. However, if loading is applied above this threshold the side resistance can degrade as indicated in Figure C4..

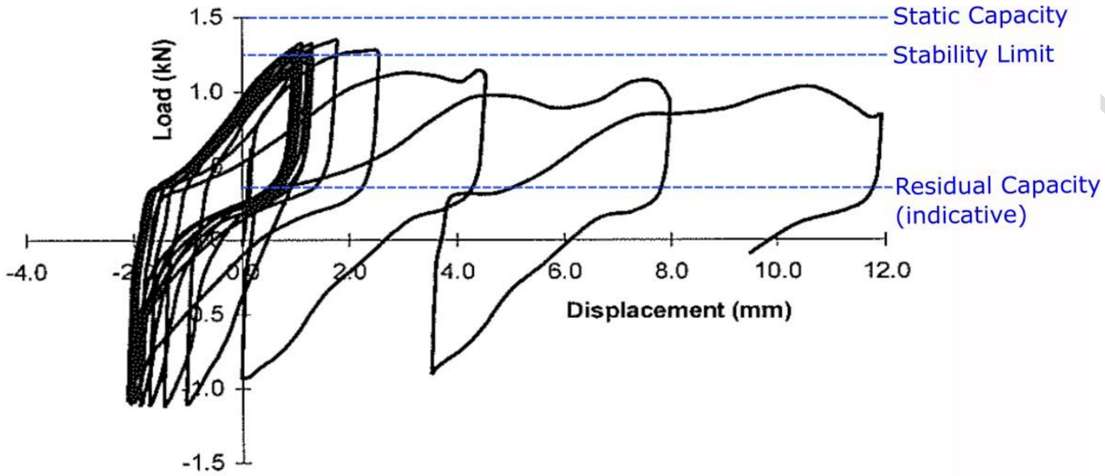


Figure C4.11: Centrifuge modelled bored pile in dense sand subject to cyclic axial load with reversal [Source McManus 2003]

The test result shown by Figure C4. is of a centrifuge test model of a bored pile in dense sand subject to cyclic axial loading with load reversal. The static capacity of the pile is greater than the loads applied during the test. The amplitude of the applied cyclic load just exceeds the threshold (stability limit) and this leads to progressive degradation of the ground to side resistance with each cycle of loading. With further cycles resistance could degrade to a residual but this test did not extend to that. In design this effect is allowed for by reducing the available capacity of the pile. In assessment demand could exceed the stability limit and therefore this effect needs to be modelled.

C4.6.9.3 Evaluating the stability limit

The static capacity, stability limit and residual capacity of pile side resistance can be investigated by tests of the type indicated in Figure C4.. The following references record results of tests which can be helpful in estimating these parameters: Sandoval, Webb and Palmer (2019), Puech and Garnier (2017), McManus (1997, 2003), Jardine et al. (2012), Turner & Kulhawy (1990) and McManus & Kulhawy (1994), McManus & Turner (1996); Poulos (1988). A wide range of values of stability limit are reported depending on the loading regime and pile and ground conditions. A stability limit of 50 to 80% of the static capacity are typical results. Limited information is available on residual capacity, but it could be less than 40% of static capacity. NZGS/MBIE Module 4 (2021) includes guidance on assessing static capacity of pile side resistance.

The stability limit "S" of the pile side resistance could be assessed by a review of literature and published test results or by load testing. The assessment is to consider factors including the loading regime and nature of the soils or rock. If a specific assessment by a geotechnical engineer is not undertaken values of S no greater than the following should be assumed.

- Reverse cyclic loading (Compression and tension):
  S = 0.6 x the moderately conservative static capacity of pile side resistance.
- Cyclic loading in tension only:
  S = 0.7 x the moderately conservative static capacity of pile side resistance.
- Cyclic loading in compression only:
  S = 0.7 x the moderately conservative static capacity of pile side resistance.

A lower value of pile side resistance and S is to be assessed and applied for sensitive soils and rock. Higher values of pile side resistance and S in some situations could be considered if supported by specific research or investigation and assessment.

C4.6.9.4 Assessment methodology

The following two stage methodology is proposed to allow for degradation of pile side resistance with cyclic loading in building assessment. The %NBS score is taken as the lowest calculated from the two stages:

Stage 1: Initial cycles of earthquake

Model as elastic plastic (bilinear) as indicated by Figure C4. with R being the moderately conservative static capacity of the pile and the deformation to mobilise this static capacity to be taken as the lower end of the expected range, i.e. stiff. Or if appropriate and for simplicity the foundation could be modelled as fixed base for Stage 1.

If structural assessment determines that demand does not exceed the assessed stability limit, S, Stage 2 is not required.

If demand exceeds S this model in conjunction with the upper estimate of stiffness considers a stiff foundation response to allow this effect on the structure to be assessed, i.e. considers the effect of the stiff foundation early in the earthquake before pile side resistance has degraded. Stage 2 is required if demand exceeds S.

Stage 2: During the earthquake

Model as elastic plastic (bilinear) as indicated by Figure C4. with the lower estimate of stiffness and the capacity R being 0.8 x S. The factor of 0.8 is included to make some allowance for the degradation of resistance with each cycle of loading (Refer Figure C4.). This models a soft foundation response to allow this effect on the structure to be assessed.

C4.7 Identifying and allowing for geotechnical step change

C4.7.1 General

A "geotechnical step change" is a **sudden** and **large** adverse change in geotechnical behaviour, with increasing shaking demand. Examples include:

- Reduced support to a foundation due to liquefaction
- Ground displacement due to liquefaction and lateral spread or cyclic displacement
- Heave of basements due to liquefaction
- Ground displacement due to slope movement triggered by earthquake shaking

Soils can exhibit nonlinear load-deformation, or change in behaviour with increasing load or deformation demand. This change in behaviour is allowed for in the methods of assessment outlined in Section C4.6, including degradation of pile side resistance with cyclic loading (C4.6.9) and reduced resistance beyond a peak (C4.6.8). This section (C4.7 Geotechnical step change) only relates to change in behaviour "with increasing shaking demand".

It is important to consider if the occurrence of any geotechnical step change could lead to a loss of gravity support of a structure and a significant life safety hazard (SLSH). A geotechnical step change is only consequential in assessing a %NBS rating of a building if that geotechnical step change leads to a SLSH in the structure. Examples where a geotechnical step change may not lead to a SLSH include:

- A frame structure with a high tolerance to deformation.
- Post liquefaction settlement.
- Slope movement of a magnitude which can be tolerated by the building

Figure C4.12 below describes reduced support to a foundation due to liquefaction as a geotechnical step change. An example of calculated reduction in bearing capacity of a shallow foundation on liquefiable soil with increasing intensity of earthquake shaking is shown. For this example, from 20% to 60% ULS shaking pore water pressures build up resulting in a reduction in bearing capacity. At 60% ULS shaking triggering of liquefaction in the supporting soil occurs and a geotechnical step change of bearing capacity is calculated.

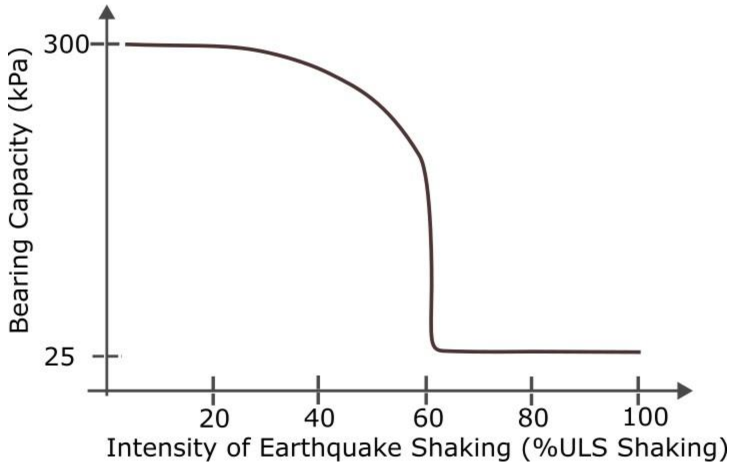


Figure C4.12: Calculated bearing capacity and intensity of earthquake shaking

A critical aspect of the geotechnical assessment is identifying potential geotechnical step change and making appropriate risk-based allowance for it. Factors associated with a geotechnical step change which are to be considered and allowed for include:

- **Uncertainty:**

  Geotechnical behaviour after a step change is likely to be considerably more uncertain than that without a step change.

- **Continuum of building performance:**

  A continuum of building performance with increasing levels of seismic shaking is expected by the building code limit state design framework. There is an expectation that up to 150%ULS shaking level stability of the structure will be maintained. Similarly, when assessing the relative performance of an existing building against new build standard (%NBS) a continuum is expected from exceedance of a structure's probable capacity through to possible loss of gravity support. The expectation is that loss of gravity support would not be expected until a shaking hazard of 1.5 times that causing the exceedance of probable capacity. If this continuum of structural performance does not exist because of a step change, the calculated %NBS score for this issue is to be reduced by a factor X to allow for this. Details of when and how X is to be applied are discussed in section C4.7.2.

- **Consequences:**

  If the step change results in a loss of continuum of building performance and the consequences in terms of life safety are severe, the calculated %NBS is further reduced to allow for this higher risk of loss of life (i.e. X is increased as described in C4.7.2).

Note:

The consideration of a step change factor in assessing %NBS follows similar reductions required in the consideration of other brittle structural failure mechanisms. For example, Severe structural weakness's (SSW) (as listed in Section C1.5.3.1) and loss of support to a precast floor system (as described in Section C5) also consider the three factors of uncertainty, continuum of building performance and consequences, as part of their assessment methodology. That methodology for these specific issues also includes a reduction of the calculated %NBS by a factor X. Two of the six mechanisms predetermined as SSW's in Section C1.5.3.1 are as a direct consequence of geotechnical step change. These are described in Section C1.5.3.1 as:

- Complex slope failure resulting in significant ground mass movement and loss of support over more than 50% of the building platform (i.e. where the building is on a slope or cliff edge), and

- Liquefiable ground supporting poorly tied together URM buildings (refer to Section C8 for definition) with more than two floors.

Where these SSW's are identified they are to be assessed as described in Section C2 (Appendix C2G). Where a geotechnical step change is identified but does not result in a SSW then that geotechnical step change is to be assessed as described in C4.7.2.

For new building design NZGS/MBIE Module 4 (2021) and the commentary to TS1170.5 (2025) propose that this continuum of performance beyond ULS be provided. In the case of the commentary to TS1170.5 (2025) it is proposed that geotechnical step change be considered for shaking intensity up to 150%ULS shaking and the building structure and its foundations be configured to avoid collapse. This is consistent with the X factor of 1.5 proposed here for assessment. Note that the expectation is not to satisfy ULS design criteria at 150% ULS shaking but to provide a continuum of performance of the structure such that at 150%ULS shaking loss of gravity support leading to a significant life safety hazard would not be expected.

C4.7.2 Geotechnical step change assessment methodology

The methodology for assessment and allowance of geotechnical step change has been developed to address the three factors described in Section C4.7.1:

- Uncertainty
- Continuum of building performance, and
- Consequences.

The methodology includes two stages. %NBS is calculated assuming the most adverse of the two stages. Stage 1 makes allowance for uncertainty and stage 2 makes allowance for continuum of building performance and consequences. Figure C4.73 presents a flow diagram describing the methodology. The flow diagram is followed by text describing the methodology.

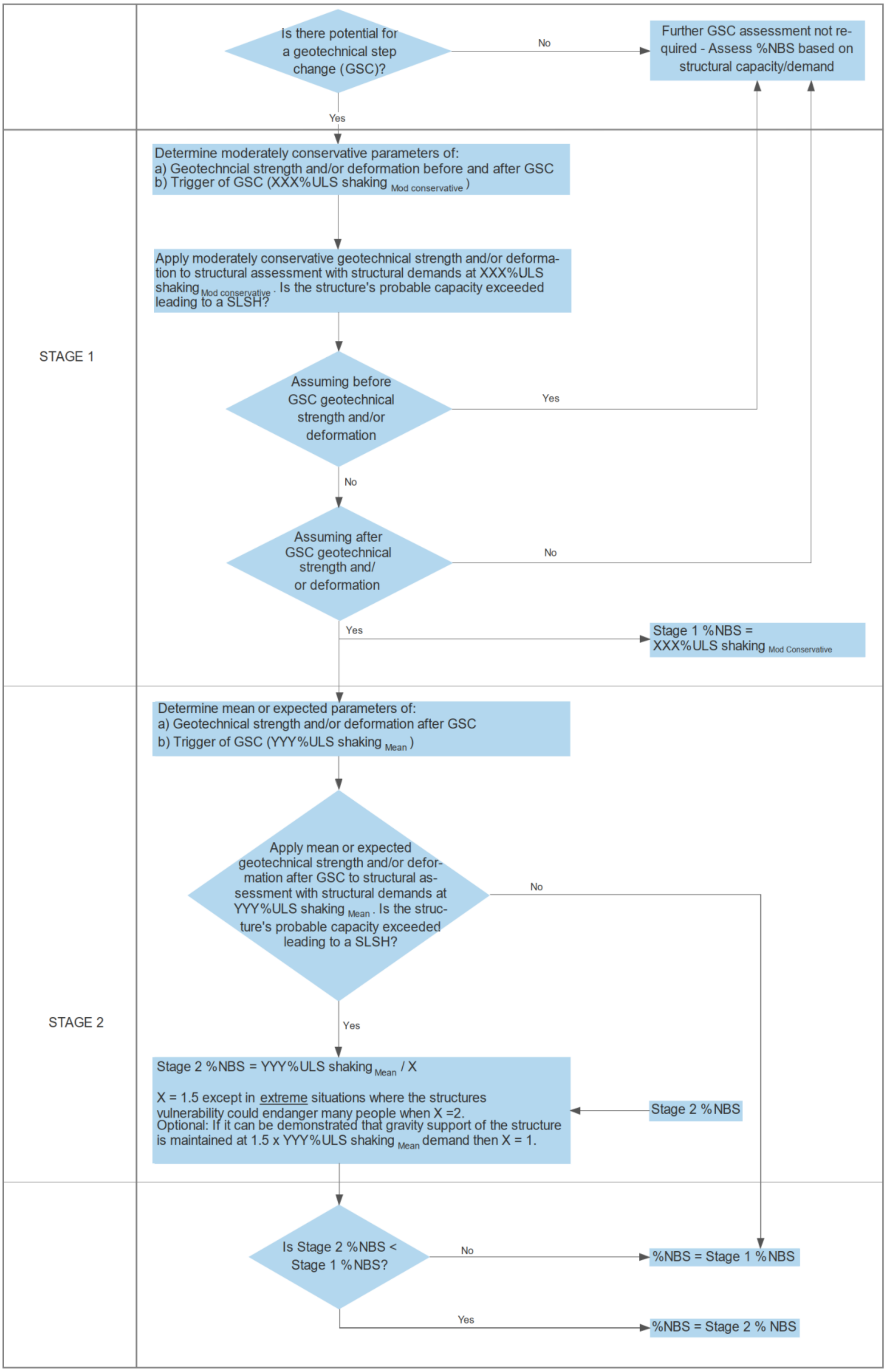


Figure C4.73: Geotechnical step change assessment methodology

Stage 1: Determine %NBS based on moderately conservative geotechnical parameters and no step change factor, X. The purpose of this stage is to make some allowance for uncertainty in geotechnical parameters. The methodology for this stage is the same as would be applied if a geotechnical step change is not expected. In reporting any geotechnical parameter the uncertainty in that parameter should also be reported (refer C4.2.5).

