PART C

C4 Geotechnical Considerations

Document Status

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.building.govt.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

Please go to www.building.govt.nz to provide feedback or to request further information about these Guidelines.

Errata and other technical developments will be notified via www.building.govt.nz

Acknowledgements

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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 and,
- 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, Steps 1, 2 and 3, outlined in Figure C1.1 of Section C1. However, the level of consideration 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).

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, and
- 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
- provision of input to soil-structure interaction (SSI) models and consideration of SSI in seismic assessment
- assessment of geotechnical aspects of foundation behaviour
- inputs to the calculation of *%NBS* (typically in a form relating to geotechnical influences on the assessment of the structure's probable capacity), and
- reporting and peer review.

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 bring assessed.

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 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 C4.5.

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. The material in some of the appendices is shown as "interim guidance" indicating that the guidance given does not yet appear in 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 (e.g. in the treatment of uncertainties).

Note:

The information regarding the status of each NZGS/MBIE module was correct at July 2017. Please check at 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**
1. Overview of the guidelines (March 2016)	• 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
2. Geotechnical investigations for earthquake engineering (November 2016)	• 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
3. Identification, assessment and mitigation of liquefaction hazards (May 2016)	• Introduces the subject of soil liquefaction; describes the various liquefaction phenomena including lateral spreading • Includes discussion on clay soils and volcanic soils

NZGS/MBIE module (publication date)	**Description**
4. Earthquake resistant foundation design (November 2016)	• 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.
5. Ground improvement of soils prone to liquefaction (May 2017)	• 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
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 re 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.
6. Earthquake resistant retaining wall design (May 2017)	• 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.
7. Landslides and rockfalls (Planned for future development)	• Will consider landslide and rockfall hazard assessment and mitigation including earthquake effects. Note: GNS Science's wealth of reporting on the Port Hills soil and rock slope stability in the Canterbury earthquake sequence is informative for landslide and rockfall hazard assessment in other parts of New Zealand.

C4.1.2.2 US geotechnical guidance

ASCE 41-13 (2014) – Foundations and geologic site hazards

ASCE 41-13 (2014) Chapter 4 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 4 of ASCE 41-13 (2014) presents general requirements for consideration of foundation load-deformation 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-deformation characteristics

- procedures for consideration of SSI effects
- seismic earth pressures on building walls, and
- 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 (mean) capacities to determine how a building may rate against minimum Building Code (B1) requirements.

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 41-13, 2014).

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 (an alternative solution to these guidelines), S_p is likely to require amendment accordingly and care will be necessary to reflect the high level of potential uncertainty in such assessments.

C4.1.3 Definitions and acronyms

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.
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines.
FE	Finite element (refer to Section C4A.3.6)
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. Step change is often a characteristic of the ground and foundation performance in a geotechnically dominated project.
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

M-O equation	Mononobe-Okabe equation (refer to Appendix C4B)
MSE	Mechanically stabilised earth
PGA	Peak ground acceleration
Probable capacity (of a foundation/soils)	Assumed probable resistance (i.e. strength) and probable deformation capacity of a foundation/soils/geohazard. The probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Probable deformation capacity/limit δ_{SC} or δ_{L}	The maximum deformation (δ_{SC} or δ_{L}) a foundation can tolerate while continuing to provide resistance R or R_{R} as appropriate
Probable strength (capacity) R	Ultimate geotechnical strength capacity or nominal resistance. Evaluated as it would be for design (refer to NZGS/MBIE Module 4: *Earthquake resistant foundation design*).
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 strength (capacity) R.
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
SPT	Standard penetration test
SSI	Soil- structure interaction
Step change	The point at which the behavior of the structures, the ground or foundation is considered to abruptly deteriorate/reduce
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 (loading or displacement) 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 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C4.1.4 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as calculated by application of these guidelines
A_{loop}	Area contained within the hysteretic curve
B	Width of foundation
c	Soil cohesion
G_{sec}	Equivalent secant modulus
H	Wall height
k_{h}	Earthquake acceleration design coefficient (calculated using W = 1)
R	Ultimate geotechnical resistance/strength capacity (Probable Strength)
$R_{\mathrm{d}} = \phi_{\mathrm{g}} R$	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 residual resistance strength capacity after a step change
S_{u}	Undrained conditions of embedded cantilever walls
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.
γ_{c}	Expected amplitudes of shear stress and shear strain respectively
δ_{cap}	Expected limiting deformation
δ_{SC}	Predicted deformation at a step change
ξ_{soil}	Equivalent viscous damping ratio
τ_{c}	Expected amplitudes of shear stress and shear strain respectively
ϕ	Strength reduction factor
ϕ_{g}	Geotechnical strength reduction factor
γ	Unit weight of the backfill

C4.2 Roles and Responsibilities

C4.2.1 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. taking into account 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, 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. 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.

C4.2.2 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)
- 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, and
  - 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, and
- 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. It should be expected that the scope of the geotechnical input may increase as the assessment proceeds and the impact of geotechnical issues on the expected behaviour of the building becomes clearer.

C4.2.3 Geotechnical engineer's role and required experience

The geotechnical engineer:

- provides advice relating to SSI effects, geohazards and soils 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, and
- 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.

C4.2.4 Roles by project category

C4.2.4.1 General

On completing Step 3 of the DSA process 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 and the process).

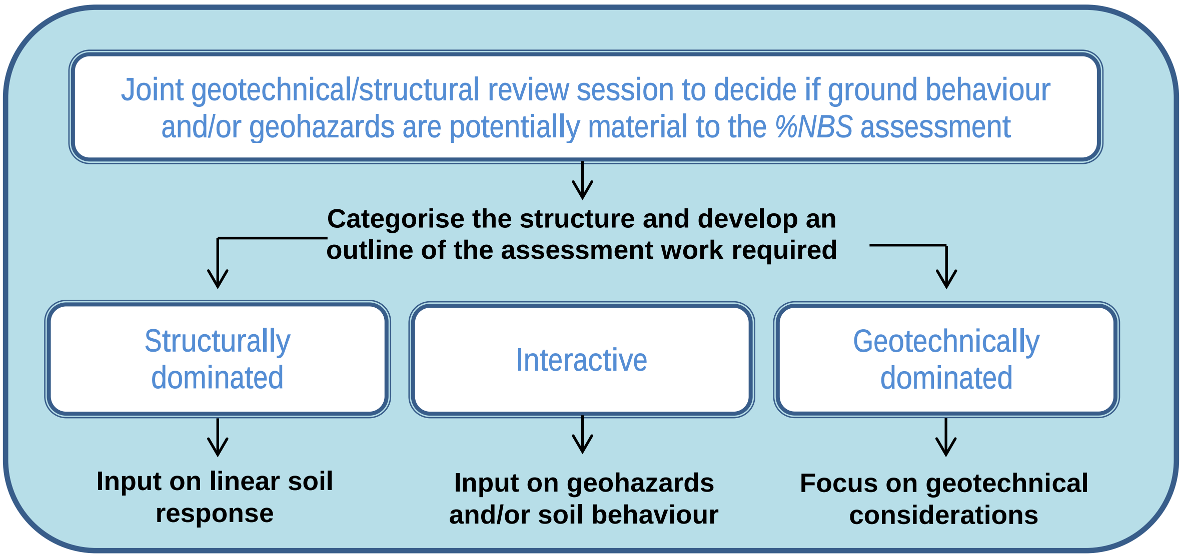


Figure C4.1: Project categorisation to reflect potential impact on the assessment of geotechnical issues

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.

C4.2.4.2 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
- soil foundation stiffness (reported as a range of linear spring stiffnesses) and probable resistance available/strength (capacity).

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
- 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.

C4.2.4.3 Interactive

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, and
- investigation and analysis is targeted and appropriate for specific building vulnerabilities.

C4.2.4.4 Geotechnically dominated

Geotechnically dominated projects are expected to include those where step change in ground and/or foundation behaviour can occur. In this category, significant interaction is expected between the geotechnical and structural engineering disciplines.

The geotechnical engineer defines the expected onset of the step change as a proportion of the shaking considered in an ultimate limit state (ULS) event (for %ULS shaking refer to Section C4.5.3). The structural engineer then confirms that a brittle structural step change directly follows the geotechnical step change and that this response occurs at a lower shaking level than any other (structural) mechanism.

The geotechnical engineer will convey the details of the geohazard anticipated to result in the critical mechanism. In some cases, spring-type representation of the ground may not be required as the criticality of the geohazard can be defined without detailed structural analysis.

Typically, the emphasis will be on details of the critical geohazard. For example, this may be by an estimate of settlement or displacement from liquefaction or lateral spread. 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.

C4.3 Assessment Process

C4.3.1 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).

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

- Stage 1 – project definition
- Stage 2 – assessment (including the geotechnical desktop study and geotechnical analysis and assessment), and
- Stage 3 – reporting within the DSA.

