The Seismic Assessment of Existing Buildings

Technical Guidelines for Engineering Assessments

Revised Draft – 10 October 2016

Section C4 – Geotechnical Considerations
This document is intended to be referenced by the Earthquake Prone Buildings (EPB) Methodology being developed under the provisions of the Building (Earthquake-prone Buildings) Amendment Act 2016. It is also intended to be endorsed by MBIE for use as guidance under section 175 of the Building Act 2004 to the extent that it assists practitioners and territorial authorities in complying with that Act.

Document Access
This draft document may be downloaded from www.EQ-Assess.org.nz in parts:

1. Part A – Assessment Objectives and Principles
2. Part B – Initial Seismic Assessment
3. Part C – Detailed Seismic Assessment

Updates will be notified on the above website.

The document is expected to be published before the Act comes into force, when the regulations and EPB Methodology associated with the Building (Earthquake-prone Buildings) Amendment Act 2016 come into force.

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 use the feedback forms on www.EQ-Assess.org.nz to provide feedback or to request further information about these draft Guidelines.
Acknowledgements

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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 performance, and will likely require some geotechnical input to the DSA process Steps 1, 2 and 3 outlined in Figure C1.1, 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 performance to the geotechnical conditions (assessments are categorised as either “structurally dominated”, “interactive” or “geotechnically dominated” for this purpose, as outlined in Section C1).

Note:
The approach to inclusion of geotechnical issues outlined in these guidelines represents a fundamental change to that traditionally used for the design of new buildings. A geotechnical engineer will need to carefully consider the material in this section to make sure this approach is understood.

The lead assessor (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 detailed assessment 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 geohazards related to seismic activity that are relevant to life safety in structures
- provision of input to soil-structure interaction (SSI) models and consideration of SSI in seismic assessment
- assessment 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.
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 seismic rating under these guidelines is focussed on those aspects, including geotechnical influences, which will potentially lead to a life safety issue for the building.

The science and practice of geotechnical earthquake engineering is far from mature and is advancing at a rapid rate. It is intended that these guidelines and the New Zealand Geotechnical Society modules (refer 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 the guidance herein appropriately.

A comprehensive list of references is provided. Assessors are expected to be familiar with relevant documents and to know what is important for the life safety assessment of existing buildings. Assessment is not design.

Note: The assessment of earthquake performance of existing buildings is not for novice staff to undertake. A high degree of experience, competence, local knowledge and engineering judgement is required to properly (i) understand the scope of work required, (ii) understand the likely vulnerabilities of the soil-structure system being assessed, and (iii) interpret and act on information acquired during the steps of the assessment process.

C4.1.2 Relevant publications

C4.1.2.1 New Zealand geotechnical guidance

The New Zealand Geotechnical Society (NZGS), is working with the Ministry of Business, Innovation and Employment (MBIE), to develop a series of modules for new design (“the NZGS/MBIE modules”) that also include information relevant to the assessment of existing structures. Their content and status is summarised in Table C4.1.

Where the requirements of these guidelines that have a focus on the assessment of existing buildings differ (e.g. in the treatment of uncertainties) from the NZGS/MBIE modules which relate primarily to new building design, specific advice is provided in this section.

Note: The information regarding the status of each NZGS/MBIE module was correct at October 2016. Please check at www.nzgs.org for updates.
### Table C4.1: Summary of joint NZGS/MBIE geotechnical guidance modules