Moderately conservative geotechnical parameters are applied to model geotechnical behaviour and allow for uncertainty in these parameters. Uncertainties are typically greater when a geotechnical step change occurs (e.g. liquefiable soil strengths, lateral spread displacements). If geotechnical step change occurs, and results in exceedance of the structure's probable capacity leading to a SLSH then the %NBS score for this issue = %ULS shaking triggering the geotechnical step change. If geotechnical step change does not occur, or does not directly result in exceedance of the structures probable capacity and/or does not lead to a SLSH, then the %NBS score would be assessed by the usual structural capacity/demand methodology. If geotechnical step change occurs residual (after step change) geotechnical capacity is considered.

Stage 2: Determine %NBS based on mean or expected geotechnical behaviour and apply reduction by step change factor X if required. The purpose of this stage is to provide a buffer beyond the structure's probable capacity to loss of gravity support of the structure and to consider consequences. Mean or expected parameters are applied to avoid accumulation of conservatism which could result if moderately conservative parameters were applied in conjunction with X.

Mean or expected geotechnical parameters are applied to model geotechnical behaviour for stage 2. If the geotechnical step change results in exceedance of the structure's probable capacity leading to a SLSH then reduction in the %NBS score is required by factor X to ensure a continuum of performance (a buffer between exceedance of probable capacity and loss of gravity support). The stage 2 %NBS score for this issue becomes:

$$\%NBS = \frac{\%ULS_{shaking}(triggering\ the\ geotechnical\ step\ change)}{X}$$

X is the step change factor. $X = X1 * X2$

- X1 is to provide a buffer beyond the probable capacity to the structure's loss of gravity support.
  X1 = 1.5. The option is available to undertake assessment to explore if gravity support can be expected to be maintained at 1.5 times the $\%ULS_{shaking}$ triggering the step change. If gravity support is maintained, then X1 = 1.0.
- X2 is to allow for the consequence of loss of gravity support.
  In general X2 = 1.0, but in extreme circumstances where loss of gravity support could result in collapse of multiple floors or other unfavourable mechanism endangering many people and X1 from above has been assessed to = 1.5, then X2 = 1.3.

If in doubt as to whether a geotechnical step change exists and if specific allowance needs to be made for it in calculating %NBS, the methodology can be applied to provide resolution i.e. if allowance for geotechnical step change is not required the methodology will determine X=1.0.

In some situations, the geotechnical step change may develop over a range of %ULSshaking (e.g. lateral spread). This can make identifying a specific %ULSshaking triggering the step change unclear. In these situations the "%ULSshaking triggering the step change" to be assumed in calculating the %NBS may be taken as the %ULS shaking which triggers the exceedance of the structure's probable capacity leading to a SLSH.

Notes:

This methodology is for assessment of the issue of the geotechnical step change. For this issue's %NBS score to dictate the %NBS rating of the building it would need to be the structures lowest scoring member/element/issue leading to a SLSH.

The "structure's probable capacity" is the capacity of the structure to resist seismic and gravity loads (i.e. ULS performance), as determined in accordance with the Guideline i.e. assuming the structures probable material strengths, which aims to measure performance against a "life safety" objective similar to a new building.

The "loss of gravity support of the structure" is deformation or damage to the structure such that it can no longer support the imposed gravity loads, and may be thought of as akin to a collapse limit state in other guidance documents such as ASCE 41. In assessing whether or not loss of gravity support is expected the structural engineer needs to consider if there are any alternative load paths and whether that loss of gravity support could pose a SLSH to more than one person. Further guidance on assessing for loss of gravity support and SLSH is provided in *"Applying Engineering Judgement in Determining When a Significant Life Safety Hazard Occurs"* (https://design.resilience.nz/resources/view/jc-25-01-applying-engineering-judgement-in-evaluating-significant-life-safety-hazard) (JC-Sar 2025).

"Loss of gravity support of the structure" refers to the structure as a whole and not just the foundations. Deformation or loss of support from the foundations contributes to "loss of gravity support of the structure", but deformation or loss of support from the foundations alone may not necessarily result in "loss of gravity support of the structure" if the structure can tolerate the resultant effects.

X=1.5 is consistent with the considering up to 150%ULS shaking for new building design proposed in the commentary to TS1170.5 (2025), refer the notes in Section C4.7.1. X =2 (X2=1.3) is a special case for existing buildings with particularly adverse details with respect to life safety. Such circumstances are not expected for a new building. The consequences in the extreme case of an existing building where X=2 is applied aligns with the response to SSW's.

The IL3 response to "may contain people in crowds" of increasing the design ULS shaking is separate and additional to what is referred to here in applying X2=1.3. The X2=1.3 is in response to the assessed vulnerability of the structure in terms of SLSH.

C4.7.3 Geotechnical step change examples

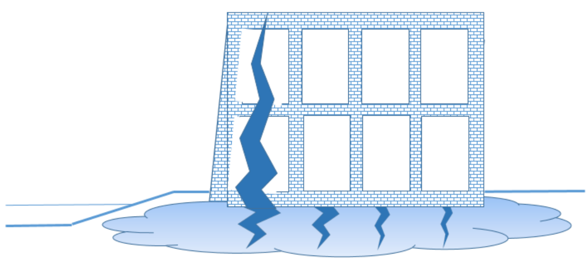
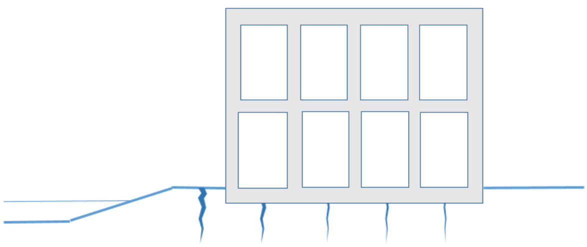
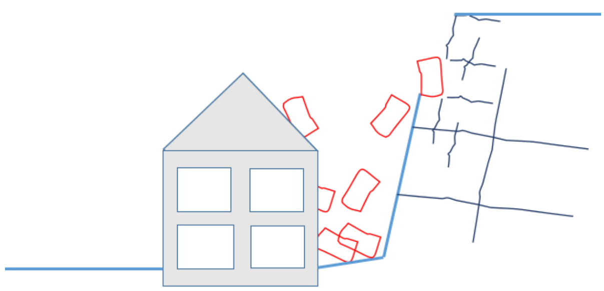
Appendix C4G summarises the application of the methodology described in C4.7.2 to assessment of several buildings affected by geotechnical step change.

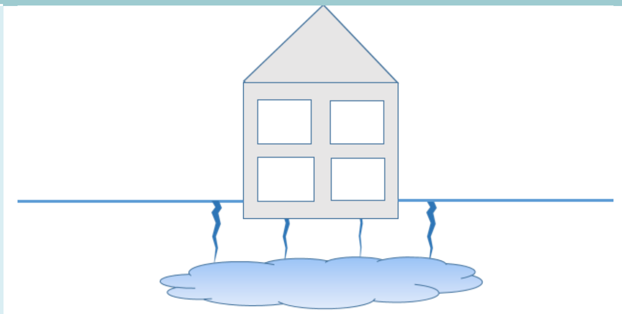
Appendix C4H provides examples of allowing for uncertainty in these assessments.

The effect of some geotechnical changes on the performance of some structures may not be sufficiently severe to warrant a reduction in %NBS by factor X, i.e. the methodology determines X1=1.0

Table C4.2 provides some examples of sites subject to geotechnical step changes and considers whether or not the effect of that geotechnical step change on the particular structure is likely to be sufficiently severe to warrant the %NBS being reduced by a factor of X (X=1.5 or 2.0). The purpose of this Table is to provide an indication of what could be expected as the outcome of assessment in accordance with this Guideline. However, the actual outcome for a particular situation will depend on specific assessment in accordance with this Guideline.

Table C4.2: Examples of geotechnical step change

Description		Is the geotechnical step change likely to result in the %NBS being reduced by a factor of X (X=1.5 or 2.0)?
Unreinforced masonry building on site subject to liquefaction and lateral (flow) spread		Likely to require % NBS to be reduced by X, unless the structure above is well tied together. If it were > 2 stories, consider as SSW.
Building on site subject to co-seismic slope movements (but slope failure/ evacuation is not expected)		Unlikely to require %NBS to be reduced by X if the building and/or its foundation is well tied together
Light timber frame dwelling in a rockfall impact zone		A geotechnical step change but inundation is not considered when assessing %NBS rating.

Description		Is the geotechnical step change likely to result in the %NBS being reduced by a factor of X (X=1.5 or 2.0)?
Light timber frame building on a site subject to liquefaction		Unlikely to require %NBS to be reduced by a factor of X.

C4.8 Reporting and Peer Review

C4.8.1 General

Reporting should follow the general requirements set out in Section C1.

In all cases, the %NBS will be defined by the structural engineer in their reporting, as detailed elsewhere in these guidelines.

The scope of investigation and analysis by the geotechnical engineer should be acknowledged in the structural engineer's assessment report and the geotechnical report should be appended, together with the peer review report where applicable.

The assessment process is collaborative and iterative (refer to Section C4.4) and, as a consequence, the geotechnical report cannot be finalised until this process has been completed. The geotechnical engineer will provide inputs during the process.

C4.8.2 Level of geotechnical reporting

The level of geotechnical reporting should be proportional to:

- The complexity of the building, ground conditions and geohazards, and to
- the significance of the geotechnical contribution to the building's performance i.e. building category; structurally dominated, interactive or geotechnically dominated. (Refer to Section C1 and C4.3.4).

C4.8.3 Staged reporting and content

To align with the collaborative and iterative assessment process consideration should be given to staged reporting. Section C4.4 outlines the proposed assessment process and for each stage it outlines under the heading "possible deliverable" the possible reporting at that stage. These possible deliverables are listed below. For further details including proposed content of these reports refer Section C4.4.

- Desktop study report
- Factual geotechnical report, and
- Interpretive geotechnical report.

Alternatively, the content of these three reports could be combined in a single report.

C4.8.3.1 Interpretive geotechnical report content

The proposed content of the desktop study report and the factual report are outlined in Sections C4.4.3.1 and C4.4.3.3 respectively. Further guidance on the content of the interpretive geotechnical report is provided here.

The interpretive geotechnical report should document the following:

- an outline of the purpose, scope and limitation of the assessment
- table(s) and cross section(s) as appropriate to describe the inferred ground model. Highlight uncertainties in the inferred model.
- a list of geotechnical issues (geohazards) identified. Categorise these as:
  a) originating from outside the building footprint and thus not influencing the *%NBS* rating
  b) jointly agreed with the structural engineer that, because of the soil and structure's expected behaviour, are not likely to be critical to the assessment of the *%NBS* rating
  c) specifically assessed.
- the assessed category of the structure (structurally dominated, interactive or geotechnically dominated) and the basis of this conclusion.
- outline of geotechnical analysis and assessment undertaken (expect this to be limited to c) above)
- geotechnical parameters recommended to be adopted by the structural engineer in analysis and assessment
- the significance of any identified geotechnical issues originating from outside the building footprint (i.e. not considered in the assessment of the %NBS rating)
- any further recommended investigation/analysis/monitoring, and
- risks and uncertainties.

Note:

In line with Section C1, communication of the seismic risk and the assessed seismic behaviour of the building is a very important part of the DSA process. The written report should be carefully written to suit its intended audience.

%NBS score or rating is a measure of the assessed performance of the structure which will be reported by the structural engineer. Geotechnical reporting should not include %NBS. Where reporting of geotechnical capacity in terms of resistance to shaking is required (e.g. triggering of liquefaction or slope movement) this is to be reported in terms of "%ULS shaking" (Refer section C4.2.7).

The level of geotechnical input and reporting required for interactive and geotechnically dominated categories of structures (refer C4.3.4) will be more than that for structurally dominated structures.

C4.8.4 Peer review

Peer review requirements should be discussed with the structural engineer. Suggested situations where geotechnical peer review might be considered are summarised in Table C4.3. The peer reviewer's comments and the engineer's responses should be summarised separately and appended to the geotechnical report.

The paper, *Guidance for Commissioning and Undertaking Reviews of Seismic Assessments* (https://design.resilience.nz/resources/view/jc-25-02-guidance-for-commissioning-and-undertaking-reviews-of-seismic-assessments) (JC-Sar 2025) provides more information on peer review for seismic assessments, including guidance on scoping of a review. That document subdivides reviews by scope into:

- High level review
- Targeted review, and
- Full technical review.

Important aspects to consider as part of scoping and undertaking a geotechnical review include:

- Scope of geotechnical input and the adequacy of this scope
- Dealing with uncertainty, and
- Geotechnical step change including whether or not it is likely to lead to a SLSH.