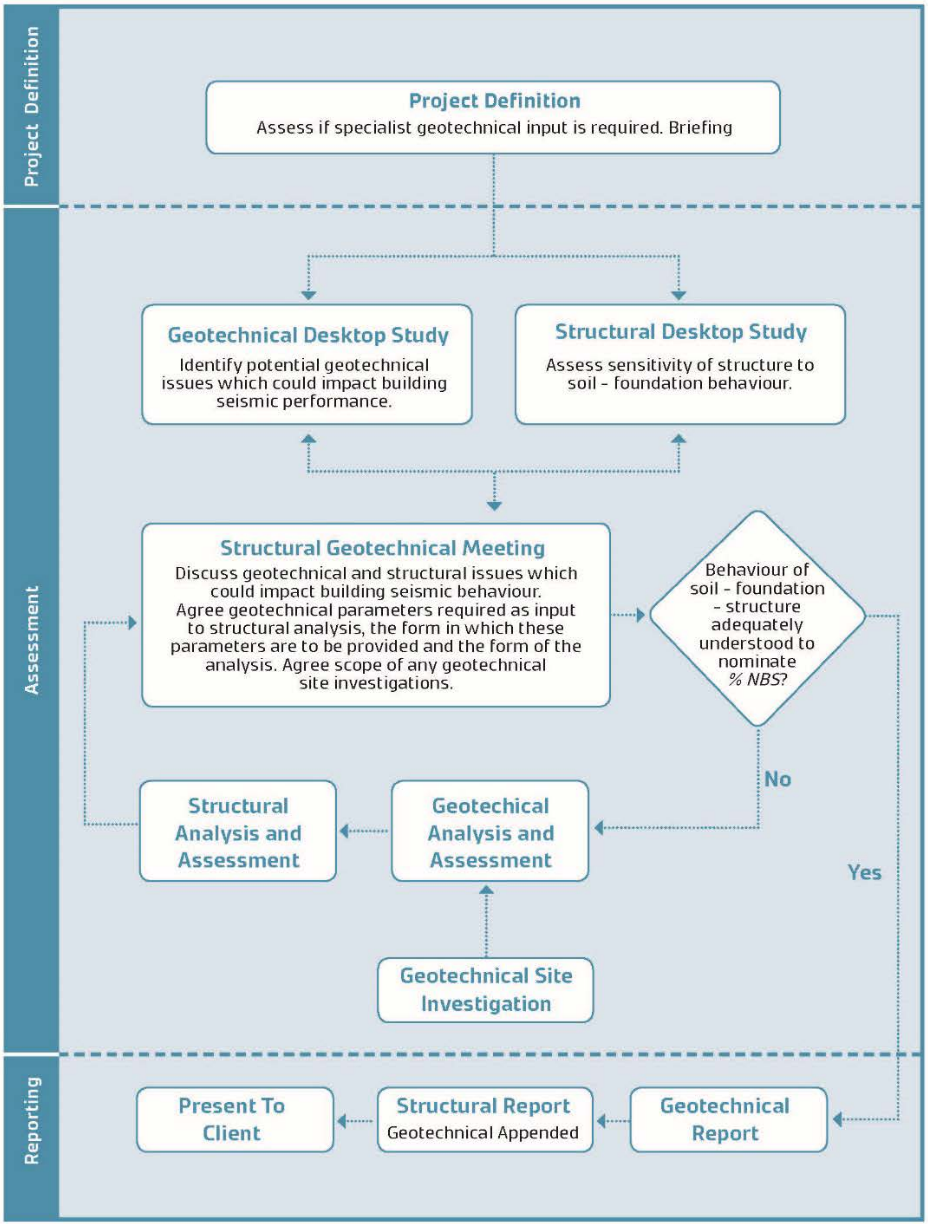


Figure C4.2: Project definition, assessment and reporting stages

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

C4.3.2 Stage 1 – Project definition

This first stage of the process outlined in Figure C4.2 is the initial review by the structural engineer, preferably in collaboration with the geotechnical engineer, to assess whether specialist geotechnical input is required and the likely scope of that work.

This involves:

- 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, 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.

Note:

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, it is likely 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.

C4.3.3 Stage 2 – Assessment

C4.3.3.1 Desktop study

The initial part of the assessment involves separate preliminary geotechnical and structural desktop investigations.

The geotechnical desktop study is to identify potential geotechnical issues that could affect the building's seismic behaviour. Section C4.4 provides guidance on undertaking this desktop study and reporting its conclusions.

The output of the geotechnical desktop study should include:

- a sketch (cross section) and information to describe the inferred ground model, including the soil profile
- a list of geotechnical issues (including geohazards) that could influence the seismic assessment of the building, and
- an outline of uncertainties.

NZGS Module 2 - *Geotechnical investigations for earthquake engineering* provides guidance on undertaking a desktop study to inform likely site ground conditions and geohazards. For assessment of an existing building, information also needs to be collated and reviewed to inform the likely details of the existing foundations. This includes collating and reviewing historic drawings, and a site inspection to challenge the accuracy of those drawings. Conversations with people involved in the original construction or subsequent site work can be another valuable source of information.

C4.3.3.2 Structural geotechnical meetings

Once the structural and geotechnical engineers have carried out their desktop investigations, 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 C4.3.3.1)
  - 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.
  - 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 C4.4).
- **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, and
    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, and
  - 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.

C4.3.3.3 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.

C4.3.4 Stage 3 – Reporting and peer review

As the assessment process (Stage 2) is collaborative and iterative, the geotechnical report cannot be completed 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.

C4.4 Site Characterisation

C4.4.1 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.

C4.4.2 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 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 also important to also highlight the uncertainties. Refer to Section C4.4.4.

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.

C4.4.3 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.*

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 Appendix C4D)
- loss of ground strength and stiffness under the building – liquefaction (sandy soils) and cyclic softening (clayey soils), post liquefaction settlement (Module 3 and Appendices C4E and C4F)
- 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 C4B and 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) relevant to the assessment needs to be considered.

Note:

NZGS/MBIE Module 1 provides an overview of assessment of slope stability. A future module may be developed to consider this further. In the interim some guidance is provided in Appendix C4C.

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.

C4.4.4 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 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.

C4.4.5 Site investigations

C4.4.5.1 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 C4.3.3.1), 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)
- dimensions of existing foundations (refer to Section C4.4.5.2 below)
- foundation load/deformation behaviour (refer to Section C4.4.5.3).

C4.4.5.2 Dimensions of existing foundations

During the desktop stage available information relating to the existing foundations should be collated and reviewed. Sources of information include:

- historic drawings and geotechnical reports, potentially sourced from council property files, building owner's or designer's files

- local knowledge including discussions with those involved in the original construction or subsequent alterations and with the building maintenance personnel, and
- site inspection to check drawings and other information against site observations.

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.

C4.4.5.3 Foundation load/deformation behaviour

Where more reliable information on foundation capacity and/or stiffness is required, it may be possible to undertake a load test on an existing foundation. Typically, this is 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).

C4.5 Key Principles

Some key principles are embodied within the approach to assessment of geotechnical issues contained within these guidelines. These include understanding 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.

C4.5.1 Difference between assessment and design

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.

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.

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 expected 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.

In design, load and resistance factored design (LRFD) is typically applied. Loads and resistances are factored to provide a level of reliability that yielding or failure of soil will not occur. This also is likely to control deformations. In an assessment this is typically replaced by a displacement-based approach. The acceptable performance for geotechnical behaviour is a function of the consequence of the geotechnical-induced deformation/loads on the superstructure's life safety performance. 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.

Gazetas (2015) presents the case for going beyond conventional seismic failure thresholds and provides case studies that illuminate the benefits and limitations of "rocking isolation", for example.

The process of assessment is often iterative and there can be limited geotechnical information available at the early stages while critical mechanisms are being identified for targeted investigation. However, where limited information is available it is important that "consistent crudeness" is applied to the modelling and assessment, i.e. to avoid reporting analysis to a degree of accuracy that is inconsistent with the uncertainty of the input parameters.

The uncertainties and unknowns associated with assessment are typically greater than in 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. Section C4.4 discussed these uncertainties and ways they might be managed.

Due to the inherent uncertainty in geotechnical engineering and, in particular, in geotechnical earthquake engineering, engineers needs 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" (i.e. 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.

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.

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. Refer to Section C4.5.2 for further discussion on the use of probable capacity in the context of the geotechnical assessment and the selection of suitable geotechnical parameters.

C4.5.2 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, as has been noted in previous sections.

To recognise this situation the following approach has been adopted in these guidelines for assessing the probable capacity/resistance for geotechnical issues. Geotechnical capacity in these terms includes both strength/resistance and deformation and is represented in terms of an assumed relationship between strength/resistance and the resulting deformation, which needs to consider potential behaviour often well into the nonlinear range.

The probable strength (capacity) is taken as the ultimate geotechnical strength 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 C4.5.3 considers assessment and modelling of resistance/deformation.

C4.5.3 Resistance-deformation/shaking behaviour

C4.5.3.1 General

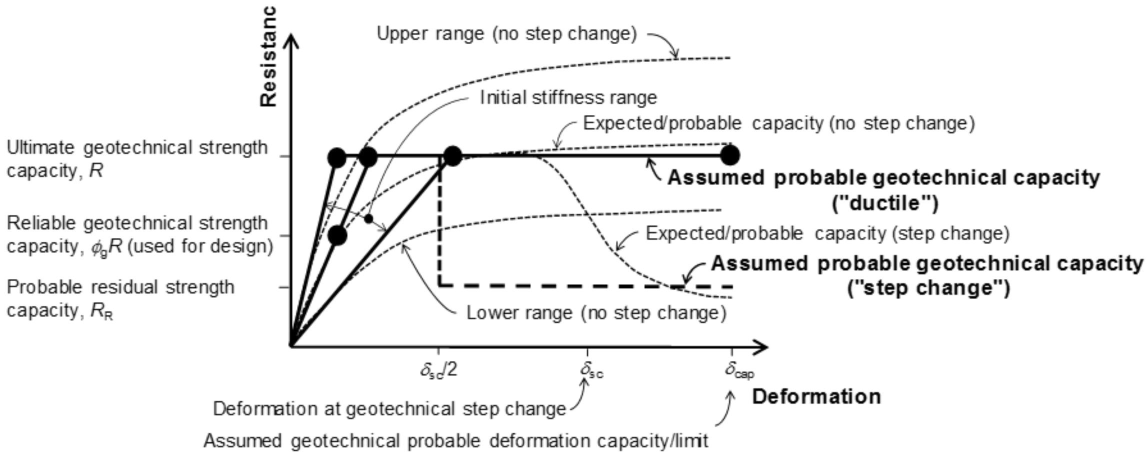
Consider the generic resistance-deformation and shaking relationships/models indicated in Figure C4.3. These might apply to the effect that a foundation soil, a foundation or a geohazard might have on the building, or how the resistance these provide to a building might be affected by increasing imposed deformation or earthquake shaking.