<table>
<thead>
<tr>
<th>NZGS/MBIE module (status at April 2016)</th>
<th>Description</th>
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</table>
| 1. Geotechnical earthquake engineering practice in New Zealand *(Published 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 investigation of potentially liquefiable sites *(Draft undergoing peer review)* | • Explains the importance of developing a geotechnical model for a site and describes the key issues to be considered  
• Guidance on planning geotechnical site investigations  
• Detailed description of various techniques available for sub-surface exploration; discussion of advantages and disadvantages of each |
| 3. Identification, assessment, and mitigation of liquefaction hazards *(Published May 2016)* | • Introduces the subject of soil liquefaction; describes the various liquefaction phenomena including lateral spreading  
• Includes discussion on clay soils and volcanic soils |
| 4. Earthquake resistant foundation design *(Draft undergoing peer review)* | • 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 *(In preparation – outline developed)* | • 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 |
| 5a. Specification of ground improvement for residential properties in the Canterbury region *(Issued September 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. Retaining walls *(In preparation – working group formed)* | • Will consider earthquake 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 8 Foundations and Geologic Site Hazards is a key geotechnical reference with respect to the assessment of existing buildings.

Chapter 8 presents general requirements for consideration of foundation load-deformation characteristics, seismic evaluation and retrofit of foundations, and mitigation of geologic site hazards. It covers:

- definition of geologic site hazards
- data collection for site characterisation
- procedures for mitigation of geologic site hazards
- soil strength and stiffness parameters for consideration of foundation load-deformation characteristics
- procedures for consideration of soil-structure interaction (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 with the New Zealand assessment methods outlined in these guidelines to ensure that the overarching philosophies are consistent. For example the New Zealand approach is 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 North American references regarding the modelling of soil-structure interaction (SSI) effects for the design of new buildings (e.g. NIST, 2011 and FEMA P-1050, 2016) 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 as some aspects of SSI damping have been included in the NZS 1170.5:2004 structural performance factor, $S_p$. 
### C4.1.3 Definitions and abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>%NBS</td>
<td>Percentage of new building standard as calculated by application of these guidelines</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone penetration test</td>
</tr>
<tr>
<td>Critical structural weakness (CSW)</td>
<td>The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential CSWs.</td>
</tr>
<tr>
<td>Detailed Seismic Assessment (DSA)</td>
<td>A seismic assessment carried out in accordance with Part C of these guidelines.</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element (refer to Appendix C4A.3.6)</td>
</tr>
<tr>
<td>LRFD</td>
<td>Load and resistance factor design (refer to Section C4.8)</td>
</tr>
<tr>
<td>M-O equation</td>
<td>Mononobe-Okabe equation (refer to Appendix C4C)</td>
</tr>
<tr>
<td>MSE</td>
<td>Mechanically stabilised earth (refer to Appendix C4C)</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak ground acceleration</td>
</tr>
<tr>
<td>Project categories:</td>
<td>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)</td>
</tr>
<tr>
<td>Geotechnically dominated</td>
<td>The structure response is likely to be governed by geohazards, ground behaviour and SSI. Step change is often a characteristic of ground and foundation performance.</td>
</tr>
<tr>
<td>Interactive</td>
<td>Geohazards, soil nonlinearity and SSI may have an influence on the critical structural mechanism(s)</td>
</tr>
<tr>
<td>Structurally dominated</td>
<td>The structural response is unlikely to be significantly influenced by geohazards, foundation soil nonlinearity or SSI up to the capacity of the structure</td>
</tr>
<tr>
<td>SCIRT</td>
<td>Stronger Christchurch Infrastructure Rebuild Team</td>
</tr>
<tr>
<td>Severe structural weakness (SSW)</td>
<td>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. For an ISA potential SSWs are expected to be noted when identified.</td>
</tr>
<tr>
<td>SLaMA</td>
<td>Simple Lateral Mechanism Analysis (refer to Section C2)</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard penetration test</td>
</tr>
<tr>
<td>SSI</td>
<td>Soil-structure interaction (also known as SFSI: soil-foundation-structure interaction)</td>
</tr>
<tr>
<td>Structural weakness (SW)</td>
<td>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 SW even though it is considered to represent an acceptable risk.</td>
</tr>
<tr>
<td>Step change</td>
<td>A yield point or narrow bandwidth in earthquake demand past which the ground or foundation performance abruptly deteriorates</td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate limit state</td>
</tr>
<tr>
<td>XXX%ULS shaking</td>
<td>Percentage of the ULS earthquake shaking implied for the ULS design of a new building in accordance with NZS 1170.5:2004. Typically referenced as the XXX percentage of the intensity of shaking (duration unchanged) with a return period as defined in NZS 1170.5:2004 for the ULS.</td>
</tr>
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</table>
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 by project category.