Table C4.3: Situations where peer review might be considered

Case	Geotechnical peer review recommended
Structurally dominated building (in the absence of any other considerations described below)	X
Interactive building (in the absence of any other considerations below)	X
Interactive building IL4*	✓
Geotechnically dominated building IL4	✓
Probabilistic seismic hazard assessment and/or site response analysis	✓
Studies that provide geotechnical input to multiple structures simultaneously	✓
Studies that define geohazard risks for multiple sites; e.g. regional liquefaction, tsunami, rockfall studies	✓

Case	Geotechnical peer review recommended
Studies where the outcome of the structural assessment is sensitive to one or more of the following: • soil-structure interaction • Geotechnical step change • geophysical investigations • numerical modelling • time-history analyses	✓
Note: * IL = Building importance level as defined in AS/NZS 1170.0:2002	

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Appendix C4A: Modelling of SSI Effects

C4A.1 General

This appendix outlines some general principles of soil-structure interaction (SSI) and discusses various analysis techniques available.

SSI can be assessed by a range of techniques with varying degrees of complexity, it is recommended that more sophisticated methods are applied in separate soil-foundation analyses, and the load-displacement behaviour be quantified and represented with simple springs in the structural model. Where separate soil-foundation models are used it is important to clearly define the interface between the soil-foundation model and the structure model, e.g. is it at the pile head or are the structural engineers modelling the pile itself in which case it is at the pile shaft/soil interface.

This appendix outlines the following techniques, listed below in order of increasing complexity:

- simplified hand analysis to evaluate influence of ground
- point springs
- spring-bed (Winkler) foundation model
- direct finite element modelling.

For most assessments only the simplified techniques will be required (see model validity comments in Table C4.4 below). If the more complex methods are to be used this should be only if:

- a more simplified method has been applied first to develop an understanding of the likely SSI effects
- the assessment of the simplified analysis indicates that more complex analysis will be beneficial in better understanding the structure's behaviour and meeting the overall objectives of the project, and
- adequate investigation and assessment has been undertaken to define geotechnical and structural input parameters to a level detail consistent with that of the analysis.

It is important to note that the more typical structural engineering approach, which is to adopt a fixed base model for the interface between the structure and the ground, can often lead to a conservative solution for the structure. It assumes that a fixed base translates to a shorter first mode period of vibration for the structure and a higher lateral load from design spectra than would be obtained if flexibility was introduced at the base. While this may be true in many cases, in others it can lead to an invalid result (e.g. NIST 2012a and NIST 2012b).

For example, overestimating the restraint available at the base of a column founded on shallow pads may provide an erroneous idea of the bending moment profile in the column and underestimate the deformations in a lateral load mechanism. Equally, assuming a rigid base under a wall may miss the potential for "foundation uplift/wall rocking" and the resulting effects.

Additionally, there is potential for the building response to be underestimated due to ignoring a possible resonance effect with the ground that is not sufficiently allowed for by the choice of the specified subsoil classification. Multi-storey buildings located on deep soil sites provide an example of this.

C4A.2 Key Principles

In carrying out SSI modelling, the first goal is to understand the sensitivity of the expected response to the various assumptions around the soil and foundation. Parametric analyses to cover uncertainties in soil load-displacement characteristics will generally be required.

When assessing seismic performance both the structural and geotechnical engineers need to recognise and accommodate the potential for nonlinear behaviour of the structure, foundations and the ground. General principles to work by include the following:

- The ground's behaviour is poorly represented by unique parameter values with uniform distributions (e.g. linear springs).
- With close collaboration, the possibility of misinterpretations and abuse of numbers (e.g. spring stiffness, modulus of subgrade reaction) can be significantly reduced and possibly averted. Two effective measures to avoid the risk of misinterpretation are:
  - for geotechnical engineers to provide force/displacement relationships (springs) directly at the locations/spacings/set out that the structural engineers require; e.g. a schedule of pile springs at predetermined lengths along a pile. This avoids the potential for conversion errors from, say, subgrade modulus to springs that might arise if undertaken by the structural engineer.
  - for a reality check of force/displacement outputs performed by geotechnical engineers after structural analysis to verify correct interpretation.
- An iterative process between structural and geotechnical designers has to be established, as soil behaviour is nonlinear and spring stiffness depends on load.
- SSI should consider soil stiffness at the upper range and at the lower range of possible values as assessed by the geotechnical engineer.
- Soil stiffnesses considered are to be those which relate to the short term and magnitude of the seismic loading.
- Serviceability deflections are often critical for the design of new structures but not for the assessment of existing structures.
- Cost and time are associated with more rigorous analysis methods. Therefore, simplified methods should be applied first to develop an initial understanding of behaviour and the likely benefits of further more complex analysis. Complex analysis should only be

embarked on when the cost can be justified in terms of improved understanding of behaviour and outcomes for the overall project.

C4A.3 SSI Modelling Approaches

Complex SSI analysis including direct nonlinear modelling of the soil and its interaction with the structure is possible and may be warranted in some situations. Table C4.4 provides some further guidance on when to use the next level of sophistication of SSI modelling for pushover based structural analysis. Additional limitations should be considered for other analysis options (e.g. dynamic analysis). For further information on each of the SSI analysis options is provided in the following sections. However, in general, specific guidance on such analyses is outside the scope of these guidelines and reference will need to be made to other documents; (e.g. NIST 2012a, NIST 2012b and FEMA P-2091 2020).

Table C4.4: SSI analysis options for pushover analysis

SSI analysis option	When to use/not to use	Comments
Fixed base model – no SSI consideration	This should not be used for high rise buildings on piles or slender wall systems with shallow foundations.	The foundation structure will still need to be assessed by hand: • global overturning stability • yielding at the soil-foundation interface.
Hand calculations	Obtain initial estimates of influence of SSI for all SSI problems	Consideration of moment-axial load interaction and contributions from foundation substructure can be difficult to incorporate with hand-calculation methods and requires some rational evaluation of their effects.
Point springs	In most cases, except in cases of high moment-axial load interaction for shallow foundations, and high shear-moment interaction of piles.	Applied under a rigid foundation element, or to replace a flexible foundation on soil but combining the total load-displacement of both. The choice of spring stiffness (and strength) should reflect the load-displacement range of interest.
Spring-bed (Winkler) foundation model	Shallow foundations, particularly mat foundations Core walls Basement/part basements Deep piles	Requires many springs ~20 per footing, or ~1 m spacing for a pile. Can capture flexibility of the foundation as well soil deformations. Spring stiffness can vary based on distance to edge, as well as changes in foundation size and loads. Can capture moment-axial load interaction for shallow foundations (both uplift and soil yielding if yielding compression with no tension springs are used). The selection of stiffer springs to capture both rotational and vertical stiffness is difficult for some foundation aspect ratios.

SSI analysis option	When to use/not to use	Comments
Advanced geotechnical SSI analyses (e.g. nonlinear finite element analyses)	Where ground deformations are potentially critical and significant, e.g. behaviour of high rise buildings adjacent to a tunnel or steep slope.	Can model complex ground conditions with buried infrastructure. There needs to be a robust process for interlinking the advanced/complex finite element ground model behaviour with the global structural models.

Note:

Irrespective of the SSI modelling approach adopted, sanity checks of complex model situations (such as the type indicated in Table C4.4) by approximate calculation and a simplified ground model are essential. The variable nature of the soil and the way in which the building interacts with it means that analysis runs to investigate the sensitivity of the results to the modelling parameter will almost certainly be required.

If SSI behaviour provides a beneficial influence to the structural performance (e.g. period elongation) the SSI analysis and geotechnical considerations should be cautiously appraised and also subjected to appropriate peer review.

C4A.3.1 Hand calculations

Hand calculations allow expected displacements to be approximated without the additional complexity of including a foundation model. A simple approach is to take the fixed base loads and apply those loads to the load-displacement curves in Section 4.6.4 to obtain displacements, then evaluating whether those displacements would negatively impact the structure.

Additionally SSI effects can be considered within the Simple Lateral Mechanism Analysis (SLaMA) assessment of the superstructure (described in Section C2) using hand calculations (e.g. Millen et al. (2020)) that can indicate whether an inelastic SSI mechanism may occur, and whether SSI flexibility matters to the overall assessment.

If SSI effects are considered to be negligible to the overall building response or the fixed-based analysis is sufficient, no further SSI analysis is required.

A simplified SSI analysis can be undertaken with upper and lower probable geotechnical parameters to determine the most adverse consequences from the probable range of deformations resulting from ground behaviour (e.g. range of foundation flexibility due to pile tension uplift) and step change scenarios (e.g. differential settlements due to liquefaction occurring or not occurring). A desktop-based geotechnical assessment may be sufficient for this.

Due to the simplicity and coarseness of this approach, the engineer should undertake relevant sensitivity analyses and consider the likely effects of the simplifications. The cost and benefit of more complex analysis needs to be considered before embarking on such analyses.

Benefits in terms of improved understanding of behaviour and outcomes for the overall project need to be considered.

In many cases more complex analysis of SSI will not be necessary.

C4A.3.2 Point springs

Point springs are recommended for foundation model in the structural model. They define the load-displacement behaviour for each mode of displacement of the foundation (e.g. vertical, rotational), see Figure C4.6(a). Point springs are highly interpretable in that the behaviour is prescribed by a backbone curve, which can be linear, bilinear or multilinear. The engineer should check that the point spring backbone curve represents the expected load-displacement behaviour of the soil-foundation interface (see Section C4.6).

C4A.3.3 Spring bed (Winkler) foundation model

This modelling approach consists of distributed springs that represent the resistance of the soil across the soil-foundation interface. This approach is appropriate for both shallow and deep foundations (refer to Figure C4A.1). The advantage of this modelling approach is that it can directly model interactions between vertical load and moment for shallow foundations, or shear load and moment for deep piles, they can also capture the flexibility of those elements.

Winkler springs can either be used as the foundation model in the structural model, or used in specific soil-foundation analyses to quantify the load-displacement behaviour that can then be represented by calibrated point springs. Soil springs can also be incorporated easily into the analysis tools used by most structural engineers.

Winkler springs typically requires many springs (~1 m spacing for piles or ~20 per footing). In some analysis packages, line or area springs can be applied.

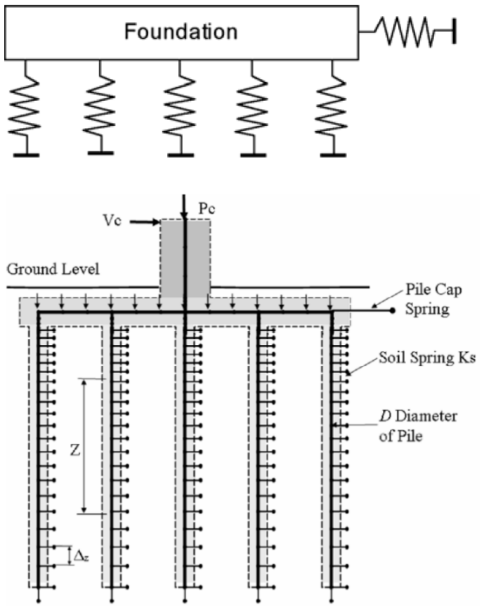


Figure C4A.1: SSI model for flexible base model using Winkler spring for shallow foundation and deep pile foundation

C4A.3.3.1 Linear springs

Key issues to consider for **shallow foundations** are:

- The use of linear springs is not suitable in situations where the foundation would experience significant uplift, or a significant area of the soil below the foundation has reached its bearing capacity.
- The definition of linear soil spring modelling parameters requires the geotechnical parameters (soil shear modulus and Poisson's ratio). In absence of robust geotechnical data, values can be used to initially test the sensitivity of the parameters (e.g. Oliver et al., 2013).
- The pressure distribution through soils beneath a raft foundation influences the equivalent spring stiffness; i.e. a larger area of loading results in a greater depth of influence and greater settlement (softer springs). This can be addressed by iterations between geotechnical and structural analysis:
  - The geotechnical engineer provides the first estimate of spring stiffnesses.
  - The structural engineer applies these to analysis and reports back to the geotechnical engineer the assessed pressure distribution and settlement distribution.
  - The geotechnical engineer applies the pressure distribution to the surface of the 3D soil model and calculates settlements. Pressures are divided by settlement to give updated spring stiffnesses to be reported to the structural engineer.
  - These iterations are repeated until the pressure/settlement calculated by the structural and geotechnical models converge.

Key issues to consider for **deep piled foundations** are:

- Deep piled foundations can be idealised using a series of uncoupled vertical axial springs along the length of the piles and pile caps being considered as a rigid element.

- Secant stiffness parameters (based on p-y curve at the expected lateral deformation) should be used for elastic analysis.
- Soil spring parameters for the piles spring can be determined using hand analysis (elastic analysis and Brom's method) or by specialist geotechnical analysis software based on nonlinear p-y curve of the soil layers.
- Adding detailed piles and soil springs into the global structural analysis can result in significant numerical complexity to the model, even for a linear analysis. It is common evaluate the behaviour with a specific soil-foundation analysis and capture the behaviour as point springs for the foundation model in the structural analysis.
- In some scenarios with significant nonlinearity expected in the piles (e.g. piles with a liquefiable layer), a pseudo static nonlinear analysis is more appropriate.

C4.8.4.1 Using compression-only or tension-only Winkler springs

The use of linear Winkler springs is no longer appropriate when the spring goes into tension, as the soil's tensile capacity is generally negligible (unless ground anchors or piles are provided). Using an iterative process, the soil springs in elastic models that are subject to tension forces can be progressively "deactivated" from the model in order to reach an acceptable equilibrium state. This, in effect, allows the shallow foundation to uplift.

If nonlinear analysis methods are used (nonlinear pushover or time history), foundation uplift and soil yielding can be explicitly modelled using compression gap elements and nonlinear springs with asymmetric capacity curves. If the analysis result is very sensitive to the nonlinear springs' parameters, a sensitivity analysis should be carried out. Due to the complexity and time involved, the sensitivity analysis can be carried out using a specific soil-foundation analysis (sub-assembly model).

Note:

The nonlinear modelling of rocking foundations can be complex resulting in erroneous results. The use of tension-only or compression-only elements in nonlinear dynamic analysis can result in "stiffness matrix spikes" and loss of energy from over-damping. The use of nonlinear contact elements may also lead to over-prediction of the damping and energy dissipation that results from the interaction between the soil and the foundation interface.