Figure C4.3(a) shows a generic relationship between resistance and increasing levels of deformation. The figure shows the probable geotechnical resistance models that are intended to be assumed for the situation where no "step change" in behaviour is expected and also when it is (refer Section C4.5.3.2 for a description of step change behaviour). The relevant features of these models are as follows:

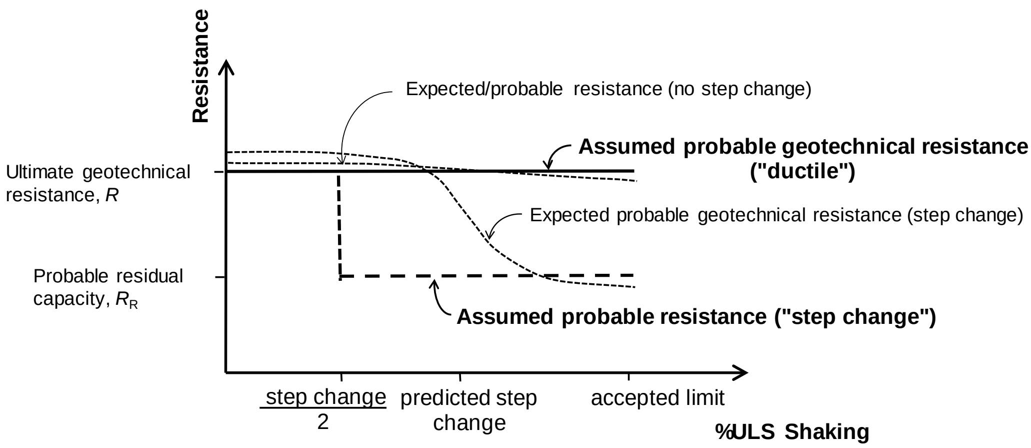
- A bilinear representation is considered adequate for most situations. This is referred to as "ductile" behaviour.
- The maximum resistance (i.e. probable strength capacity) is taken as the ultimate geotechnical strength capacity normally calculated for the purposes of design, but before application of the usual geotechnical strength reduction factors.
- The deformation limit of the model will typically be well beyond the deformations usually considered for design.
- When a step change in behaviour is expected it will be necessary to estimate the deformation at which this is expected and also to consider the probable residual strength capacity that might be available beyond the step change. In line with the assessment philosophy that has generally been adopted in these guidelines around step change behaviours, the deformation at which the step change is indicated is divided by 2 when defining the model. Beyond this halved deformation, the resistance is assumed to be limited to the residual capacity. The objective is to determine a *%NBS* score which has the resilience that is likely to be inherent in current new building design.

The resistance provided by some foundation soils or geohazards (e.g. liquefaction, slope stability, lateral spread) can be influenced by the dynamic effects of the earthquake shaking. Figure C4.3(b) shows a generic relationship between the resistance provided and increasing levels of shaking, illustrated here in terms of increasing %ULS shaking. This figure shows what is intended in the case of a predicted step change where resistance may be lost or significantly reduced, as the shaking level (intensity and duration) reaches a threshold value. A step change factor of 0.5 is also introduced to define this behaviour.

The uncertainties in the relationships/models could be large. It is recommended that the evaluation of the potential sensitivity of geotechnical issues assumes upper and lower ranges of initial stiffness (often twice and half respectively of the estimated values). The geotechnical engineer will need to advise the nature of the uncertainties and when sensitivity analysis of outcomes might be appropriate.



(a) Resistance versus deformation



(b) Resistance versus %ULS shaking

Figure C4.3: Generic resistance-deformation versus shaking relationships for geotechnical issues

When a residual capacity is expected to be maintained after a step change, the geotechnical engineer can either:

- assume the probable resistance is the residual resistance from the outset, or
- if the assessment is to be based on the pre-step change resistance, the deformation should be limited to 50% of the predicted deformation at the step change, as outlined above.

The geotechnical engineer should also nominate the probable deflection/deformation capacity (limit), δ_{cap}, beyond which the relationship is not expected to be valid.

This method is a pragmatic approach to what can be complex issues. It recognises that the primary geotechnical issue is not usually the available level of resistance available but whether or not there is likely to be step change behaviour and whether or not a residual capacity is expected post the step change.

The evaluation of the score for a SSW involving a geotechnical issue may be treated in a similar fashion, but without the expectation of a residual capacity.

C4.5.3.2 Derivation of soil-foundation models

Section C4.5.2 and Figure C4.3 outline the general principles for modelling soil-foundation behaviour for seismic assessment. This section sets out the steps to derive the soil-foundation model parameters.

Step 1 - Qualitative assessment

The first step is a qualitative assessment of the likely soil-foundation behaviour. Is it “ductile” behaviour or could a “step change” be expected?

- Ductile behaviour may be assumed if a step change in resistance is not expected or the resistance is not expected to decrease by more than 20% over the extent of expected deformations.
- Table C4.2 below identifies soil-foundation types which could exhibit step change behaviour.

Step 2 - Selection of parameters

The following guidance is provided for evaluating parameters to be applied in modelling soil-foundation behaviour. In evaluating these parameters due consideration must be given to soil response to the shaking and the dynamic nature of the applied loading (cyclic and reverse loading, push pull). NZGS/MBIE Module 4 - *Earthquake resistant foundation design* considers these factors in its guidance.

Ductile behaviour

Refer to Figure C4.3(a). Ductile behaviour is to be modelled as elastic-plastic. To evaluate this simple model the engineer must establish the following parameters:

- ***R*, ultimate geotechnical (strength) capacity**: this is the assumed limiting resistance provided by the soil-foundation with increasing deformation. It is the same value as is assessed for design before the design strength reduction factor is applied. Strength reduction factors are not applied in assessment of an existing building. NZGS/MBIE Module 4 provides guidance on evaluating R.
- **initial stiffness:** the initial stiffness assumptions will rarely prove to be critical in a seismic assessment but stiffness values may be requested by the structural engineer for inclusion in the structural modelling. When requested it is recommended that a range be provided. If it proves critical to the assessment of the behaviour of the building, refinement of the top or bottom end of the range can be undertaken at a later stage. Table C4.3 provides guidance for evaluating initial stiffness for various soil-foundation types.

- **$\boldsymbol{\delta_L}$, deformation limit:** this is the deformation limit over which the soil-foundation can be assumed to provide resistance R. Beyond this limit a reduction of more than 20% in R could be expected, or behaviour cannot be predicted. Table C4.3 provides examples of evaluation of δ_L for various soil-foundation types.

Step change behaviour

Foundation soils that are likely to degrade significantly in strength when subjected to earthquake shaking cannot be considered as "ductile" and will require special consideration, involving both the geotechnical and structural engineer. Such behaviour can lead to sudden loss in building support once a threshold level of shaking is exceeded. The threshold may occur as a result of deterioration in the strength of the soil/foundation and/or deterioration in ability to provide support due to dynamic effects. This is referred to in these guidelines as "step change" behaviour and, if it is judged that it could lead to a significant life safety issue for the building, may result in the limiting score for the building that determines its earthquake rating. It is the identification of potential step change behaviour in the building behaviour that should be the focus of the geotechnical and structural engineer.

Step change may involve a deterioration in resistance to a residual value. In such cases it may be appropriate to carry out the assessment based on the residual strength. If the resistance prior to the step change is to be relied on or is necessary to prevent a significant life safety risk, allowance will need to be made in the scoring to provide confidence that the risk of the step change occurring is at an acceptable level.

In its static condition and during lower levels of earthquake demand the ground is assumed to remain in a competent, stable state. With increasing earthquake demand the ground can gradually deform but at tolerable levels, with the capacity at yield exceeding demand.

In the range of earthquake demand (i.e. combinations of magnitude and peak ground acceleration) being considered there can be a threshold point (or a narrow "bandwidth") up to which gradual ground deformations may have occurred but suddenly, at further increasing demand, the ground or foundation performance abruptly deteriorates. In these guidelines this is termed a "step change" in geotechnical behaviour. The abrupt transition in geotechnical conditions may or may not have significant consequences for the foundation's integrity or the structure's stability (Clayton et al., 2014).

Examples of features that can lead to a geotechnical step change are:

- liquefaction – elevated pore water pressure at lower levels of earthquake demand can occur in liquefaction-prone soils; but over a small "bandwidth" of earthquake demand liquefaction triggering can occur and lead to an abrupt loss of soil shear strength. The consequence can be abrupt, large foundation deformation. For shallow foundations, the step change may manifest as a severe rotation and/or settlement. The severity of the soil and foundation deformation could be significantly exacerbated if lateral spread can also occur.
- slope instability – soil and rock slopes can withstand earthquake shaking with little or no deformation. However, at elevated levels of earthquake shaking they can reach a point where mass movement (e.g. soil slope failure, rockfall or cliff collapse) is expected.

- retaining walls – as for slopes, retaining walls can withstand a degree of earthquake shaking with little or no deformation. However, with increasing earthquake shaking there can come a point at which the wall fails. A wall supporting a foundation could fail leading to a step change in foundation support and large deformations in the structure. Similarly, a wall retaining land upslope of a building could experience abrupt collapse.
- foundation element failure – pull-out of a foundation element, such as an anchor or pile in tension, has the potential to lead to a geotechnical step change. However, there will often be a residual capacity which can be relied on, or the additional deformation that occurs in the structure as a result is tolerable. Step change behaviour could also be experienced with compression loading and sensitive soils.