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 careful consideration, 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, 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. With this comes the responsibility on the lead engineer to be able to identify:

- the possible geotechnical issues (and the uncertainties in the assumed geotechnical conditions)
- their potential influence on the seismic performance of the building, and
- when it is appropriate – if not essential – to seek specialist assistance from an experienced geotechnical engineer.

This is important as there are a number of geotechnical hazards (geohazards) that can have a significant effect on a building’s performance but may not be readily apparent to a non-geotechnical engineer. These include:

- loss of ground strength and stiffness – liquefaction (sandy soils) and cyclic softening (clayey soils)
- land instability – lateral spreading, slope instability and instability of retaining walls affecting the support of the structure,
- fault rupture, complexities of near-fault effects, and
- other geohazards such as tsunami and tectonic movement leading to flood inundation (with the potential to cause debris impact and foundation scour), rockfall, slope and retaining wall instability from above. While these particular hazards do not affect the seismic rating for the building they may, nevertheless be important considerations if a holistic seismic assessment is to be achieved.

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 assessor 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.

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 geotechnical brief in collaboration with the geotechnical engineer is an important step in the assessment process (refer to Section C1, DSA process Step 1).

**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 (preferably 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 where 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.

An experienced structural engineer will know the assessment process and how best to communicate the staged approach involved to the client. He or she should also know how best to communicate the fact that a particular project may terminate at completion of the ISA or advance into more detailed work, with or without geotechnical engineering input, depending on the characteristics of the structure and the ground conditions.
C4.2.3 Geotechnical engineer’s role and required experience

The geotechnical engineer:

- provides specialist advice relating to SSI effects, geohazards and soils as they relate to foundation behaviour
- provides specialist advice relating to geotechnical uncertainties (refer to Section C4.4.7)
- 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
- works collaboratively with the structural engineer, and
- identifies SSI effects, geohazards and soil-foundation behaviour which could impact on the building’s seismic performance and life safety threat to occupants.

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.

Note:

The advising CPEng geotechnical engineer must have relevant experience in the field or have their work reviewed by a suitably experienced and qualified CPEng geotechnical engineer. The geotechnical professional should be competent with suitable relevant training and experience in foundation investigations and geotechnical earthquake engineering (refer also to NZGS/MBIE Module 2).
C4.2.4 Roles by project category

C4.2.4.1 General

On completing Step 3 of the DSA process (refer to Section C1) 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 C2 for a description of the project categories and to Section C1 for a description of the process).

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

Interaction should be sufficient to convey the general characteristics of the ground model and provide simple parameters for capacity or linear (up to limiting capacity) spring-type representation of the ground in the structural model, if appropriate.

Only the near fault factor, hazard scaling factor and site subsoil class and/or linear soil response may be material to the assessment. Most of the geotechnical work is expected to be completed by the end of the DSA process Step 3, although a degree of interaction between the disciplines is expected (as the need arises) through Steps 6 to 11.

Analysis should be undertaken as appropriate for the complexity of the structure: from a simple assessment based on nominal capacity to a more complex analysis using lower and upper characteristic linear elastic (to a limiting capacity) soil properties to consider SSI effects, as appropriate.
**C4.2.4.3 Interactive**

These projects generally require significantly more detailed geotechnical input and a level of geotechnical investigation consistent with the detail required. Significant interaction is expected between the geotechnical and structural engineering disciplines.

A staged approach can be employed, with re-evaluation on completion of each stage so that investigation is targeted for specific structural vulnerabilities. While much of the geotechnical work is expected to be completed by DSA process Step 3, further interaction between the disciplines is likely to be required through Steps 6 to 11.