C4.8.4.2 Nonlinear soil springs

In some scenarios where the soil is near the ultimate capacity, and SSI has a significant influence on the seismic response of the building, nonlinear analysis of the SSI effects will be warranted.

There are a number of relevant articles in the literature on the modelling of nonlinear soil behaviour using bilinear or trilinear capacity curves with substructuring/indirect modelling for the purpose of pseudo static pushover analysis (NIST (2012) and Cubrinovski and Bradley, 2009).

Two approaches for shallow and deep foundations are illustrated in Figure C4A.2 and C4A.3 below.

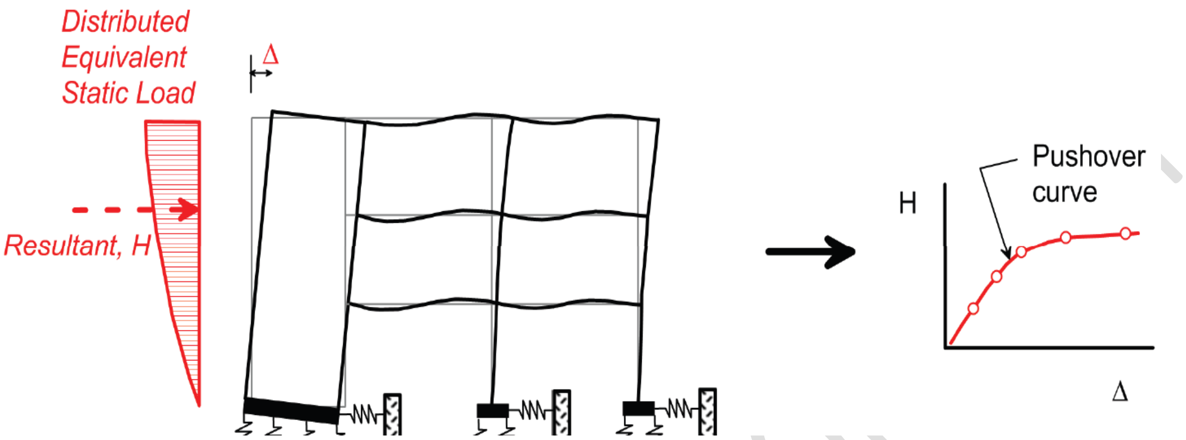


Figure C4A.2: Schematic illustration of a pushover analysis and development of a pushover curve for a structure with a flexible base (NIST, 2012a)

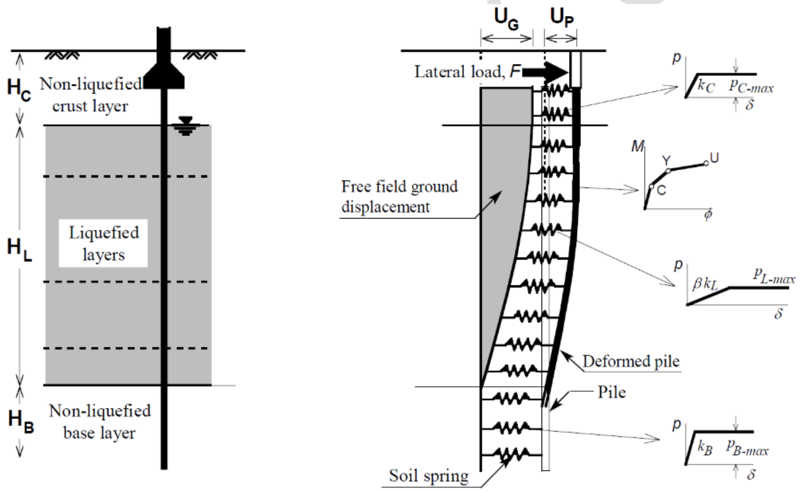


Figure C4A.3: Schematic illustration of a pseudo static pushover analysis and development of a pushover curve for a bridge pier with a flexible pile base (Cubrinovski and Bradley, 2009)

C4A.3.4 Direct finite element modelling

It is possible to undertake a direct simulation of the SSI and the nonlinear responses of the soil and structure using a direct approach, in which the entire SSI system is analysed in a single model/step. SSI using a direct analysis approach can be performed using finite element (FE) computer programmes. Figure C4A.4 shows an example of such analysis.

There are a number of technical challenges related to the use of a direct analysis approach, including the definition of critical input parameters (e.g. a constitutive model for various soil types), the geotechnical information of the underlying soil, the definition of boundary conditions, the modelling of a 3D foundation in a 2D plane strain problem (if using 2D modelling software), and the complexity of such a complex nonlinear model.

Methods of this level of complexity would only be considered in exceptional cases where a critical issue has been identified for a larger project requiring specific detailed analysis. Before undertaking a direct analysis approach:

- Separate, less complex analyses should be undertaken so the benefits of carrying out a direct analysis can be assessed and also to provide a check against the outputs of the direct analysis.
- Sufficient investigations should be undertaken to provide a level of detail in understanding the geotechnical and structural input parameters in keeping with the detail of the analysis.

There is a greater need for a rigorous checking of the input parameters and analysis assumptions for the FE model. Independent peer review of the inputs and outputs is recommended.

Note:

Cubrinovski and Bradley (2009) provides an example of the use of effective stress analysis using a direct approach for the analysis of piles in liquefiable ground.

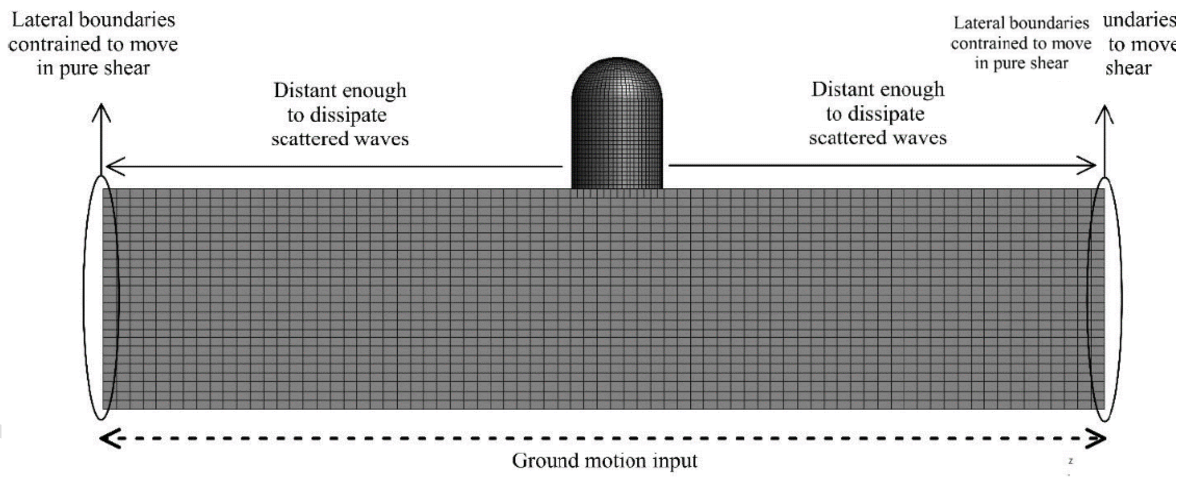


Figure C4A.4: Direct FE modelling (Cubrinovski and Bradley, 2009)

Appendix C4B: Assessment of Retaining Walls

(Supplement to NZGS/MBIE Module 6: Earthquake resistant retaining wall design)

C4B.1 Introduction

Retaining walls are often associated with, or even integral to, a structure under assessment. The assessment of retaining walls may require close collaboration between the structural and geotechnical engineer as these are loaded by, and typically derive their restraint from, the ground but may also contain elements that require structural input.

Note:

NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* provides relevant guidance. This appendix supplements that guidance with specific information relating to assessment.

C4B.2 Historical Performance

Observations made during the Canterbury earthquake sequence of 2010-11 provide a useful insight into the performance of existing retaining walls under seismic shaking. Refer to Appendix A of NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* for a commentary on observations from Christchurch. However, care should be exercised in extrapolating these findings to other walls and ground conditions elsewhere in New Zealand. Also, note that there were few, if any, instances of retaining wall performance during the Canterbury earthquake sequence affecting the life safety performance of buildings.

Note:

Other useful references include Anderson et al. (2015) and Kendall Riches (2015).

A number of aspects of retaining wall design contribute to better than expected earthquake performance when walls are apparently loaded beyond their design capacity. In general terms, there is conservatism in static design methods and in simplifications of pseudo static design methods. In addition, there is the typical robustness of retaining walls.

Where appropriate these aspects (listed below) should be considered while undertaking an assessment of an existing retaining wall:

- the use of strength based design, where wall displacement could have been used to limit seismic loads in the design
- the use of elastic design for wall elements where ductility might be acceptable
- use of the Mononobe Okabe (M-O) equation

- assuming $c = 0$ (cohesion of the soil) to derive loads on a wall supporting ground, but with the shear strength actually due to both c and ϕ (friction angle of the soil)
- considering sloping ground behind the wall where an unrealistically large seismic active earth pressure coefficient was assumed in design
- assuming homogenous soil properties in design, but where actual strength properties increased with depth/distance from the wall but were not taken into account over the extent of theoretical slip; or design was based on the weakest material and/or characteristic (i.e. conservative) parameters
- adopting unrealistically high active earth pressure values for cases with high seismic accelerations or steep back-slopes, and
- ignoring wave scattering and dynamic effects for calculation of seismic pressures on high walls.

Note:

NCHRP, 611 (2008) states: "The overall performance of walls during seismic events has generally been very good, particularly for mechanically stabilised earth (MSE) walls. This good performance can be attributed in some cases to inherent conservatism in the design methods currently being used for static loads".

C4B.3 Identification of Retaining Walls requiring Assessment

C4B.3.1 General

A retaining wall will only need to be assessed if its performance could affect the ability of the structure being supported to meet its own performance criteria.

Accordingly, the focus of any retaining wall assessment should be on the consequence for the supported structure. Even if it indicates that the wall is at risk of "failure" under the earthquake shaking considered, this failure is only considered consequential if it results in:

- the structure not meeting life safety performance criteria, or
- loss of emergency egress from the structure.

In the context of the life safety assessment of existing buildings, the behaviour of supporting retaining walls will often not be the governing issue for the performance of the structure. The following questions are suggested for initial consideration:

- Is there a significant risk that the wall may be of low capacity? (For example, it is a historic stone/masonry wall with no redundancy, or liquefaction is likely.) If yes, then assess the consequences for the structure's performance on the assumption that the wall may fail.

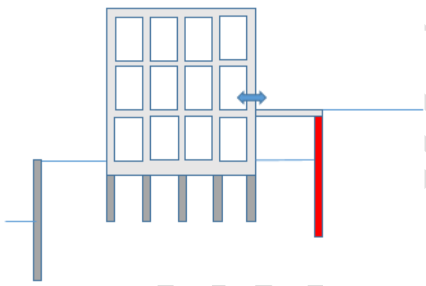
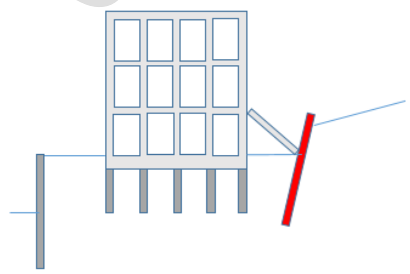
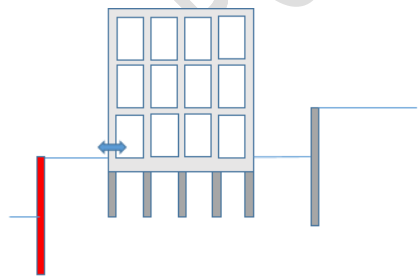
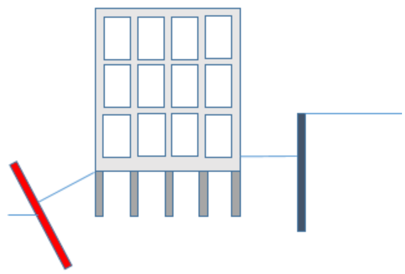
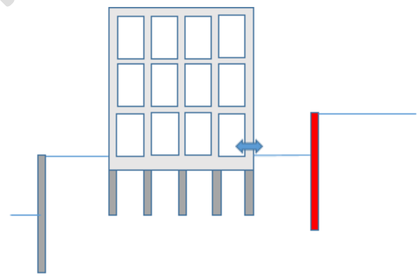
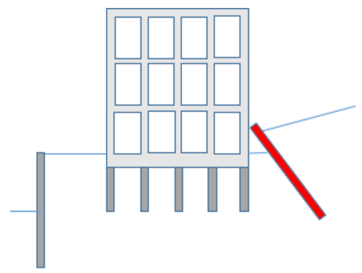
- Is there a significant risk of excessive (e.g. > 200 mm) horizontal displacement? (For example, it is a historic mass concrete gravity wall with an undersized foundation.) If yes, then assess consequences for the structure's performance.
- Can the structure tolerate horizontal wall displacement of 100 mm? If no, then assess in more detail.

The retaining wall's performance should be considered across a spectrum of earthquake demand (XX%ULS). There are a number of mechanisms by which a retaining wall can impact on structural seismic performance. Some examples are presented below.

C4B.3.2 Loss of emergency access/egress to the building

Table C4B.1 gives some examples where poor performance in a wall may impact on emergency access/egress and hence on the building's earthquake rating.

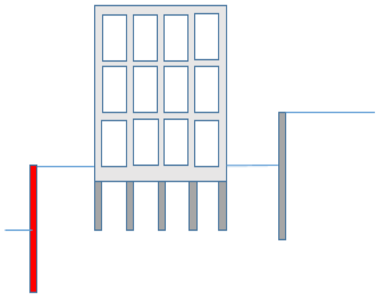
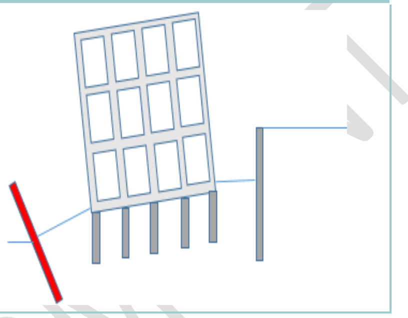
Table C4B.1: Examples of impact on emergency access/egress

Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall supporting structure required for building egress		
Instability in a retaining wall supporting ground that provides building egress		
Instability in a wall supporting ground above a building egress		

C4B.3.3 Loss of support to foundation soil

Table C4B.2 gives an example where poor performance in a retaining wall providing support to the building foundations may impact on the building's earthquake rating.