Failing slopes or retaining walls can either remove foundation support (if the slope or wall is downslope of the structure) or cause soil/rock/debris lateral impact on vertical structural support members. Falling soil/rock/debris can also have direct life safety impacts on life outside and within a structure (e.g. occupants impacted or buried by rockfall) but as noted above this will not affect the earthquake rating for the building itself.

The severity of foundation deformation and consequences for the structure's stability are a function of:

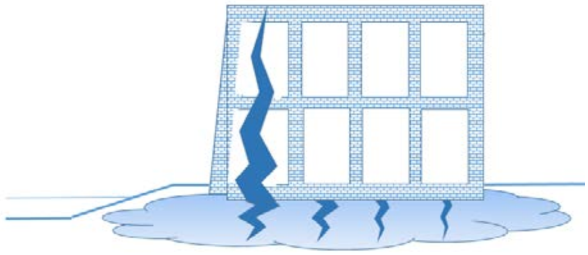
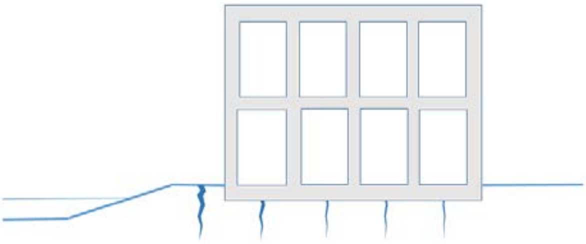
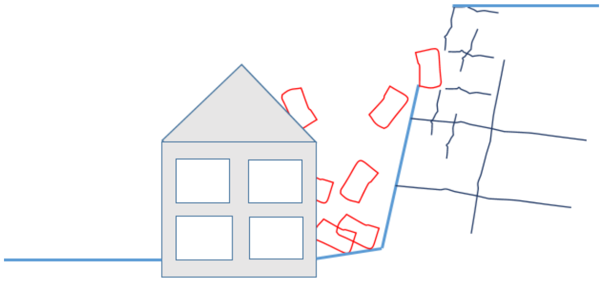
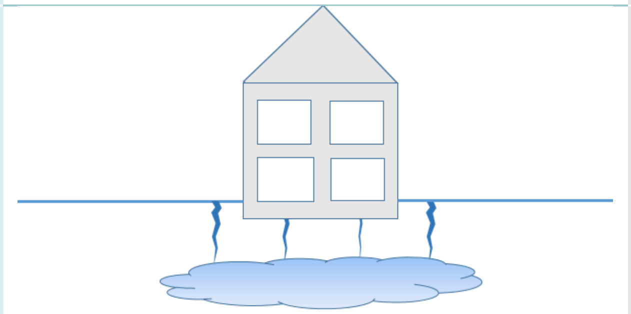
- the severity and nature of the ground deformation; how much of the structure's support system is affected, and
- the structure's resistance to foundation deformation or rupture.

In this regard, a structure on a mat foundation or well-tied footings is more resilient to ground deformation than a structure on discrete footings, although relatively high levels of differential settlement of individual footings may still be tolerable when the structure itself is well tied together.

Geotechnical step change will only be an issue for setting the earthquake rating if it in turn results in a step change behaviour of the building structure, i.e. a structural step change, and then only one that would result in a significant life safety hazard.

Table C4.2 provides some examples of buildings/sites and considers whether or not they have the potential for structural step change behaviour.

Table C4.2: Examples considering the potential for step change

Description		Step change potential?
Unreinforced masonry building on site subject to liquefaction and lateral (flow) spread		Likely to be a structural step change behaviour unless the structure above is well tied together
Building on site subject to coseismic slope movements		**Unlikely** to be a structural step change if the building and/or its foundation is well tied together
Light timber frame dwelling in a rockfall impact zone		Likely to be step change but **not** an earthquake rating issue
Light timber frame building on a site subject to liquefaction		**Unlikely** to be a structural step change

Note:

While many sites may be subject to seismic geohazards, these guidelines anticipate that few of these will result in a true step change in behaviour. In very few cases it is anticipated that a geotechnical step change will, in isolation, set the earthquake rating. More commonly, geohazards may tend to exacerbate pre-existing structural weaknesses or be shown not to have a direct effect on the life safety objective.

Refer to Figure C4.3. Step change behaviour can occur in two situations:

- when a rapid decrease in resistance is expected with increasing imposed deformation, and
- when a rapid decrease in resistance is expected at a particular earthquake shaking threshold.

In both situations it will be necessary to estimate the probable resistance available prior to the step change and the residual resistance available beyond the step change up to the limiting displacement δ_L and also either the deformation or the %ULS shaking at which the step change is predicted. Guidance for their evaluation is as follows:

- R, resistance pre step change: R is evaluated as discussed above and in NZGS/MBIE Module 4.
- deformation, δ_{SC}, or predicted %ULS shaking triggering step change: this is the deformation or intensity of shaking (%ULS shaking) at which the step change in soil-foundation behaviour is estimated to occur. In modelling, R is assumed to be available up to a deformation of $\delta_{SC}/2$ or %ULS shaking to trigger step change/2. Beyond these levels a residual resistance of R_R is assumed. The halving of the deformation (or %ULS shaking) to trigger the step change is to provide some resilience against the step change occurring. Table C4.3 includes guidance on evaluating δ_{SC}.
- R_R, residual resistance: Table C4.3 provides guidance on evaluating R_R.
- δ_L, deformation limit: δ_L is evaluated as discussed above and in Table C4.3.

Note:

The factor of 2 applied above can be considered as a deformation margin that needs to be applied if reliance is going to be placed on the pre step change resistance/strength capacity.

Example parameters

Table C4.3 provides example parameters. These parameters are not to be relied on for a specific situation. The geotechnical engineer is to consider the soil conditions and foundation details that are appropriate for the particular project and undertake specific assessment of parameters with due consideration of the effects of shaking and dynamic loading. Reference should be made to NZGS/MBIE Module 4 for guidance.

Table C4.3: Indicative soil-foundation modelling parameters

Soil-foundation type	Ductile or step change behaviour	Example initial stiffness, displacement at load = R		Example deformation limit δ_L	Example trigger for step change δ_{SC}	Example residual resistance R_R
		"Stiff" end of range	"Soft" end of range			
Shallow pad or strip foundation on granular soil Foundation width B	Ductile	Elastic analysis based on short term (immediate) soil stiffness	10% of B	30% of B		
Shallow pad or strip foundation on cohesive soil Foundation width B	Ductile Sensitive soils could exhibit step change	Elastic analysis based on short term (immediate) soil stiffness	5% of B	15% of B		
Pile foundation in granular soil Pile base diameter B	Ductile	Lesser of elastic analysis of pile base, or 10 mm. 10 mm assumes load is resisted by shaft resistance alone.	10% of B	30% of B		

Soil-foundation type	Ductile or step change behaviour	Example initial stiffness, displacement at load = R		Example deformation limit δ_L	Example trigger for step change δ_{SC}	Example residual resistance R_R
		"Stiff" end of range	"Soft" end of range			
Pile foundation in cohesive soil Pile diameter B	Ductile Sensitive soils could exhibit step change	Lesser of elastic analysis of pile base or 10 mm. 10 mm assumes load is resisted by shaft resistance alone.	5% of B	15% of B But not greater than 75 mm for shaft resistance		
Screw pile	Ductile	½ X the displaced measured in representative load tests	2x the displacement measured in representative load tests	30% of B		
Grouted ground anchor in tension	Step change with displacement possible depending on loaded soil/rock type.	½ X the displacement measured in representative load tests	2x the displacement measured in representative load tests Or Bar/tendon elastic stretch assuming resistance distributed along full bond length, plus 10 mm			Depends on soil/rock type and method of anchor construction. Could be ½ of the peak resistance in rock.
Foundation in/on soils prone to liquefaction or cyclic softening	Step change possible. Liquefaction potential analysis required along with assessment of consequences of liquefaction to foundation.				Liquefaction triggering analysis. Extent of assessed liquefaction must be sufficient to compromise foundation capacity.	Analysis considering liquefied residual soil strengths
Foundation on or above a slope prone to underslip as a consequence of seismic shaking	Step change possible. Seismic slope stability analysis required.				Seismic slope stability analysis	Zero if slope evacuation from beneath the foundation is predicted to occur. Allow for reduced support adjoining slip scarp.

C4.6 Consideration of SSI Effects

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.

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.

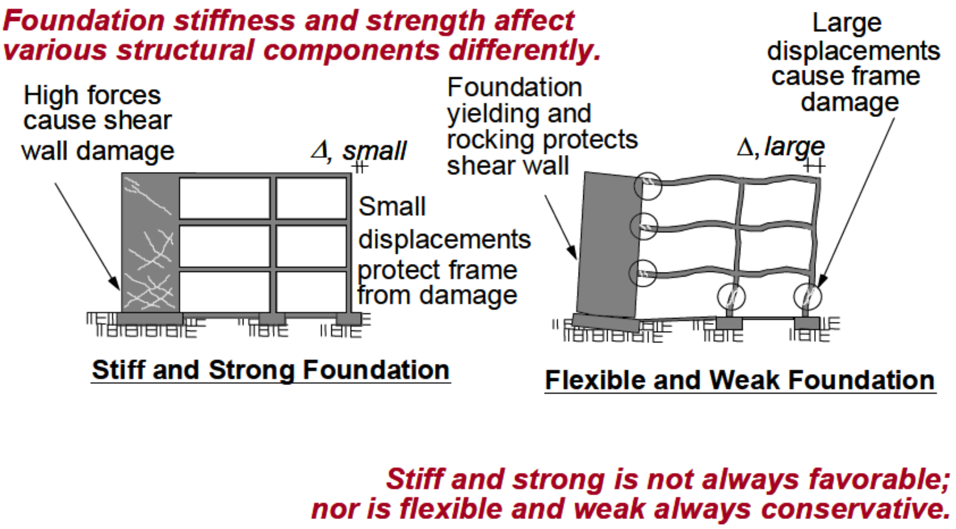


Figure C4.4: Influence of SSI on structural performance (figure adapted from Mahoney, 2005)

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

SSI effects are complex but can often be simplified for assessment; particularly initial screening to assess sensitivity of behaviour.