The geotechnical engineer will convey the ground model and provide initial parameters. Analysis is undertaken as appropriate for the complexity of the structure, including SSI effects in a linear or multi-linear elastic (to a limiting capacity) manner using lower and upper characteristic strength/stiffness and geohazard derived force/imposed displacement details. Soil parameters are provided to the structural engineer for before and after initiation of geohazard(s).

Information for the structural model may, for example, be an estimate of settlement due to liquefaction effects in conjunction with the plastic deformation under a footing caused by structural inertial loads. The structural engineer is expected to provide details of constant (long-term) and cyclic loads so that ground and foundation response can be considered for appropriate load levels.

After a joint meeting, liaison may continue in the provision of modified parameters representing nonlinear behaviour, if appropriate, in detailed consultation with the structural engineer who will be accessing the effect that the geotechnical issues have on the seismic behaviour of the building, and in particular life safety.

**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. The structural engineer then confirms that a brittle structural step change follows directly the geotechnical step change and that this response occurs at a lower shaking level than any other (structural) mechanism.

It is anticipated that these cases will generally be identified by the end of the DSA process Step 3. Interaction between the structural and geotechnical engineering disciplines is expected through Steps 6 to 11 of the DSA process. However, the structural analysis required may be limited due to the governing effects of the ground behaviour and the geotechnical engineer may need to lead the assessment. The structural engineer will be required to confirm the structural response resulting from the geotechnical mechanisms.

Aspects to consider include:
- the likely range of responses considering lower, most likely and upper characteristic strengths/stiffness
- the time it takes for a geotechnical step change to occur during shaking, and whether this is likely to be after the primary structural actions
- uncertainty around the brittle nature of the foundation and/or soil behaviour, and consideration of alternative structural load paths (which may involve extremely large deflections).

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 Geotechnical Influences on Seismic Assessment

#### C4.3.1 General

The geotechnical issues and hazards (geohazards) that need consideration in a seismic assessment fall into two broad categories; those that need to be considered in establishing the seismic rating for the building and those that may potentially affect a building during or after earthquake shaking but which are not intended to be scored or included in the seismic rating.

All identified geohazards that fall within both of these categories are intended to be noted in an assessment, but only those that fall within the first are intended to be quantified, if establishing a seismic rating for the building is the extent of the agreed scope of the assessment. Guidance on when a geohazard might be relevant is provided in Sections C4.3.3 and C4.7.

**Note:**

Only geohazards that could affect the seismic rating must be identified, if the sole purpose of the assessment is to determine a seismic rating. However, other hazards, if known to be present, should be noted in the assessment with appropriate warnings regarding the extent of the investigation and their potential effect.

When setting a seismic rating for a building it is important to recognise that the focus is on the building and the extent to which life safety is affected by it. The seismic rating attaches to the building, and a life safety issue related to the building cannot exist in isolation from it. Therefore, any assessment of the effect of geotechnical issues and geohazards on the seismic rating must be focused on how these are likely to affect the building. This is the primary reason why these guidelines stress the need for a joint approach from both the geotechnical and structural engineer, refer to Section C4.2.

Traditionally, the geotechnical design for new buildings has been significantly influenced by the need to reliably limit soil deformations to meet serviceability requirements. This has
the consequence that many geotechnical engineers have not had to focus on the way in which geotechnical issues may manifest themselves under severe loading conditions. This, together with the focus on life safety as it is affected by the supported building (not necessarily the foundation soils in isolation) requires a fundamental change in approach for many geotechnical engineers. Comparatively large deformations in foundations/foundation soils may be, and often will be, tolerable and consistent with achieving the life safety objectives for the building. The onus is then on the geotechnical engineer to be able to estimate the relationship between resistance and deformation for the particular geotechnical aspect or foundation element, often well beyond what has traditionally been necessary. This is discussed further in Section C4.3.2.