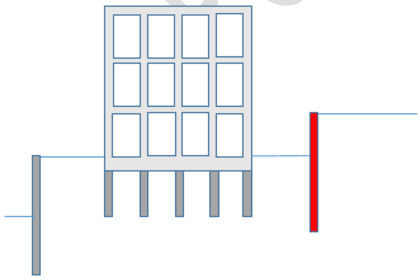
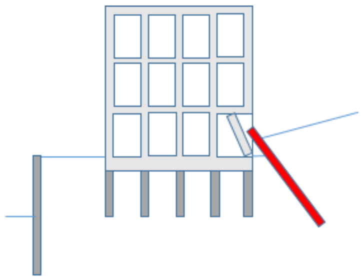
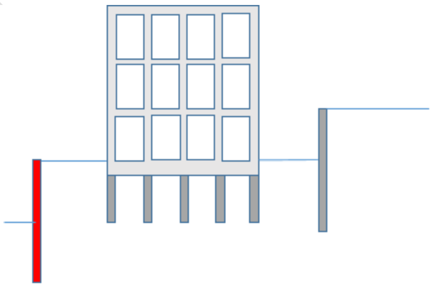
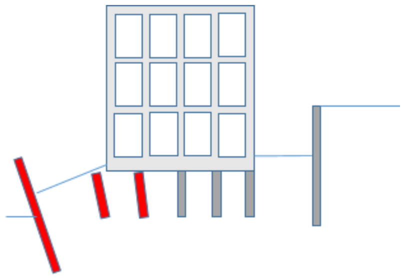
Table C4B.2: Example of loss of support to foundation soil

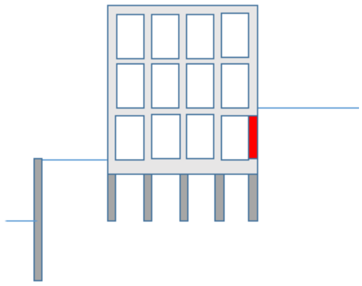
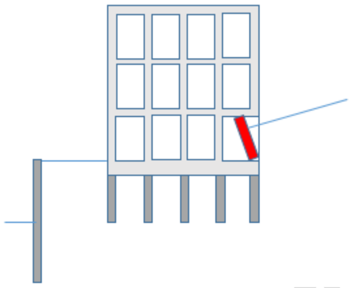
Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall below building foundations removing vertical support		

C4B.3.4 Lateral loading or deflection of a key building element

Selected examples where poor performance of retaining walls that may result in excessive increased lateral loading or reduction in lateral support and, in turn, may impact on the building's earthquake rating are shown in Table C4B.3.

Table C4B.3: Examples of lateral loading or loss of lateral support to foundation soil

Mechanism	As designed	Potentially unacceptable performance
Instability in a retaining wall impacting on building. Does **not** influence the earthquake rating of the building.		
Instability of a retaining wall generating lateral loading on foundations supported at a deeper level		

Mechanism	As designed	Potentially unacceptable performance
Instability in a basement retaining wall		

C4B.4 Modes of Deformation and Methods of Assessment

Refer to NZGS/MBIE Module 6 - Earthquake resistant retaining wall design for information on the modes of deformation to be considered and methods of assessment for various types of retaining walls.

Appendix C4C: Slope Instability Hazard

(Supplement to NZGS Slope Stability Guidance Series)

C4C.1 Introduction

The NZGS Slope Stability Guidance Series (Section 7.8 of Unit 1 and all of Unit 4) provides guidance on seismic stability of slopes. Slope stability assessment in the context of seismic assessment of existing buildings requires special consideration of the potential impacts of instability or slope displacement on the building. Figure 7.3 of Unit 1 provides a useful summary of the process for seismic stability analysis (noting that the seismic demand in assessment of buildings will be defined as %ULS and require assessment at a number of percentages of ULS seismic demand).

C4C.2 Examples of seismically induced slope stability

Examples of circumstances in which seismically induced slope instability may be an issue include:

- where there is a history of slope instability or a geomorphology that is indicative of historic instability
- when there is no evidence of historic instability but the topography, geology, groundwater conditions and seismic conditions are such that instability is possible
- steep slopes (greater than 35º), such as gorges and cliffs where rockfalls are common
- slopes that have been altered, such as cuttings along roads and quarries, or where vegetation has been removed
- underlying weathered or shattered rocks that weaken the slopes
- soils that have liquefaction or cyclic softening potential with sloping ground or a nearby free face
- active landslides or old landslides that might start moving again, and
- in the vicinity of active fault scarps.

Note:

This list has been adapted from the AGS Practice Note Guidelines for Landslide Risk Management, 2007.

C4C.3 Assessment Process

Stage 1 – Initial assessment of stability

A great deal of information on slope stability can usually be obtained via desk study and/or site inspection by a suitably experienced person. Input and review by an engineering geologist is recommended.

It is recommended to start with a natural scale sketch of the system model: the ground, the foundations and the structure. ASCE 41-17 (2017) Clause 8.2.2.4 is a useful guide for screening purposes.

Engineers are referred to geohazard assessments that have been carried out for territorial authorities and regional councils to identify the potential hazards that are likely to be appropriate for the site in question. These are typically in the form of hazard maps. There may also be specific slope hazard reports in urban areas. Additional guidance on this desk study is included in NZGS/MBIE Module 2.

Stage 2 – Site inspection

Input by an engineering geologist is recommended during the site inspection and associated reporting. Relevant geohazard information that is obtained from a walkover of the site, desk study of geohazard references and local knowledge can be combined in a site inspection report. This should include the following information:

- a brief description of the site shape, size, geological profile (refer to maps and memoirs), expected site subsoil class, terrain, vegetation, springs, erosion features, evidence of slope instability on site and on adjoining site(s), where relevant. Comment on depth to groundwater and seasonal fluctuation, if known.
- a description of how the building sits in relation to the site (e.g. with reference to an annotated aerial photo). Comment on proximity of the building footprint to slope edges, slope height and proximity to water courses/river banks (these details are relevant in terms of seismic slope stability and also for potential lateral spread hazard), and
- a description of geohazard sources located outside the site boundaries that could impact on building performance. This is particularly relevant for slope instability uphill of the site or retaining walls on adjacent property.

Stage 3 – Site investigation

If a site investigation is required the site-specific scope should be determined. A CPEng geotechnical engineer or PEngGeol engineering geologist should be engaged for scoping and management of a site investigation. Refer NZGS/MBIE Module 2.

Stage 4 – Analysis

Jibson (2011) provides a useful overview of methods for assessing the stability of slopes during earthquakes, including a list of useful references.

Jibson (2011) describes three families of analyses for assessing seismic slope stability as follows, with each having its own appropriate application:

- **Level 1 – Pseudo static analysis**
  - only suitable for preliminary or screening analyses because of its crude characterisation of the physical process

- **Level 2 – Permanent deformation analysis**
  - a valuable middle ground between a Level 1 and Level 3 analysis
  - simple to apply and provides far more information than pseudo static analysis
  - rigid-block analysis suitable for thinner, stiffer landslides, which typically comprise the large majority of earthquake-triggered landslides
- **Level 3 – Stress deformation analysis**
  - best suited to large earth structures such as dams and embankments, as it is too complex and expensive for more routine applications
  - coupled analysis is appropriate for deeper landslides in softer material, which could include large earth structures and deep landslides
  - modelled displacements provide a useful index to seismic slope performance and should be interpreted using judgement and according to the parameters of the investigation.

Note:

Refer to Barbour and Krahn (2004) for insights and guidance on numerical modelling.

C4C.4 Defining Seismic Accelerations for Slope Stability Analysis

Refer to NZGS/MBIE Module 1 (2016) - *Overview of the Guidelines.*

Ground shaking can be subject to significant amplification near the crest of steep slopes and ridgelines, such that PGASITE can be significantly greater than a PGA determined via NZGS/MBIE Module 1 (2016). NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design*, MBIE (2021) and Eurocode EN 1998-5:2004 provide information on topographic amplification factors.

Appendix C4D: Seismic Performance of Foundations

(Supplement to NZGS/MBIE Module 4: Earthquake resistant foundation design)

C4D.1 Introduction

NZGS/MBIE Module 4 - Earthquake resistant foundation design provides guidance relevant to the assessment of foundations. This appendix supplements that guidance and provides specific information relating to seismic performance of existing foundations and observations from the Canterbury earthquake sequence of 2010-11 and other earthquakes. A description of foundation types historically used in New Zealand and their strengths and weaknesses is also provided.

Following the Canterbury earthquakes, liquefaction-induced ground failure did not result in any direct fatalities in Christchurch's central business district (CBD) despite the widespread damage to residential and commercial buildings (Cubrinovski and McCahon, 2012; Murahidy et al., 2012). However, rockfall and landslides at the fringe of the city resulted in five fatalities (Dellow et al., 2011).

A similar conclusion can be drawn from the 14 representative buildings studied by the Canterbury Earthquakes Royal Commission (CERC Vol 2, 2012). While ground failure (e.g. liquefaction) and foundation damage were observed at a number of sites (e.g. the Town Hall, police headquarters, and 100 Armagh St Apartments), these buildings have generally satisfied the life safety performance required by the New Zealand Building Code.

As a general observation of building performance in Christchurch, if the superstructure was robust (well-tied together), integral and responding in a ductile manner, foundation failure exacerbated the inelastic demand on the superstructure's plastic hinges but did not necessarily result in a uncontrolled displacement response.

C4D.2 Shallow Foundations

Foundation elements are considered to be shallow when the depth to breadth ratio is less than 5 (i.e. $D/B < 5$). Some behaviours of shallow foundations to be considered in assessment are outlined below.

Some foundations have suffered from non-uniform aspects such as basements under only parts of the building, irregular footprints with differential movements in plan, or piles installed to provide tension capacity under only parts of a shallow foundation. Particular attention should be given to the areas around such features in looking for damage, differential movement, etc. A number of buildings have suffered differential movement due to uplift of basements under part of the ground floor.

Basements can be exposed to high uplift pressures generated in liquefied sands or in loose gravels. This can result in vertical displacement as well as damage to the basement floor, depending on the construction as a raft or slab between footings or piles. Uplifted basements, particularly those on gravels rather than liquefied sands, may have large voids below them. Basement walls may have been subjected to lateral earth pressures much higher than normal static loading. Many basements were partially flooded after the 22 February 2011 Canterbury earthquake because of damage to walls, floor or tanking.

Gapping has occurred adjacent to footings as a consequence of cyclic lateral displacement during the shaking.

Where rocking of foundations has occurred (or is suspected to have occurred) gaps may have developed underneath foundation elements or under the edges of elements.

C4D.3 Deep Foundations

Foundation elements are considered to be deep when the depth to breadth ratio is greater than or equal to 5 (i.e. $D/B > 5$). Some behaviours of deep foundations to be considered in assessment are outlined as follows:

- Common issues for deep foundations that need to be considered include the loss of side resistance (skin friction) in piles, which may occur from pore water pressure increase during shaking, even if full liquefaction does not trigger. Where full liquefaction is triggered at depth all side resistance above may be effectively lost or reversed because of settlement of the overlying strata. In such cases, so-called "negative skin friction" may contribute to pile settlement.
- Unless they are adequately embedded in dense soils, bored cast-in-place piles are perhaps the most susceptible to settlement caused by pore water pressure rise and liquefaction above the base of the pile, because the gravity loads are carried initially almost entirely by side resistance. If this mechanism is overloaded, the pile will settle until the end bearing mechanism is mobilised (which could be as much as 5 to 10 percent of the pile diameter). This can potentially be exacerbated if poor construction has left a zone of disturbed material at the base of the piles.
- Cyclic axial loading during the earthquake may cause loss of capacity and settlement, especially for piles that carry only light gravity loads and rely mainly on side resistance. This is discussed in more detail in Section C4.6.9.
- Pile settlement may also be from liquefaction of sand layers below the founding layer. For example, many parts of Christchurch have dense gravel or sand layers that may be several metres thick but underlain with much looser sands. Deeper liquefaction may not have been considered in the pile design, particularly of older buildings.
- Damage to foundations may not always be evident from the surface, particularly where a large area has been subject to lateral displacements. Where there is evidence of relative motion between the structure and the ground, pile heads and the connection to the structure should be checked for overload in shear. Shear transfer from the ground to the building is typically assumed to be carried by friction underneath the building and by passive resistance of the soil against buried foundation beams and walls, etc. The friction mechanism will typically fail quickly with any settlement of the ground and the passive

mechanism degrades rapidly with development of gapping. For this reason, and because the earthquake shaking was stronger than design levels, it is likely that the piles may have carried far more shear than the designer ever intended.

- Kinematic interactions between the ground and the piles need to be carefully considered. Ground deformations are known to have been significant around many parts of Christchurch, including both dynamic and permanent deformations. These ground deformations may impose significant strains within piles resulting in pile damage and permanent deformation well below the ground surface. Physical investigation of such damage is difficult and expensive and may be impractical. Analytical procedures are available as a first step to try and estimate the pile strain levels and therefore likelihood of damage.

C4D.4 Soil-Structure Interaction

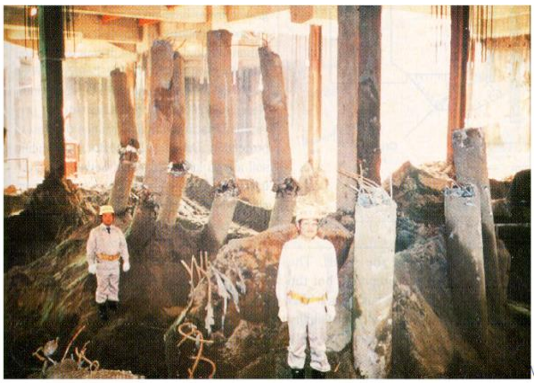
Reconnaissance reports of past earthquakes confirm that the seismic performance of buildings can be significantly influenced by the geotechnical performance of the supporting ground. Buildings have collapsed or been significantly damaged due to either foundation (shallow or deep) “failure” and/or liquefaction-induced settlements. Similarly, there are buildings that could have collapsed but have not done so due to the beneficial effect of SSI.