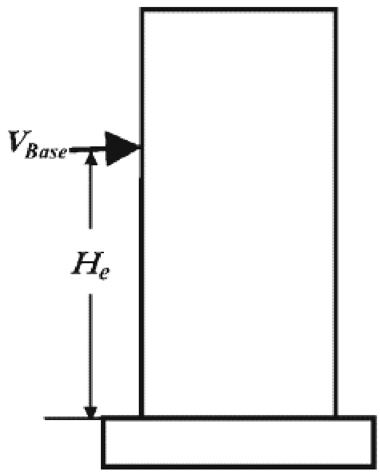
For example, this could be as simple as recognising that the soil support for a footing may not be rigid and reflecting on what this means for the rigidity of a supported column and its ability to receive flexural resistance/restraint at the base. This may influence the possible actions in the column 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., 2016). The amount 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.

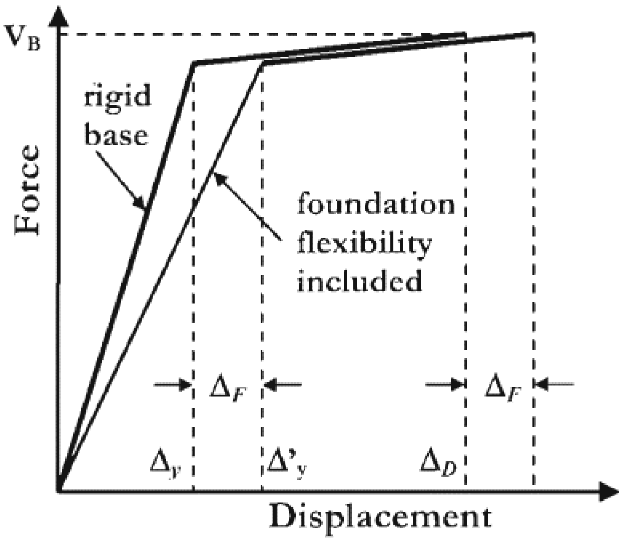
Note:

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 is confidence in the assessment of the parameters used.



(a) Structure



(b) Force-Displacement

Figure C4.5: Influence of foundation flexibility on displacement and ductility capacity in the structure

Complex 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 further information on each of the SSI analysis options refer to Appendix C4A. 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-1050-1, 2015.

Table C4.4: SSI analysis options

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.
Simplified flexible base model using linear Winkler springs	Shallow foundations Core walls Basement/part basements	The superstructure needs to be assessed for a fundamental period considering both fixed base and flexible base, i.e. building period shift due to foundation flexibility is to be considered. Consider whether sufficient number of springs have been included.
Simplified flexible base model using compression-only or tension-only Winkler springs	Rocking/uplift foundations The use of tension-only elements in dynamic analysis has risks with respect to stiffness matrix spikes and loss of energy via over-damping.	Examples: Kelly, 2009 for rocking foundation and Wotherspoon et al., 2004 for rocking shallow foundations. Consider a large range of soil spring parameters based on desktop study (e.g. 10,000 kN/m to 100,000 kN/m for vertical stiffness in gravel) in the initial sensitivity runs before specialist geotechnical inputs.
Flexible base model using nonlinear soil springs (either explicit nonlinear or equivalent linear springs) and site response analysis	Shallow foundations Core walls Basement/part basements	Equivalent linear springs need iteration between structural analysis and geotechnical p-y curve analysis. The use of rotational springs or multi-axial springs will need careful consideration of the assumed effective damping and equivalent linearisation of the nonlinear system.
Flexible base – nonlinear dynamic history (e.g. Nonlinear time history analysis computer packages)	Irregular system on complex soils and foundations Soil and foundation could potentially result in catastrophic step change behaviour.	The shape of the hysteresis curve should be realistic and reflective of the ground conditions. No additional damping should be included for foundation radiation damping, etc. Horizontal springs can artificially damp out ground acceleration – these should be used with care.
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	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 Figure C4.6) 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.

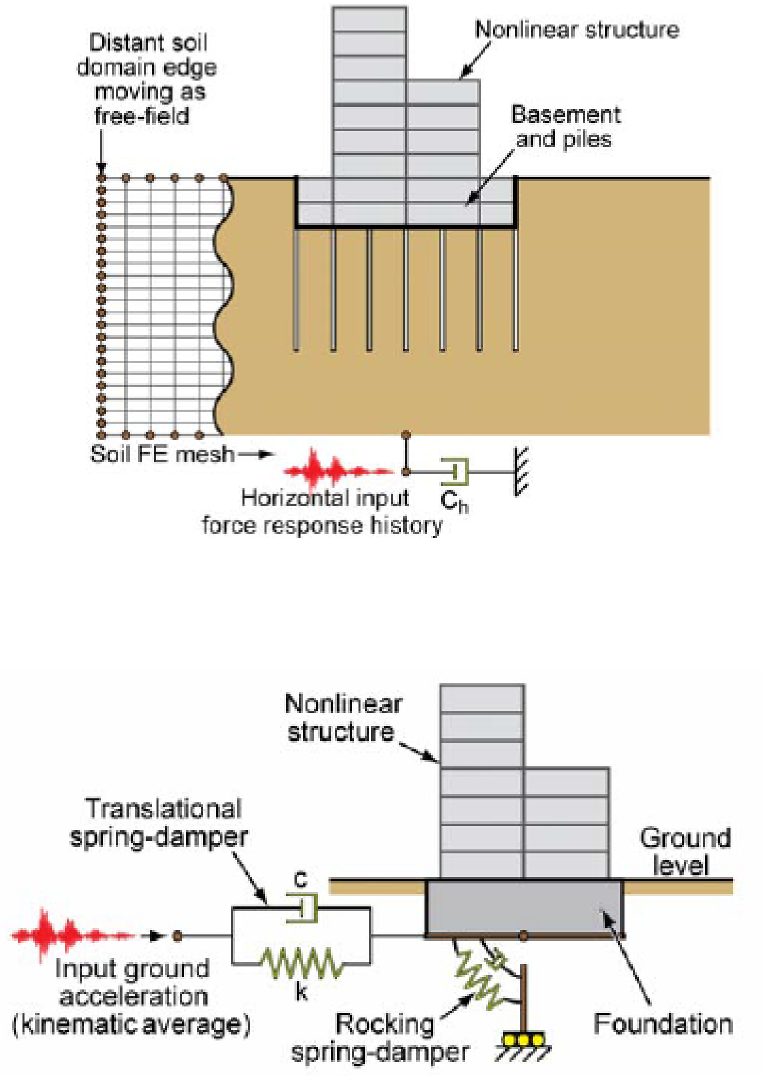


Figure C4.6: Direct and indirect SSI modelling (Deierlein et al., 2010)

Further information on SSI is provided in Appendix C4A.

C4.7 Calculation of *%NBS*

The basis for the earthquake rating for the structure is *%NBS*, which is the ratio of the ultimate (probable/expected/mean) capacity of the lowest scoring element/member/issue compatible with 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 clear that if there is to be consistency between the scoring of structural elements and scoring of geotechnical issues there must be consistency in the manner in which *%NBS* is determined for geotechnical issues (soil response and geohazards).

The determination of ULS demand/actions for geotechnical related issues also often uses a slightly different approach to that which is used in the assessment of the structural aspects. Whereas the structural engineer will determine ULS demand actions by loading a model of the structure with the stylised loadings/deflections defined for new buildings, the geotechnical engineer will often consider the demand in terms of particular earthquake parameters such as earthquake magnitude and peak ground acceleration (PGA). This very specific definition of demand can lead to a misunderstanding of what is expected if the shaking levels are higher.

The approach taken in these guidelines for scoring a geotechnical issue when demand must be expressed in terms of a particular level of earthquake shaking is as follows:

Step 1: Determine the earthquake characteristics that would be applied to the design for a new building for that particular geotechnical issue. These could include earthquake magnitudes and PGA. This is defined as ULS shaking.

Step 2: Establish the acceptance criteria (strength/deformation) that would lead to a significant life safety hazard in the structure or damage to neighbouring buildings.

Step 3: Analyse the geotechnical issue for the same magnitude earthquakes as for the ULS shaking to determine the PGAs at which the acceptance criteria are just exceeded. The lowest of these will be the PGA capacity unless a step change in behaviour has been identified for the particular geotechnical issue under consideration.

Step 4: If a step change is indicated, halve the PGA at the step change and take the lower of this value or the value determined in Step 3 as the PGA capacity.

Note:

The intention is that the margin of 2 to any identified geotechnical step change behaviour that could lead to a significant life safety hazard in the structure is reflected in the *%NBS* score for that issue.

Step 5: The *%NBS* score for the particular geotechnical issue is the ratio of the PGA representing the capacity and the ULS shaking.

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 C4.3) and, as a consequence, the geotechnical report cannot be completed 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 significance of the geotechnical contribution to the building's performance (refer to Section C1 for characteristics of the three project categories and Section C4.8.3.2 for the expected differences in reporting scope).