**Note:**

The focus on soil/foundation deformation rather than strength or resistance in these guidelines, and particularly the need to identify how the geotechnical issues might influence step change behaviour in the structure from a life safety point of view, will represent a significant change in emphasis for many geotechnical engineers. The concepts outlined in these guidelines for detailed assessment, and the differences of these from the typical approaches to geotechnical design for new buildings will need to be fully understood if detailed assessments are to achieve the intended objectives, at an appropriate level of reliability.

Assessment of geotechnical issues is intended to be carried out using probable soil properties/resistances in line with the approach adopted for assessing the structure. Refer to Section C4.4.2 for further discussion.

**Note:**

Probable soil properties may be considered as “expected” values. They will be at a lower level of reliability than would be used for design but should allow for the uncertainties involved, especially when the building’s behaviour from a life safety perspective is likely to be sensitive to the predicted values.

The intent is that the probable soil properties/resistances be those that the geotechnical engineer would typically choose as the ultimate geotechnical parameters for the purposes of design, i.e. prior to the application of strength reduction factors. Consider, for example, the ultimate capacities of a pile type determined from on-site testing of 800 kN, 600 kN and 400 kN. The geotechnical engineer would typically take 400 kN as being a representative ultimate pile load from these test results and then apply a capacity reduction factor to obtain the reliable capacity for the purposes of design. The probable pile capacity from these results would be 400 kN.

### C4.3.2 Influence of foundation soils

Many soils have considerable ability to deform under increasing levels of seismic shaking/cyclic loading without significant loss of resistance. Structural engineers refer to this characteristic in structures as ductility and for some time now have made use of this to justify the ability of a structure to continue to sustain earthquake loads well after the nominated “yield” capacity in the structure has been exceeded.
When foundation soils exhibit ductile behaviour the structural engineer may be able to justify that the resulting large deformations in the structure are still compatible with achieving the required life safety objectives. In such cases the structural engineer could be expected to take the resistance/deformation relationship provided by the geotechnical engineer, evaluate the effect on the structure and confirm the seismic rating.

Foundation soils that will likely degrade significantly in strength when subjected to earthquake shaking cannot be considered as ductile and 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 as “step change” behaviour and, if it is judged that it could lead to a significant life safety issue for the building, it may result in the limiting score for the building that determines its seismic rating. It is the identification of potential step change behaviour that should be the focus of the geotechnical 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. This is discussed further in Section C4.4.3.

C4.3.3 Geohazard influences

Not all geohazards are required to be considered when evaluating the seismic rating of a building. In simple terms, geohazards that propagate from outside the building footprint should not be considered as influencing the seismic rating. The rationale for this is provided in Part A of these guidelines. However, if any geohazard that could affect the building or the site has been identified during the assessment process, it is expected that it will be referred to in the assessment report.

Table C4.2 summarises the geohazards to be considered, their potential relevance to the %NBS rating of a structure, and the potential foundation damage mechanisms that could affect the structure’s behaviour. Numbers in brackets relate to the relevant NZGS/MBIE module.

**Note:**

Even if a geohazard is present at a site it may not have material consequences for the structure’s behaviour or its seismic rating. The goal of the geotechnical assessment is to understand the consequences of the geohazard on the structure’s behaviour and its %NBS rating and in particular evaluate if its presence would likely cause a significant life safety hazard to develop on the building.

For example, an underslip or liquefaction may occur at a site, but if the foundations can maintain effective structural support the geohazard may not govern or be particularly influential on the %NBS rating.
Geohazards have the potential to cause large deformation of the ground supporting a foundation and the foundation itself, and/or can cause damaging lateral impact on a structure. They should be considered separately to the foundation deformation and inertial effects; although there may well be interaction (e.g. liquefied soil would affect soil damping properties). Identification of potential step change behaviour due to geohazards is as important in an assessment as that for the foundation soils noted above and should be the focus of the geohazard assessment.