Figure C4D.1 shows overseas examples of (a) building collapse and (b) brittle pile shear failure, both as consequences of ground liquefaction and foundation failure from the 1964 earthquake in Niigata, Japan. Both mechanisms would not have been identified by an engineer undertaking a simple pinned/fixed-based structural analysis. It is noted the level of understanding of liquefaction risk was minimal at the time of this earthquake.

The building in Figure C4D.1(b) remained in service for 20 years after the earthquake despite the hidden shear failure of the piles, illustrating the difficulty in predicting foundation performance and identifying foundation damage post-earthquake (Yoshida and Hamada, 1990).

Photo by Joseph Penzien, courtesy of NISEE

(a) Niigata 1964 – tilt of housing blocks due to liquefaction-induced bearing capacity failure



(b) Pile shear failure observed in an excavation 20 years after the Niigata 1964 earthquake

Figure C4D.1: Significant building damage and collapse due to ground failure (Yoshida and Hamada, 1990)

There are several notable examples where the geotechnical foundation system's step change behaviour led to a brittle failure mode in the substructure and superstructure.

Figure C4D.2 illustrates the example of a five storey building damaged in the Christchurch earthquake of 22 February 2011 (Kam et al., 2011). The site (in Madras St, central Christchurch) showed evidence of moderate liquefaction surface manifestation.

The foundation of the core wall on the southern elevation lost its bearing capacity, possibly during or after the earthquake event, and the wall had settled about 450 mm vertically. The settled core wall appeared to have pulled the floor slab and the rest of building towards it. The external ground beam connected to the wall, and a number of frame beam-column joints had failed in a brittle shear mechanism (refer to Figure C4D.2(c)) which is likely to be a consequence of both seismic shaking and induced vertical displacement demand from the wall's foundation failure. The building's lateral load system was severely compromised due to the foundation-wall system failure and it partially collapsed in a subsequent aftershock.

(a) Plan | (b) South east elevation | (c) Shear failure of ground beam connected to shear wall

Figure C4D.2: Five storey building with shallow foundation failure beneath core walls (adopted from Kam et al., 2011)

Figure C4D.3 below presents several examples of significant building residual deformations due to foundation "failure" observed in the Christchurch CBD (Kam et al., 2011).

(a)-(b) 1980s high rise on basement and raft foundation; with beam plastic hinges observed throughout the building

(c) 1980s low rise on shallow foundation with significant differential settlement and sliding movement

Figure C4D.3: Building foundation "failure" (Cubrinovski and McCahon, 2012

C4D.5 Information on Foundation Types used in New Zealand (Potential Strengths and Weaknesses)

Table C4D.1 below summarises the foundation types likely to be encountered in New Zealand buildings, together with their likely strengths and weaknesses.

Note:

This information is for general guidance only. Each site and structure should undergo site-specific engineering assessment.

Table C4D.1: Summary of traditional foundation types

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
Driven timber piles				
Driven timber tip armoured	1890 -	Round poles top driven to a set	• Durable when quality hardwood used, especially when submerged • Consistent capacity	• Degradation/rot, especially at top • Poor engagement into foundation
	1890 - 1920	End tapered and protected with steel to penetrate stiffer layers		
Driven steel piles				
I or H sections	Typical post-1970s	Commonly bare steel, sometimes galvanised or coated	• Consistent capacity • Could be driven through stiff layers • High shear capacity • (Can be) ductile in bending • Degradation less of an issue due to large area-to-surface ratio	• Rusting/degradation potentially very significant • Variable engagement into foundation
Tube/pipe	Typical post-1970s			
Railway irons	1890s -	Cast iron prior to ~1910		
Driven concrete piles				
Precast	1915-		• Base bearing capacity consistent • Side friction variable dependent upon installation technique, but should be calculated considering it as a displacement pile	• Shear failure. Existing piles often have few stirrups and can fail in a brittle manner during ground lurch or lateral spreading. • Franki/bulb piles are likely to have poor curtailment of reinforcement into the consolidated base, and so little tension capacity. They also may have “necked” shafts. • Top fixity: does this work in both directions? Is it truly fixed at the top?
Franki/bulb piles	1960s-1980s	Drilled pile, concrete poured at base and driven to provide consolidated end bearing and spread		
Driven precast plug	1970s-	Drilled pile with precast pile driven out through base		
Bottom driven steel tube	1980s-	Permanent steel tube liner driven by dropping a weight on a plug of dry mix concrete in the base of the tube. Reinforcing cage and concrete placed after driving		
Driven cast-insitu	1980s-	Driven tube with sacrificial steel base, casing withdrawn during casting		

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
Bored piles				
Straight	1860-	Multiple drilling techniques	• Very old (<1910) piles may have high quantities of non-Portland cement and hence be very durable • Often large robust sizes • Reinforcing easy to curtain into foundation beams • Be careful for distribution between skin friction and end bearing (relative stiffness and strength)	• Base cleanout quality critical for end-bearing dependent piles (esp. bells) • Shear may be critical for piles with fewer stirrups underground lurch or lateral spreading • Top fixity? • Belling quality (collapse)?
Straight grooved	1990-	Sides grooved with special tool after drilling		
Belled	1960-	Specialist technique		
Steel screw piles	1990-	Specialist technique	• Records correlate capacity with installation torque • Testing results should be available • Robust against shear • Small drag-down effect if liquefaction settlements in upper layers	• Helices very flexible: vertical displacements often govern for seismic loads (soil/structure interaction • Small contribution to base-shear resistance
Ground anchors				
Drilled and inserted	1960-	Drilled and grouted hole, bar or strand anchors	• High capacity can be installed in small space • Free length gives controlled plastic elongation if required • Testing records may be available • Can often be re-tested to prove capacity	• Poorer performance under cyclic load • Limited compression capacity: critical if building settles due to liquefaction • Little to no shear capacity: vulnerable to lurch or lateral spreading • Durability critical, especially around anchorages (esp. for both ends of deadman anchorages) • Potential "brittle" behaviour (reduced
Pressure grouted/drilled	1990-	Proprietary bar drilled specialist technique		
Deadman	18??-	Relies on steel bars back to mass or reinforced concrete passive acting blocks		
Mechanical expansion	1970-	Rock bolts with expansive ends		
Grout expansion	1990-	Proprietary grouted tubes which "unroll"		
Mechanical tip	1990-	Proprietary bearing engagement e.g.		

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
		"Duckbill/Manta Ray"		grout to country bond with strain)
Shallow				
Brick strip	1840-	Nominal widening, sometimes incorporating site concrete	• Predictable, well tested behaviour in "good ground" • Pads often oversized for older buildings • Rafts can mitigate differential displacement	• Affected significantly by liquefaction • Strip footings often undersized/highly stressed under brick walls • Pre-1930s footings may not have continuous reinforcement
Concrete strip	1840-	Reinforced or unreinforced		
Ground beam	1950-	Reinforced, likely spreading point loads		
Isolated pad caisson	1840-	Reinforced		
Raft	1970-	Reinforced		
Domestic				
Timber ordinary	1840-	Rounds or squares excavated and concreted in place	• Typically small loads per unit	• Degradation of timber with time • Often lack of distributed resistance • Ensure structure fixed to foundations • Shallow piles have little or no cantilever capacity
Timber anchor	1980-	Square excavated and concreted in place		
Timber driven	1960-	Round or square		
Concrete ordinary	1920-	Precast, sometimes cast in "kerosene tins"		
Concrete strip	1930-	Typical subfloor walls		
Brick strip	1860-	Single or two courses wide, sometimes in site concrete		

Appendix C4E: Liquefaction Assessment

(Supplement to NZGS/MBIE Module 3 - Identification, assessment and mitigation of liquefaction hazards)

C4E.1 Introduction

NZGS/MBIE Module 3 - *Identification, assessment and mitigation of liquefaction hazards* provides guidance on assessing susceptibility of soils to liquefaction. This appendix supplements that guidance and provides information on completion of initial screening for liquefaction and approaches in relation to seismic assessment of existing buildings.

C4E.2 Liquefaction screening

In the absence of nearby site investigation information for liquefaction assessment, the MBIE/MFE *Planning and engineering guidance for potentially liquefaction-prone land (2017)* can be used. Section 4.4.3 of the MBIE/MFE guidance provides several qualitative approaches for liquefaction and lateral spreading. Section 4.4.4 and Table 4.3 also provides semi-quantitative approaches.

Many local councils across NZ have developed liquefaction vulnerability maps MBIE/MFE (2017) to determine liquefaction vulnerability. These are often available within councils online maps.

C4E.3 Sites tested from historic earthquakes

Where there is sufficient information available, the calculated performance of the site can be compared with actual performance observed during historic earthquakes.

On sites which have reliable earthquake records (PGA, Magnitude) it may be appropriate to adjust typical liquefaction analysis parameters where calculated liquefaction damage indices (e.g. free field settlement, liquefaction severity number) do not correlate well with the observed damage. Given there are a number of uncertainties and parameters to represent these uncertainties it is important to understand why the calculated performance is different to what was observed, and a moderately conservative approach should be applied in this determination. Key considerations are:

- Groundwater level during the earthquake
- Susceptibility of the soils to liquefaction and the associated fines content
- Whether the soil conditions are well represented by the investigation information (for example CPTs may not represent the density of pumice well).
- The magnitude and number of cycles experienced in the historic earthquake compared to the seismic demand parameters for the site (NZGS Module 1). Magnitude scaling factors may need to be considered.

For example, a higher probability of liquefaction could be chosen (PL of 50% rather than 15%) where liquefaction analysis shows an over-prediction of damage for the particular earthquake at PL of 15%.

Step change factors are still applicable in this instance if a step change has been identified.

Appendix C4F: Influence of Shaking Levels on Ground Stability and Liquefaction Triggering

NZGS/MBIE Module 1 - Overview of the guidelines includes guidance on assessing shaking hazard at a site. NZGS/MBIE Module 3 - *Identification, assessment and mitigation of liquefaction hazards* includes guidance on assessing intensity of shaking to trigger liquefaction. This appendix supplements those modules by providing an overview of slope instability and liquefaction potential at various intensities of earthquake shaking. The content of this appendix is not intended to replace screening and assessment of ground stability and liquefaction but rather to provide an initial screening and a reality check following assessment.

When discussing triggers for seismic slope instability or liquefaction, peak ground acceleration (PGA) is often referred to. This is a measure of ground acceleration at a particular site by instruments.

The Modified Mercalli Intensity (MMI) scale uses personal reports and observations to measure earthquake intensity and is therefore more subjective. As an indication of PGA force, an earthquake that results in 0.2 g may cause people to lose their balance and is approximately equivalent to MM7 (Dowrick et al., 2008).

An important step is for the engineer to determine how the land deformation may impact on the integrity of the foundation and structure in terms of life safety protection. Land damage on its own is not the problem per se: it is the effects on the performance of the structure and people that should be established. Understanding if and how the land may deform is an initial step in the assessment process.

As an initial screening tool to appreciate whether a particular PGA at a site could trigger instability or liquefaction, correlation can be made between the PGA in question (refer to NZGS/MBIE Module 1), modified for terrain amplification effects as appropriate, and the MMI, and then onto generic descriptors of land stability and building behaviour (Dowrick et al., 2008). Refer to Table C4F.1 below for examples of the correlation. The MMI-PGA correlation is extracted from Saunders and Berryman (2012).

The following table provides an approximate correlation between PGA, MMI and land damage descriptors provided by Dowrick et al., 2008. Additional comments have been added based on experiences from the Canterbury earthquake sequence of 2010-11 (comments by Dowrick et al., 2008 that are not representative of recent experience are retained in italics for reference).

Table C4F.1: Approximate correlation between PGA, MMI and land damage descriptors

PGA, g	MMI	Land descriptors*
<0.03	<MM5	• Land/slope issues are unlikely.
0.03-0.08	MM5	• Loose boulders may occasionally be dislodged from steep slopes.
0.08-0.15	MM6	• Loose material may be dislodged from sloping ground, e.g. existing slides, talus and scree slopes. • A few very small (≤10^3 m^3) soil and regolith slides and rockfalls from steep banks and cuts. • A few minor cases of liquefaction (sand boil) in highly susceptible alluvial and estuarine deposits.
0.15-0.25	MM7	• Small slides such as falls of sand and gravel banks, and small rockfalls from steep slopes and cuttings common. • Instances of settlement of unconsolidated, or wet, or weak soils. • Very small (≤10^3 m^3) disrupted soil slides and falls of sand and gravel banks, and small rockfalls from steep slopes and cuttings are common. • Fine cracking on some slopes and ridge crests. • A few small to moderate landslides (10^3 - 10^5 m^3), mainly rockfalls on steeper slopes (>30º) such as gorges, coastal cliffs, road cuts and excavations. • Small discontinuous areas of minor shallow sliding and mobilisation of scree slopes in places. • Minor to widespread small failures in road cuts in more susceptible materials. • A few instances of non-damaging liquefaction (small water and sand ejections) in alluvium.
		Added comment: **Widespread damaging liquefaction in alluvial soils experienced across Christchurch and environs including lateral spread.**
0.25-0.45	MM8	• Cracks appear on steep slopes and in wet ground. • Significant landsliding likely in susceptible areas. • Small to moderate (10^3-10^5 m^3) slides widespread; many rock and disrupted soil falls on steeper slopes (steep banks, terrace edges, gorges, cliffs, cuts, etc.). • Significant areas of shallow regolith landsliding, and some reactivation of scree slopes. • A few large (10^5-10^6 m^3) landslides from coastal cliffs, and possibly large to very large (≥10^6 m^3) rock slides and avalanches from steep mountain slopes. • Larger landslides in narrow valleys may form small temporary landslide-dammed lakes. • Roads damaged and blocked by small to moderate failures of cuts and slumping of road-edge fills. • Increased instances of settlement of unconsolidated, or wet, or weak soils. • Evidence of soil liquefaction common, with small sand boils and water ejections in alluvium, and localised lateral spreading (fissuring, sand and water ejections) and settlements along banks of rivers, lakes and canals etc.
		Added comment: **Widespread severely damaging liquefaction in alluvial soils experienced across Christchurch and environs including severe lateral spread and wide-area damage to structures on shallow foundations.**