C4.8.3 Report content

C4.8.3.1 General

All geotechnical reports should document the following:

- an outline of the purpose, scope and limitation of the assessment
- a list of the existing information considered in the desktop study. Relevant information should be included in an appendix where appropriate.
- the scope of any site investigations undertaken. Results and location plan should be included in an appendix.
- 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.
- 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.

C4.8.3.2 By assessment category

For **structurally dominated projects** specific content should include:

- potential geohazards identified and the basis for their relevance to the seismic performance of the building. Engineering judgement by a suitably experienced engineer is a valid basis for deeming a geohazard non-relevant.
- geotechnical parameters for use in structural analysis and assessment including bearing capacities and, where required, simplified linear soil/foundation stiffnesses up to the relevant capacities.

For **interactive projects** specific content should include:

- potential geohazards identified, and a summary of their evaluation and relevance to the seismic behaviour of the building. For geohazards that potentially influence the behaviour of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard).
- geotechnical parameters provided to the structural engineer for use in analysis and assessment
- estimates of soil parameters provided to the structural engineer for before and after initiation of geohazard(s).

For **geotechnically dominated projects** specific content should include:

- potential geohazards identified, a summary of the critical geohazard, details of evaluation, and relevance to the seismic performance of the building. For geohazards that potentially influence the behaviour of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard) and should specifically address evaluation to ascertain if the geohazard results in a step change.
- where applicable, geotechnical parameters provided to the structural engineer for use in analysis and assessment
- where applicable, estimates of soil parameters provided to the structural engineer for before and after initiation of geohazard(s)
- assessment of the *%NBS* score for the geotechnical issue.

C4.8.4 Peer review

Peer review requirements should be discussed with the structural engineer. Suggested situations where peer review might be considered are summarised in Table C4.5. The peer reviewer's comments and the engineer's responses should be summarised separately and appended to the geotechnical report.

Table C4.5: Situations where peer review might be considered

Case	Peer review recommended
Structurally dominated project (in the absence of any other considerations described below)	X
Interactive project (in the absence of any other considerations below)	X
Interactive project IL4*	✓
Geotechnically dominated project IL4	✓
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	✓
Studies where the outcome of the structural assessment is sensitive to one or more of the following: • soil-structure interaction • 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. This appendix outlines the following techniques, listed below in order of increasing complexity:

- simplified hand analysis to evaluate influence of ground
- simplified flexible base model using linear Winkler springs
- flexible base model using equivalent linear springs
- simplified flexible base model using compression-only or tension-only Winkler springs
- nonlinear pseudostatic analysis with explicit nonlinear soil springs
- direct finite element modelling.

For most assessments only the simplified techniques will be required. 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 lower 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.

However, perhaps more significantly, there is potential for the building response as a whole 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, precision should not be assumed in any assessment of the interaction. However, the sensitivity to the expected response of the various assumptions should be understood. Parametric analyses to cover uncertainties in soil load-deformation 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 cannot be 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. This could be values of 50% and 200% of the expected value.
- 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.

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, damping resulting from plastic soil behaviour, etc.). However, these beneficial influences are the subject of ongoing research and therefore any reductions in seismic demand resulting from their adoption should be approached with caution.

C4A.3 SSI Modelling Approaches

C4A.3.1 Simplified hand analysis to evaluate influence of ground

The engineer can undertake hand calculation of the capacities of the soil, foundation and structure systems based on preliminary and conservative assumptions of the ground model. A comparison of these capacities in addition to the Simple Lateral Mechanism Analysis (SLaMA) assessment of the superstructure (described in Section C2) will indicate whether an inelastic mechanism will occur in the foundation or soil, or 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 bound 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 further 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 further more complex analysis of SSI will not be necessary.

C4A.3.2 Simplified flexible base model using linear Winkler springs

The SSI is modelled directly by linear soil springs, considering axial, shear and rotational flexibility. The modelling of the soil flexibility will allow a more realistic load distribution and transfer between the structure and supporting ground. This method is appropriate for both shallow and deep foundations (refer to Figure C4A.1). This approach is also referred to as the substitute or indirect method.

This approach is advantageous as it is consistent with how structural engineers typically used to consider SSI in new building design. Linear soil springs can also be incorporated easily into the analysis tools used by most structural engineers. In many cases, the structural response is not very sensitive to the soil spring values used. However, an upper/lower range of the spring flexibility should be considered. This range could be 50% to 200% times the expected value.

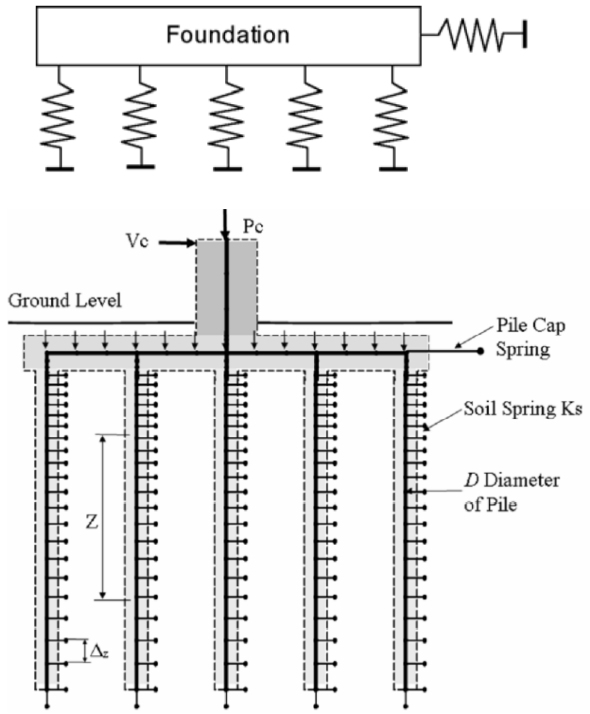


Figure C4A.1: SSI model for flexible base model using Winkler spring for shallow foundation and deep pile foundation

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

- 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 discretisation of the Winkler spring – typically, vertical springs are applied at 1 m centres. In some analysis packages, line or area springs can be applied.
- 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.
- The clear difference in including horizontal springs from vertical: horizontal springs, which are typically used for friction and/or passive soil resistance should be used with care.

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 to consider the pile foundation using a refined sub-model of the critical pile-superstructure and pseudostatic nonlinear analysis (refer to Section C4A.3.5 below).
- In some scenarios with significant nonlinearity expected in the piles (e.g. piles with a liquefiable layer), a pseudostatic nonlinear analysis is more appropriate.

C4A.3.3 Simplified flexible base model 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. As 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 sub-assembly model.

The nonlinear modelling of rocking foundations can be a complex area 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.

C4A.3.4 Flexible base model using equivalent linear springs

The nonlinear behaviour of the soil can be modelled using equivalent linear springs (NIST, 2012a and ASCE 41-13, 2014) for both linear dynamic analysis and nonlinear pushover analysis.

The equivalent linear model simplifies the nonlinear behaviour of soil by characterising the hysteresis loops by:

- an equivalent secant modulus, $G_{\mathrm{sec}} = \frac{\tau_{\mathrm{c}}}{\gamma_{\mathrm{c}}}$ where τ_{c} and γ_{c} are the expected amplitudes of shear stress and shear strain respectively

- an equivalent viscous damping ratio, ξ_{soil} that is directly proportional to the hysteretic energy dissipated, where:

$$\xi_{\mathrm{soil}} = \frac{1}{2\pi}\frac{A_{\mathrm{loop}}}{G_{\mathrm{sec}}\gamma_c^2} \qquad \text{…C4A.1}$$

The G_{sec} values used need to be checked and iterated with analysis results to ensure the equivalent secant modulus is taken at the tangent to the peak shear stress/strain point. In the absence of definitive justification ξ_{soil} = 5% is recommended to be used together with the structural performance factor, S_{p}, as per the building structural ductility capability.

C4A.3.5 Nonlinear pseudostatic analysis with explicit nonlinear soil springs

Modelling approach

In some scenarios where 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 pseudostatic pushover analysis (FEMA 440 (2005) and Cubrinovski and Bradley, 2009).

Two approaches for shallow and deep foundations are illustrated in the following figures.

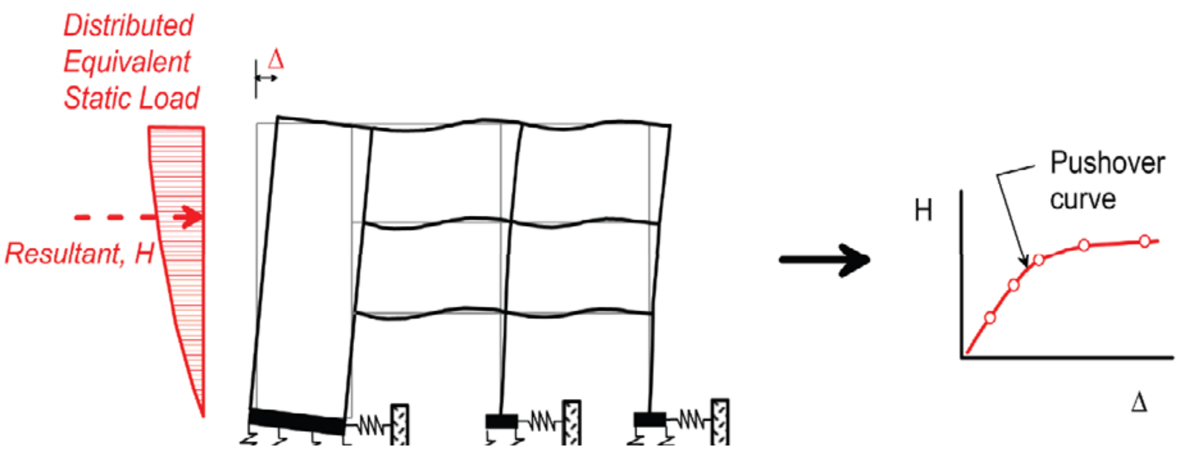


Figure C4A.2: Schematic illustration of a pushover analysis and development of a pushover curve for a structure with a flexible base (NIST, 2012a)