PGA, g	MMI	Land descriptors*
0.45-0.60	MM9	• Cracking of ground conspicuous. • Landsliding widespread and damaging in susceptible terrain, particularly on slopes steeper than 20º. • Extensive areas of shallow regolith failures and many rockfalls and disrupted rock and soil slides on moderate and steep slopes (20º-35º or greater), cliffs, escarpments, gorges, and man-made cuts. • Many small to large (103-106 m^3) failures of regolith and bedrock, and some very large landslides (106 m^3 or greater) on steep susceptible slopes. • Very large failures on coastal cliffs and low-angle bedding planes in Tertiary rocks. Large rock/debris avalanches on steep mountain slopes in well-jointed greywacke and granitic rocks. Landslide-dammed lakes formed by large landslides in narrow valleys. • Damage to road and rail infrastructure widespread with moderate to large failures of road cuts and slumping of road-edge fills. Small to large cut slope failures and rockfalls in open mines and quarries. • Liquefaction effects widespread with numerous sand boils and water ejections on alluvial plains, and extensive, potentially damaging lateral spreading (fissuring and sand ejections) along banks of rivers, lakes, canals, etc.). Spreading and settlements of river stop-banks likely.
		Added comment: **Widespread severely damaging liquefaction in alluvial soils experienced across Christchurch and environs including severe lateral spread.**
0.60-0.80	MM10	• Landsliding very widespread in susceptible terrain. • Similar effects to MM9, but more intensive and severe, with very large rock masses displaced on steep mountain slopes and coastal cliffs. Landslide-dammed lakes formed. Many moderate to large failures of road and rail cuts and slumping of road-edge fills and embankments may cause great damage and closure of roads and railway lines. • Liquefaction effects (as for MM9) widespread and severe. Lateral spreading and slumping may cause rents over large areas, causing extensive damage, particularly along river banks, and affecting bridges, wharfs, port facilities, and road and rail embankments on swampy, alluvial or estuarine areas.
080-0.90	MM11	• Environmental response criteria have not been suggested for MM11 as that level of shaking has not been reported in New Zealand or (definitively) elsewhere.
> 0.90	MM12	• As above.

Note:
* Land descriptors are based on Dowrick et al. (2008). Comments that do not reflect recent experience are retained (in italics) for reference. Refer to Dowrick et al. (2008) for full descriptors of building damage. Additional comments (in bold) are based on experiences from the Canterbury earthquake sequence.

Appendix C4G: Examples of applying geotechnical step change methodology

Section C4.7 presents guidance on identifying and allowing for geotechnical step change. This appendix summarises the application of that guidance to six example buildings. These example buildings are outlined by Figures C4G.1 to C4G.3 and their assessment is summarised in Table C4G.1. Below Table C4G.1 are notes highlighting aspects of the geotechnical step change assessment.

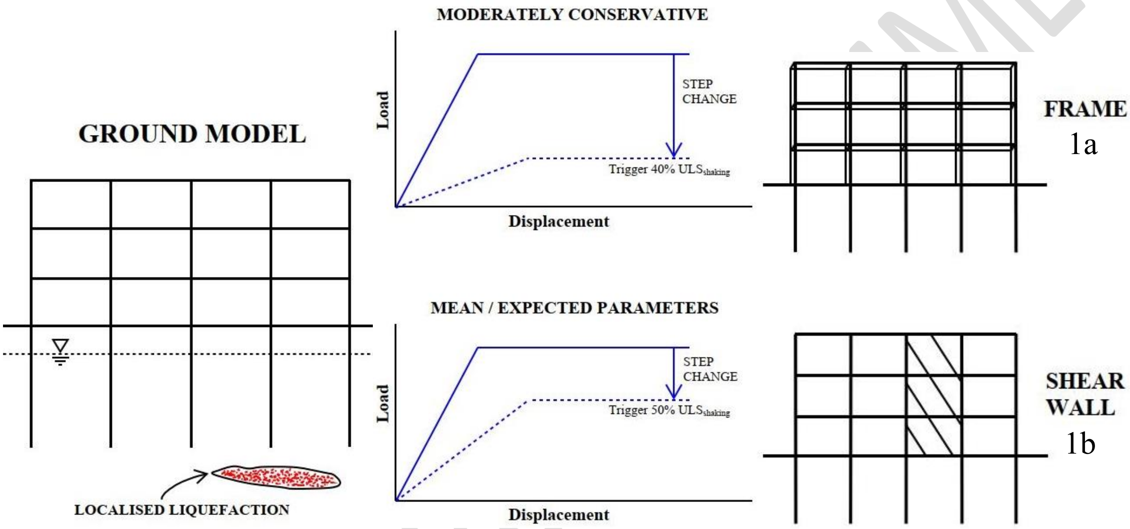
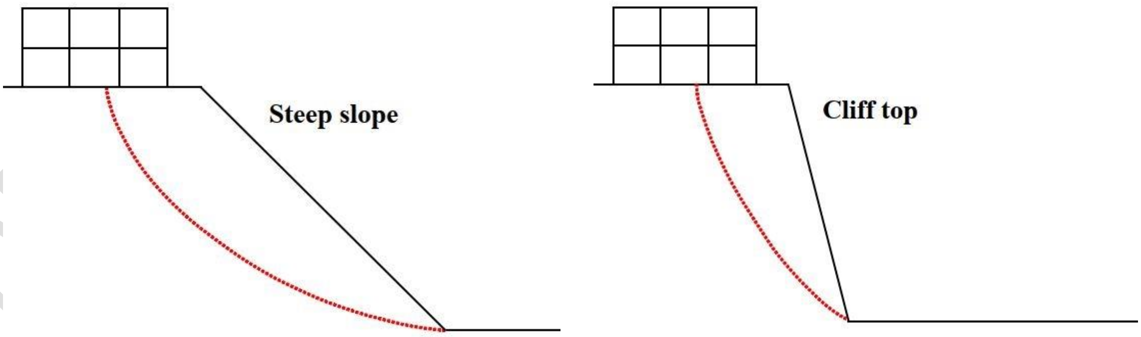
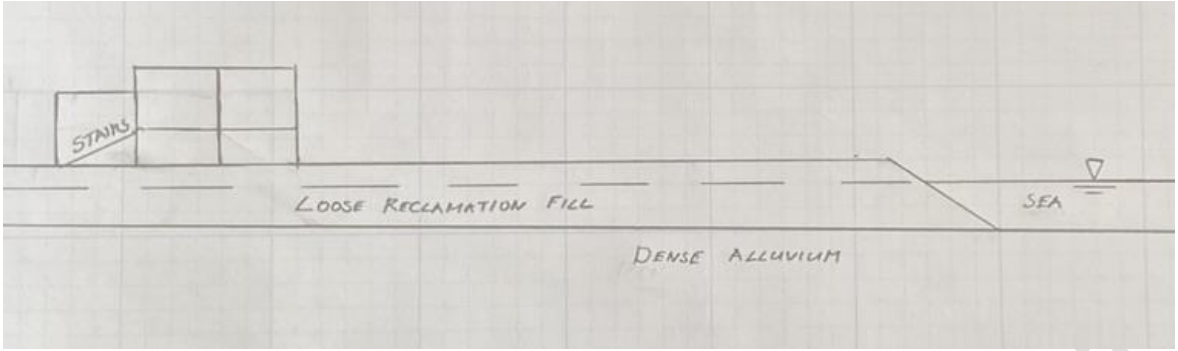


Figure C4G.1: Localized liquefaction compromising some piles
1a Frame structure
1b Shear wall structure

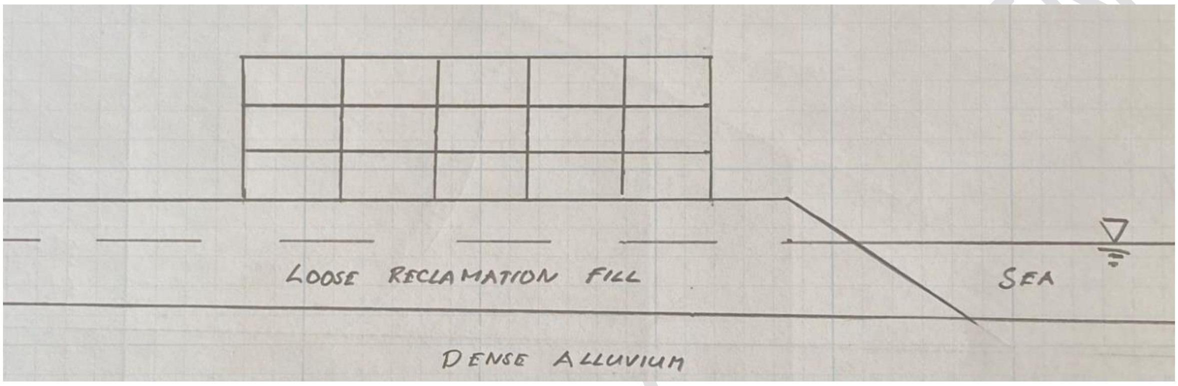


2a Building above steep slope **2b Building on cliff top**

Figure C4G.2: Slope instability



3a Lateral spread (Lateral stretch)



3b Lateral spread (Edge failure)

Figure C4G.3: Lateral spread

Table C4G.1: Summary of assessment

Example	ID	Refer Figure C4F.1		Refer Figure C4F.2		Refer Figure C4F.3	
		1a	1b	2a	2b	3a	3b
	Description	Localized liquefaction compromising some piles		Slope instability below a building		Lateral spread stretching a building	
		Moment frame	Shear wall	Above a steep slope	At a cliff top	Lateral stretch	Edge failure
Stage 1 Moderately conservative parameters	%ULS shaking triggering geotechnical step change	40%	40%	80%	100%	50%	50%
	Structure's probable capacity exceeded due to geotechnical step change (yes/no)	Yes	Yes	Yes (>200 mm displacement)	Yes (evacuation, m's of displacement)	Yes (>75 mm differential displacement)	Yes (>200 mm displacement)
Stage 2 Mean parameters and step change factor	%ULS shaking triggering geotechnical step change	55%	55%	100%	120%	70%	70%
	Structure's probable capacity exceeded due to geotechnical step change (yes/no)	No	Yes	No (<200 mm displacement)	Yes (Evacuation)	Yes (>75 mm)	Yes (>200 mm displacement)
	Gravity support loss expected at 1.5 x the step change trigger or less (yes/no)	NA	Yes	NA	Yes	No (<150 mm)	Yes (>700 mm displacement)
	X = X1 x X2, X1: Buffer, X2: Consequence.	NA	X = 1.5 X1 =1.5, X2 = 1.0	NA	X= 2.0 X1 =1.5 X2 = 1.3	X=1 X1=1.0	X= 2.0 X1 =1.5 X2 = 1.3
	%ULS shaking / X	NA	35%	NA	60%	70%	35%
Lesser of the two stages	%NBS	40%NBS	35%NBS	80%NBS	60%NBS	50%NBS	30%NBS
Reality check	Is this assessment reasonable? (yes/no)	Yes	Yes	Yes	Yes	Yes	Yes

Notes highlighting aspects of geotechnical step change assessment:

General

The critical case %NBS, stage 1 or stage 2, is highlighted red in Table C4G.1.

As highlighted by the last row of Table C4G.1, on completion of the assessment the structural and geotechnical engineers should discuss the conclusions and undertake a reality check on the proposed %NBS. Does this %NBS feel right considering what has been observed in historic earthquake? (Refer C4.2.8 Closing the loop)

1. Refer Figure C4G.1 and Table C4G.1

   To model the localized liquefaction structural analysis is undertaken assuming the capacity and stiffness of a critical pile is reduced while other piles remain unaffected (Refer C4.5.7 Spatial variation of soils and reporting ground models). The conclusions of this analysis and assessment are:

   a. 1a Moment Frame

      Stage 1: Assuming the moderately conservative geotechnical parameters the beam moment probable capacity is exceeded as a direct result of the geotechnical step change.
      Stage 2: Assuming the mean geotechnical parameters the geotechnical step change does not lead to exceeding the structures probable capacity.
      Consequently Stage 1 governs the assessed %NBS.

   b. 1b Shear wall

      Stages 1 and 2: The geotechnical step change and associated deformation of the shear wall results in the beam moment probable capacity being exceeded.
      A buffer does not exist between exceeding probable capacity and loss of gravity support (X1=1.5). A limited number of people occupy the building (X2=1.0). (X=1.5).
      Stage 2 governs the assessed %NBS.

2. Refer Figure C4G.2 and Table C4G.1

   Earthquake shaking results in slope instability and displacement causing damaging deformation of the building.

   a. 2a Top of a steep slope

      Slope displacements are limited.
      Stage 1: As a moderately conservative estimate 80%ULS shaking results in slope displacement (200 mm) which results in deformation of the building exceeding its probable capacity.

Stage 2. Based on a mean estimate slope displacements mobilised by 100% ULS shaking are relatively small and tolerable to the structure.
Stage 1 governs the assessed %NBS.

b. 2b Top of a cliff

Earthquake shaking triggers evacuation (collapse) of the cliff. This results in loss of gravity support (collapse) of the building. Ie Stage 2 governs and the % NBS is factored down by X accordingly.

3. Refer Figure C4G.3 and Table C4G.1

Earthquake shaking triggers liquefaction and lateral spread of a reclamation fill. Piled buildings are located on the reclamation and are at risk of being deformed by the lateral spread.

a. 3a Lateral stretch.