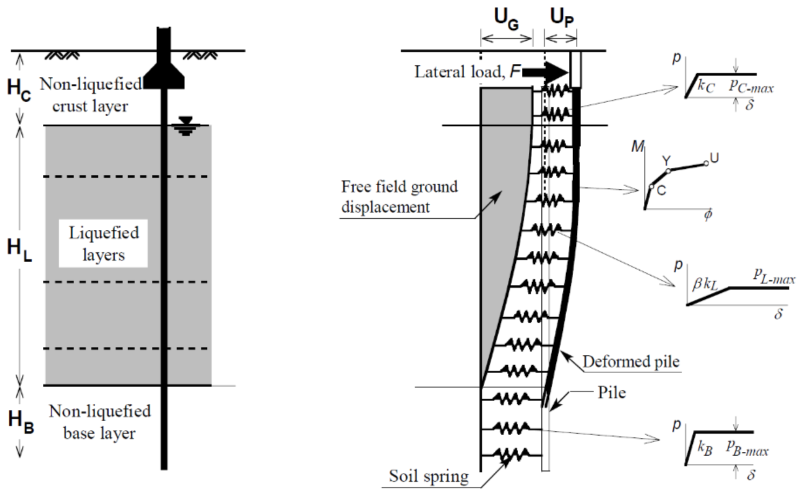


Figure C4A.3: Schematic illustration of a pseudostatic pushover analysis and development of a pushover curve for a bridge pier with a flexible pile base (Cubrinovski and Bradley, 2009)

Damping approach

Damping related to foundation-soil interaction can significantly supplement damping that occurs in a structure due to inelastic action of structural components. The damping from foundation-soil interaction is associated with hysteretic behaviour of soil (not to be confused with hysteretic action in structural components) as well as radiation of energy into the soil from the foundation (i.e. radiation damping). These foundation damping effects tend to be important for stiff structural systems (e.g. shear walls, braced frames), particularly when the foundation soil is relatively soft.

Due to the uncertainty associated with soil hysteretic and radiation damping, ξ_{soil} is limited to 10% and ξ_{soil} = 5% is recommended unless there is strong evident to suggest the use of a higher damping value. Refer to Section C2 for the treatment of additional soil damping (as ξ_{soil}) for nonlinear pseudostatic analysis.

C4A.3.6 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, 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 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 given the “black box” nature of such analysis. 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.

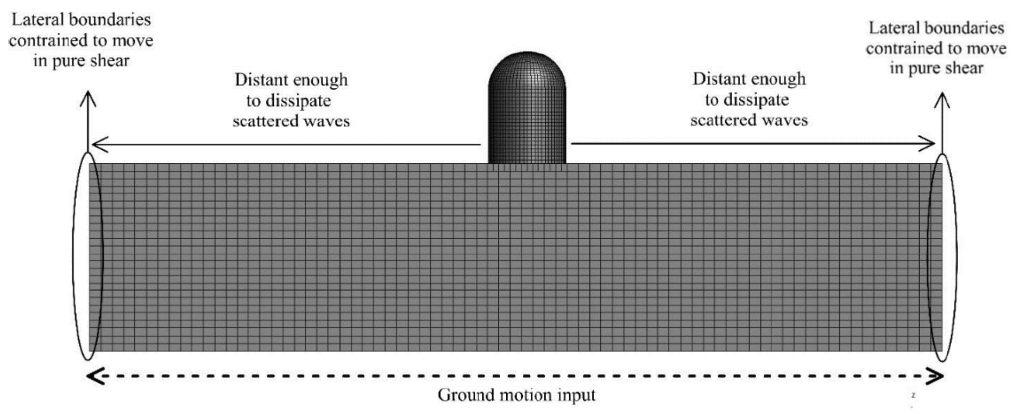


Figure C4A.4: Direct FE modelling (Cubrinovski and Bradley, 2009)

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 pseudostatic 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:

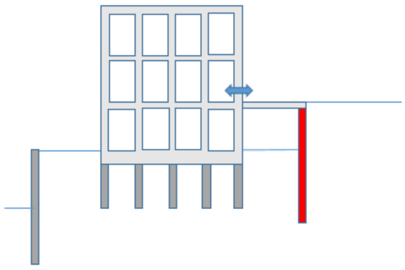
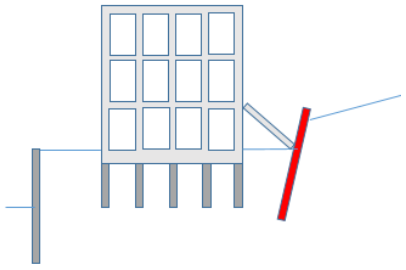
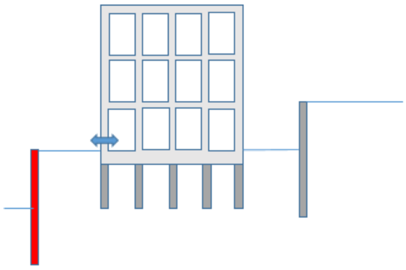
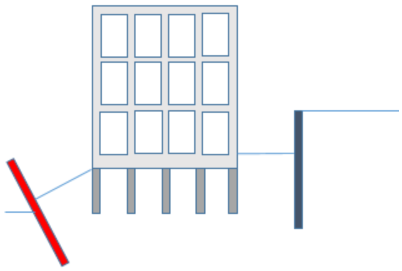
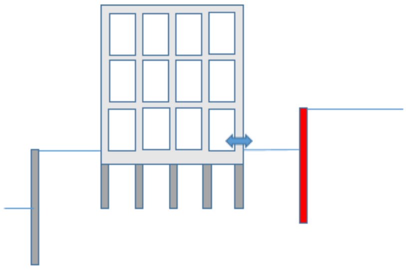
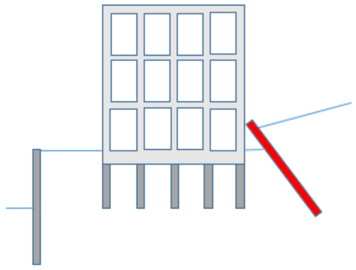
- Is there a significant risk that the wall may be of low capacity? (For example, it is a historic stone/masonry wall with no redundancy, or liquefaction is likely.) If yes, then assess the consequences for the structure's performance on the assumption that the wall may fail.
- Is there a significant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

As outlined in Section C4.5.3, the retaining wall's performance should be considered across a spectrum of earthquake demand. 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.

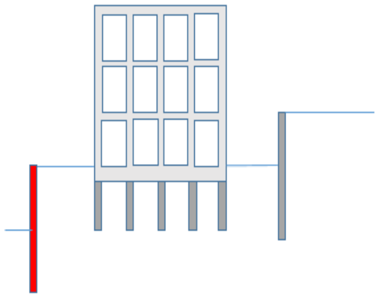
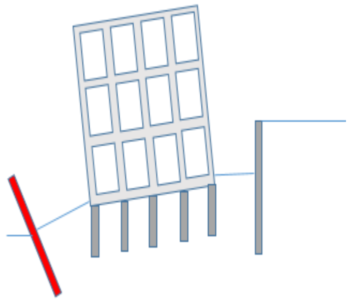
Table C4B.1: Examples of impact on emergency access/egress

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.

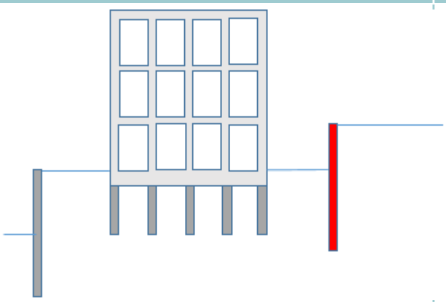
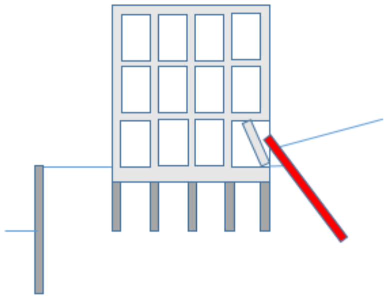
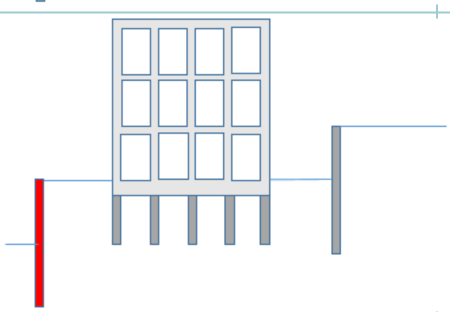
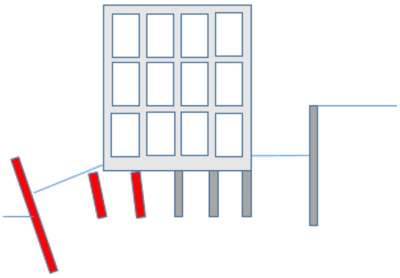
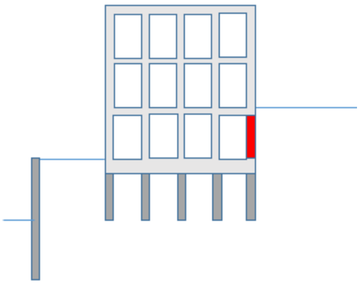
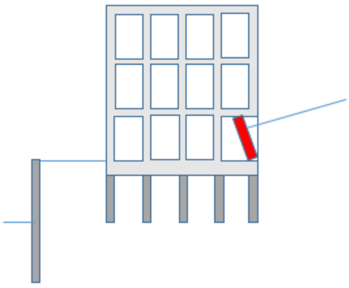
Table C4B.2: Example of loss of support to foundation soil

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		
Instability in a basement retaining wall		

Method A – Force-based assessment

For force-based assessment of retaining walls, refer to NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design.* This provides relevant guidance including how to allow for displacements.

C4B.4 Modes of Deformation

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

C4B.5 Seismic Loads

Refer to NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design* for guidance on deriving retaining wall seismic loads.

C4B.6 Methods of Assessment

Method B – Displacement-based assessment

If the forced-based assessment indicates that ductile-type behaviour of the wall can be expected but the magnitude of deformations could be an issue for the building, further assessment via a displacement-based assessment is likely to be required. Available methods use the results of Newmark sliding block regression analyses published by researchers such as Bray and Travasarou (2007) and Jibson (2007). In applying these methods the soil parameters assumed need to relate to those at the magnitude of strain/displacement being considered.

Method C – Dynamic numerical analysis method

For complex structures or walls posing a high risk or of significant monetary value, dynamic or time history analysis may be necessary. For further guidance on selecting and scaling appropriate earthquake records refer to NZS 1170.5:2004 and the NZTA Bridge Manual (2013). These complex analyses should only be considered if the results of the methods referred to in Method B above indicate critical issues and uncertainties that require further consideration.

C4B.7 Coincident Building and Earth Pressure Loads

The potential for coincidence of structural loading and retained soil loading should be considered (refer to Table C4B.4).

Factors to consider include:

- physical coupling between structure and walls (e.g. a wall propped by the building)
- the potential for the wall's and structure's natural frequencies to coincide (e.g. squat structure/tall wall), and
- the potential for liquefaction or other time-related effects such as lateral spread-generated kinematic loading coinciding with peak inertial loading.

Table C4B.4: Factors to be considered for loading on retaining walls

Details	Structure/ soil loading likely to coincide	Use conservative assumption or undertake specific analysis	Structure/ soil loading unlikely to coincide	Comments
Basement retaining wall	✓	x	x	
Wall retaining a building platform Structure $> 3H$ behind	x	x	✓	Consider slope of land between wall and structure and the presence of sensitive/ liquefiable ground
Wall retaining building platform Structure between $1H$ and $3H$ behind wall	x	✓	x	
Wall retaining building platform Structure $< 1H$ behind wall.	✓	x	x	
Where liquefaction derived pressures or lateral spread flow loads are already accounted for in design	x	✓	x	
Inertial load from wall elements excluding MSE	✓	x	x	

Note:
H = wall height

C4B.8 Estimation of Backfill Settlement

Loss of foundation support/settlement of backfill behind a retaining wall may occur through a number of mechanisms including erosion, densification and deformation at constant volume due to wall displacement/rotation.

- Erosion of backfill may occur where services carrying water with a significant head are ruptured due to otherwise acceptable seismically induced ground movement. Such effects can typically be assumed to be localised and unlikely to lead to collapse. The associated flooding effect can also cause increase in water pressure on retaining walls leading to collapse or deformation. Further investigation may be warranted in some circumstances.
- Densification will tend to occur during earthquake shaking in granular soils, particularly where this is poorly compacted. This settlement may be damaging to supported structures and can lead to wall deformation. However, unless the structure is particularly sensitive or the backfill is especially loose and deep, the risk of wall collapse can be assumed to be low.
- Significant settlement can be anticipated in retained ground if wall deflections occur during earthquake shaking or due to lateral spread. CIRIA C580 (Gaba et al., 2003) provides methods of estimating the magnitude of retained ground settlement and potential consequences for a range of structural types.

Appendix C4C: Slope Instability Hazard

(Interim Guidance)

C4C.1 Introduction

This appendix provides an overview of slope seismic instability as interim guidance, as there is currently no NZGS/MBIE module on this topic.

Slope stability assessment requires an understanding of a number of key attributes, including:

- slope geometry
- potential defects or structure/zones/planes of weakness in the soil/rock
- the groundwater regime
- soil/rock strength and potential for strength loss, including through liquefaction/cyclic softening, and
- the foundation system and/or retaining wall system embedded in the ground.

Note:

While an unstable slope may interact with the structure, the integrity of the structure or its life safety attributes may not always be compromised. In some cases the structure can withstand the predicted loss of support, displacements, impact or loading that arise from slope instability.

C4C.2 Scale

The scale involved in slope stability can be significantly greater than for other aspects of seismic assessment such as foundations or retaining walls. As a consequence it is important to look beyond the immediate site. Coseismic landslides and rockfalls can range from discrete, localised events up to massive events. Many contemporary examples of seismically induced slope instability can be found, including those associated with the Canterbury earthquake sequence of 2010-11.

C4C.3 Local knowledge

Stability conditions vary widely across New Zealand. Consequently, local knowledge is beneficial, particularly where calibrated by observed behaviour during past earthquakes or inferred from geomorphic evidence. Advice should be sought from an engineering geologist when detailed assessment of slope hazards is warranted.

C4C.4 Influence of ground conditions

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 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.5 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 41-13 (2014) 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 – Pseudostatic 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 pseudostatic 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.6 Defining Seismic Accelerations for Slope Stability Analysis

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

Ground shaking can be subject to significant amplification near the crest of steep slopes and ridgelines, such that PGA_{SITE} can be significantly greater than a PGA determined via NZGS/MBIE Module 1. NZGS/MBIE Module 6 - *Earthquake resistant retaining wall design*, MBIE (2014) 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 excacerbated 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. $D/B \geq 5$). 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.
- 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

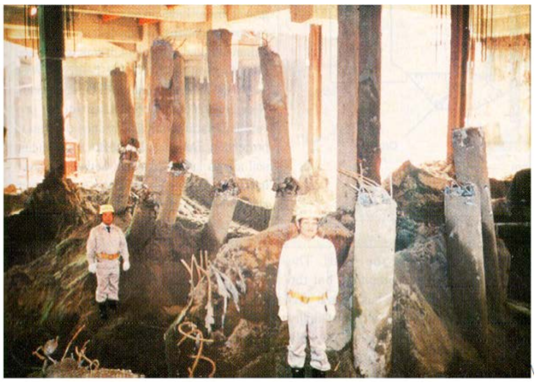
Reconnaissance reports of past earthquakes confirm that the seismic performance of buildings can be significantly influenced by the geotechnical performance of the supporting ground. Buildings have collapsed or been significantly damaged due to either foundation (shallow or deep) "failure" and/or liquefaction-induced 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.2I) 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 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)

The table 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	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
Driven timber tip armoured	1890 - 1920	End tapered and protected with steel to penetrate stiffer layers		
Driven steel piles			• 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
I or H sections	Typical post-1970s	Commonly bare steel, sometimes galvanised or coated		
Tube/pipe	Typical post-1970s			
Railway irons	1890s -	Cast iron prior to ~1910		
Driven concrete piles			• 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?
Precast	1915-			
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		
Bored piles			• 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	1860-	Multiple drilling techniques		
Straight grooved	1990-	Sides grooved with special tool after drilling		
Belled	1960-	Specialist technique		

Foundation type	Era	Brief description	Likely strengths	Likely weaknesses
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			• 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 grout to country bond with strain)
Drilled and inserted	1960-	Drilled and grouted hole, bar or strand anchors		
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. "Duckbill/Manta Ray"		
Shallow			• 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
Brick strip	1840-	Nominal widening, sometimes incorporating site concrete		
Concrete strip	1840-	Reinforced or unreinforced		
Ground beam	1950-	Reinforced, likely spreading point loads		
Isolated pad caisson	1840-	Reinforced		
Raft	1970-	Reinforced		
Domestic			• 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 ordinary	1840-	Rounds or squares excavated and concreted in place		
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: Initial Screening for Liquefaction Susceptibility

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

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 an initial screening tool.

Soils susceptible to liquefaction may substantially lose vertical load-bearing capacity during an earthquake. Loss of vertical support for the foundation causes large differential settlements and induces large forces in the building superstructure. These forces are concurrent with all existing gravity loads and possibly seismic forces during the earthquake.

ASCE 41-13 (2014), Table 8-1, reproduced below in Table C4E.1, provides a means of **initial screening only** of the ground profile to determine the site's liquefaction susceptibility. Refer to ASCE 41-13 (2014), Section 8.2.2 for further details.

Table C4E.1: Estimated susceptibility to liquefaction of surficial deposits during strong ground shaking (ASCE 41-13 (2014), Table 8-1)

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)			
		Modern <500 years	Holocene <11,000 years	Pleistocene <2 million years	Pre-Pleistocene >2 million years
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plan	Locally variable	High	Moderate	Low	Very low
Alluvial fan, plain	Widespread	Moderate	Low	Low	Very low
Marine terrace	Widespread	-	Low	Very low	Very low
Delta, fan delta	Widespread	High	Moderate	Low	Very low
Lacustrine, playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dune	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	Low	Unknown	Unknown
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)			
		Modern <500 years	Holocene <11,000 years	Pleistocene <2 million years	Pre-Pleistocene >2 million years
(b) Coastal Zone Deposits					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach, high energy	Widespread	Moderate	Low	Very low	Very low
Beach, low energy	Widespread	High	Moderate	Low	Very low
Lagoon	Locally variable	High	Moderate	Low	Very low
Foreshore	Locally variable	High	Moderate	Low	Very low
(c) Fill Materials					
Uncompacted fill	Variable	Very high	-	-	-
Compacted fill	Variable	Low	-	-	-

Note:

Adapted from Youd and Perkins, 1978

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.

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 ($\leq 10^3$ 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 ($\leq 10^3$ 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 ($\geq 10^6$ 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.