The building is located 70m back from the reclamation edge. The building comprises two separate structures. Each structure is well tied together but there is limited tying from one structure to the other. A set of stairs spans from one structure to the other and lands on a ledge 150mm wide. The differential displacement between the two structures due to lateral spread is assessed. 75mm differential displacement is taken as the limit of the probable capacity of the stair support. 150mm results in loss of gravity support. %ULS shaking triggering 75mm and 150mm differential lateral spread are assessed and applied in the assessment. In Stage 2 (mean parameters) 70% ULS shaking triggers 75mm differential displacement and exceedance of the stair support probable capacity. 1.5 times this %ULS shaking (105%) is assessed to result in more displacement but not so much as to result in loss of gravity support of the stairs (150mm). A buffer between probable capacity and loss of gravity support exists such that the %NBS does not need to be down rated in Stage 2 (X=1.0). Stage 1 governs.

c. 3b Edge failure

The building is located immediately adjoining the reclamation edge. The building is poorly tied together and has a precast floor system. In the event of earthquake shaking triggering liquefaction edge failure lateral spread beneath the seaward portion of the building is assessed to be likely. The associated deformation in the structure is expected to result in loss of support to a number of floors in the building. Applying this assessment to the methodology results in a down rating of the %NBS by X=2. (X1=1.5; there is no buffer between probable capacity and loss of gravity support. X2=1.3; loss of a number of floors could be expected).

Appendix C4H Allowing for uncertainty in predicting soil deformation near strength capacity.

C4H.1 Introduction

Section C4 of the Guideline proposes that the geotechnical engineer assess and report on potential soil/foundation deformations. This includes situations where available prediction methods may not be reliable, such as:

- Load displacement behaviour of a foundation where the demand exceeds 70% of the assessed ultimate geotechnical capacity of the foundation.
- Load displacement behaviour of foundations bearing above liquefied soils or soils subject to cyclic softening.
- Magnitudes of lateral spread or cyclic displacement.
- The magnitude of slope or retaining wall displacements due to earthquake shaking.

There are limited empirical or numerical analysis methods available to assist in making assessments in these situations and the results of these analyses, including those of dynamic finite element analysis, have considerable uncertainty. Predicted deformations could vary from actual values by a factor of 5 or more. When reporting geotechnical parameters, the uncertainties in these parameters should also be reported (refer C4.2.5).

This Appendix provides guidance through three examples of how these uncertainties can be allowed for in assessment. The three examples are:

- Foundation demand > 70% of the foundation geotechnical capacity
- Foundation demand > foundation capacity reduced by liquefaction effects
- Lateral spread deformation > the deformation capacity of the structure or greater than the structure's strength capacity to resist the lateral spread.

These examples demonstrate that even though the soil/foundation deformation cannot be reliably predicted, assessment of the building is possible by careful consideration of the following factors. These factors are likely to be more influential to the assessment than the absolute value of soil/foundation deformations.

- Vulnerability of the structure to these deformations.
- How deformation potential changes with increasing intensity of earthquake shaking.

Because of the uncertainties involved simplified analyses, parametric studies and judgement are likely to be more appropriate for most assessments than complex detailed analyses.

In design strength reduction factors of typically 0.5 are applied and, where a liquefaction or lateral spread hazard is identified, mitigation measures are normally put in place. This design process avoids the need to assess deformation of soils in the situations listed above.

For building assessment, the structural engineer also needs to deal with considerable uncertainty including assessment of the structure's behaviour well beyond the limits normally considered in design. For example, to assess if a structure presents a significant life safety hazard (SLSH), the structural engineer may need to consider the behaviour of the structure well beyond that of ultimate limit state. This assessment beyond ultimate limit state includes considerable uncertainty and requires application of judgement.

C4H.2 Example 1: Foundation demand > 70% of foundation geotechnical capacity

Although this example has been based on consideration of a shallow foundation a similar approach could apply to a deep foundation.

Section C4.6.4.1 describes how the nonlinear load displacement behaviour of a shallow foundation could be modelled for a foundation demand >70% of the ultimate geotechnical capacity. C4.6.4.1 refers to the green line on Figure C4.9 which is reproduced below. Refer to Section C4.6.4.1 for details of developing this model and the sensitivity analysis considered necessary because of the uncertainties in predicting deformations at these high demands.

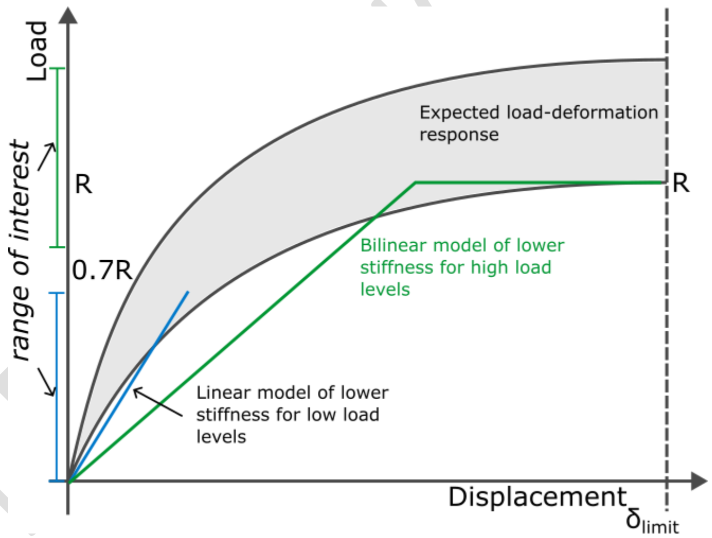


Figure C4.9: Expected load-displacement behaviour and possible foundation models. Blue line: foundation model for low demand (load) levels. Green line: foundation model for high demand (load) levels.

Although with simplification and uncertainty, this model suggested in C4.6.4.1 allows the structural engineer to test the vulnerability of the structure to potentially large displacements at demands greater than 70% of the foundation's ultimate geotechnical capacity (>70%R). If this structural assessment concludes that the foundation behaviour results in exceedance of the structure's probable capacity leading to a SLSH then the %NBS score for this issue is to be calculated as described in Part A and Section C1 of these Guidelines, i.e. the structure's probable capacity divided by demand.

C4H.3 Example 2: Foundation demand > foundation capacity reduced by liquefaction effects

Figure C4.12, from section C4.7.1 and reproduced below, shows an example of calculated reduction in bearing capacity of a shallow foundation on liquefiable soil with increasing intensity of earthquake shaking. Build-up in pore pressure with increased intensity of shaking results in reduced bearing capacity. In this example liquefaction is triggered (Factor of safety against liquefaction =1) at 60%ULS shaking. This relationship between bearing capacity and intensity of earthquake shaking can be developed by methods presented in NZGS/MBIE Modules 3 and 4.

Although this example has been based on consideration of a shallow foundation a similar approach could apply to a deep foundation.

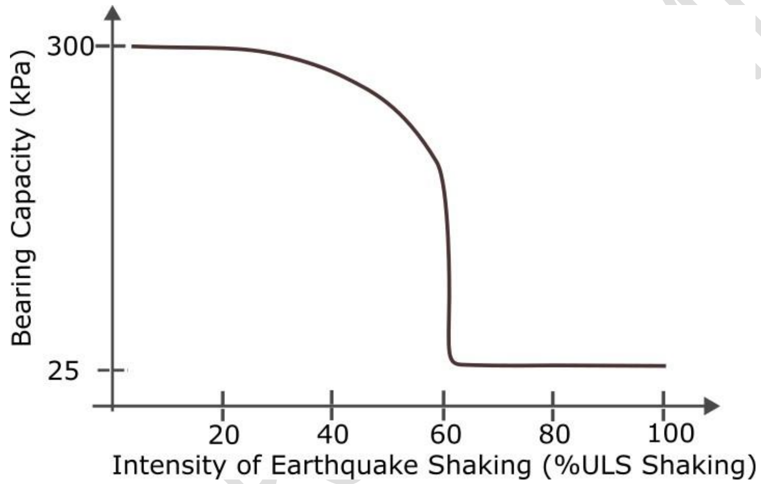


Figure C4.82: Calculated bearing capacity and intensity of earthquake shaking

The load deformation behaviour of this foundation could be modelled as bi-linear as described in Section C4H.2. In line with the methodology for assessing this geotechnical step change described in C4.7.2 the structural analysis could consider two situations separately: namely, without liquefaction with a bearing capacity of 300kPa and after liquefaction with a bearing capacity of 25kPa. This structural analysis is to test the vulnerability of the structure to deformation. The results of the analysis could be applied to assess the %NBS score as described in Section C4.7.2.

The change in bearing capacity with increasing intensity of shaking and the vulnerability of the structure to foundation displacement are likely to be more influential to the assessment than the prediction of the load deformation behaviour at a specific %ULS shaking.

If the structural assessment indicates foundation demands less than the foundation bearing capacity allowing for liquefaction, foundation displacements could be assessed by methods proposed by Bray et.al. (2012). The structure's ability of the structure to tolerate these potential settlements should be assessed.

C4H.4 Example 3: Lateral spread deformation > the deformation or strength capacity of the structure

The following describes a three-step process for this assessment.

Step 1: Lateral spread assessment

Geotechnical investigations, modelling, analysis and assessment are undertaken to evaluate the potential for lateral spread as described in NZGS/MBIE Module 3. This should include application of multiple methods, parametric studies, consideration of case histories and judgement. It should consider any restraint to lateral spread provided by the subject structure or by any other structures. The output from this assessment is an understanding of the potential for lateral spread at the site including;

- How the geometry of lateral spread could vary across the site and with depth. This is to be applied to develop scenarios of ground deformation to apply in soil structure interaction (SSI) analysis to test the vulnerability of the structure. These scenarios are best described by simple sketches.
- The magnitude of lateral spread potential and how this could vary with %ULS shaking.
- Uncertainty.

Step 2: Soil structure interaction (SSI)

The scenarios of geometry of lateral spread from step 1 plus soil properties are applied to SSI analysis and assessment. The magnitude of soil deformation is increased until damage to the structure which could lead to a SLSH is indicated. This step necessarily requires close collaboration between structural and geotechnical, parametric studies, and judgement. The output from this assessment is an understanding of the vulnerability of the structure to potential lateral spread including:

- Simple sketches of the ground and structure deformation. These sketches aid communication between structural and geotechnical and help the understanding of the SSI and particularly what is leading to the identified SLSH.
- For each scenario the magnitude of lateral spread which could lead to a SLSH.
- An understanding of the uncertainties.

Step 3: Judgement and assessment of % NBS

The %NBS score for the issue of lateral spread is to be assessed as described in C4.2.7 and C4.7.2. The magnitude of lateral spread triggering a SLSH from Step 2 and the relationship between %ULS shaking and magnitude of lateral spread from Step 1, are the inputs to this assessment of %NBS. However, uncertainty must still be accounted for. A possible approach to this issue of uncertainty is to think in terms of likelihood rather than absolutes. The absolutes are unknown. This possible approach is described by the following example:

<u>Example 3b from Appendix C4G</u>

Background

Refer to Appendix C4G for a description of this example, including:

- Figure C4G.3: A cross section
- Table C4G.1 column 3b: A summary of the assessment of the %NBS
- Notes 3b: Brief notes supporting the summary of the assessment of the %NBS.

The subject structure is located on reclaimed land adjoining a reclamation edge. It is supported on piles extending through the reclamation fill to found in underlying dense ground. The critical scenario of lateral spread has been assessed to be a failure of the reclamation edge imposing ground deformation around, and lateral load on, the seaward line of piles. The associated deformation in the structure is assessed to result in loss of support to a number of floors in the building, a significant life safety hazard (SLSH).

Assumed inputs to the assessment

- 100%ULS shaking: 0.35g M7.1
- Lateral spread displacement at the head of the seaward piles which results in exceedance of the structures probable capacity leading to a SLSH: 200 mm. (This value was assessed from the SSI analysis in step 2).

Application of likelihood approach to the assessment

Consider a series of scenarios of shaking demand. For this example, we have considered 40, 50, 70 and 90%ULSshaking as indicated in the left-hand column of Table C4H.1 below. For each of these scenarios (specific levels of shaking) consider the results of the lateral spread assessment (from Step 1) and judge the likelihood of the displacement exceeding the tolerance limit for the structure. That limit was assessed to be 200mm from Step 2. The judged likelihoods for this example are recorded in the right-hand column of Table 4CH.1. A likelihood of approximately 15% is representative of a moderately conservative estimate of the %ULSshaking which could cause 200mm or more of lateral spread (i.e. 50%ULSshaking for this example, refer Table C4H.1). A likelihood of approximately 50% is representative of a mean or expected estimate of the %ULSshaking which could cause 200mm or more of lateral spread (i.e. 70%ULSshaking for this example, refer Table C4H.1). These moderately conservative and mean estimates of lateral spread can then be applied to the two Stage assessment of %NBS for a geotechnical step change as described in C4.7.2. The following describes that 2 Stage assessment.

- Stage 1

  Apply a moderately conservative geotechnical assessment, assess the %ULS shaking triggering exceedance of the structure's probable capacity leading to a SLSH. For this example, that is 50%ULSshaking. 50%NBS is calculated.

- Stage 2

  Apply a mean geotechnical assessment, assess the %ULSshaking triggering exceedance of the structure's probable capacity leading to a SLSH. For this example, that is 70% ULSshaking. In calculating the %NBS apply a step change factor of 2.

- 35%NBS is calculated for this example.

  The %NBS is the lower of the two stages. (i.e. 35%NBS).

Table 4CH.1: Assessed likelihood

%ULS shaking	PGA (M7.1)	Likelihood of lateral spread assessment exceeding 200 mm at pile head. Judged values based on the results of Step 1, lateral spread assessment.
40	0.14g	5%
50	0.18g	15%
70	0.25g	50%
90	0.32g	75%
40	0.14g	5%
50	0.18g	15%

Conclusion

There is considerable uncertainty in predicting lateral spread and its impact on a building, however breaking the problem down into steps plus careful informed judgement can provide a basis for the assessment of the building. In particular:

- Identify the magnitude of displacement which could lead to a SLSH.
- Assess the likelihood of this displacement occurring at various levels of earthquake shaking.

C4H.5 Conclusion

Where soil/foundation deformation cannot be reliably predicted, assessment of the building is possible by careful consideration of the following factors. These factors are likely to be more influential to the assessment than the absolute value of soil/foundation deformations.

- Vulnerability of the structure to these deformations.
- How deformation potential changes with increasing intensity of earthquake shaking.

When reporting geotechnical parameters, the uncertainties in these parameters should also be reported and the structural analysis should include sensitivity analysis (refer C4.2.5).