Table C4.2: Summary of geohazards and their potential effects on a building

<table>
<thead>
<tr>
<th>Geohazard1</th>
<th>Geohazard considered in %NBS rating</th>
<th>Potential effect on a building</th>
<th>Differential settlement of foundations2</th>
<th>Lateral extension of foundations</th>
<th>Lateral impact of mass3 on structural support members</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fault rupture4 (1)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Bearing capacity failure (4), including liquefaction or cyclic softening, and lateral spread (2 and 3)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Retaining wall instability5 (6)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Underslip (7)</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Overslip including rockfall/boulder roll (7)</td>
<td>✗</td>
<td>-</td>
<td>-</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Tsunami6</td>
<td>✗</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
<tr>
<td>Dam break6,7</td>
<td>✗</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. Numbers in brackets relate to the relevant NZGS/MBIE module.
2. Includes differential settlement and/or foundation rotation.
3. Impacting mass may be soil, rock, debris and/or water.
4. Only direct effect of fault displacement on building is to be included in the assessment of the seismic rating. Earthquake-induced tectonic land subsidence and subsequent permanent inundation in coastal areas are other related geohazards.
5. Only retaining walls providing support to the building will affect the seismic rating. While the stability of retaining walls upslope of building may present a hazard to occupants within and adjacent to the building they will not affect the building’s rating. Their presence should nevertheless be noted in an assessment report.
6. Tsunami could potentially lead to inundation and scour leading to the undermining of foundations but the effects are not expected to be included in the assessment of the seismic rating.
7. Dam break is not covered in this guidance nor intended to be included in the assessment of the seismic rating. When necessary to consider for other purposes, guidance can be found in New Zealand Society on Large Dams (NZSOLD) guidelines.
C4.3.4 Soil-structure interaction effects

The response of a structure to earthquake shaking is affected by interactions between three linked systems: the structure, the foundation, and the soil underlying and surrounding the foundation. SSI refers to the interaction of responses of these systems in an earthquake, with each component having the potential to influence the earthquake response and deformation of the other. SSI is discussed in Section C4.4.5.

Note:
The terms soil-structure interaction (SSI) and soil-foundation-structure interaction (SFSI) are both used to describe this effect. These guidelines use the term SSI, following the US references in which the foundation is considered part of the structure. In these guidelines SSI describes the effects of the interaction and not the analyses that are used to quantify the effects which has often been the focus. The use of sophisticated analyses are not encouraged without simplified analyses and sensitivity analyses being carried out in tandem. Refer Appendix C4A.

C4.4 Key Principles

C4.4.1 Difference between assessment and design

Generally assessment is not design in reverse as the approaches followed, including the objectives, are not the same. This is particularly the case for consideration of geotechnical issues. For design the aim is to set limits for the geotechnical parameters below 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 little cost premium.

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 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.

Existing building assessment is primarily concerned with life safety and 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 onset of general damage are not the focus. This is in contrast with the approach for design.

The concept of geotechnical “failure” (i.e. where seismic demand exceeds a strength/resistance capacity for a given level of deformation) as utilised in new building design is typically replaced in an assessment 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 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 or reliability of the input parameters.

Due to the inherent uncertainty in geotechnical engineering and, in particular, in geotechnical earthquake engineering, assessors 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 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 bound 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 C4.4.2 for further discussion on the use of probable capacity in the context of the geotechnical assessment and the selection of suitable geotechnical parameters.

C4.4.2 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 and 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 and it may not be possible to establish mean soil properties, for example, with an equivalent degree of certainty. 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 pragmatic approach has been adopted in these guidelines for assessing the probable capacity for geotechnical issues. Capacity in these terms includes both strength and deformation and is best 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.

Consider the generic strength/resistance-deformation relationship indicated in Figure C4.2. This might apply to the effect that a foundation soil, a foundation or a geohazard might have on the building.

The uncertainties in the relationship could be large. It is not unusual in the evaluation of the potential sensitivity of geotechnical issues to assume upper and lower bounds of initial stiffness and sometimes strength to be twice and half respectively of the estimated values.

It is proposed that the probable strength capacity be taken as the ultimate resistance capacity usually determined for the purposes of design, before resistance (strength reduction) factors have been applied.

If ductile behaviour is expected then this resistance may be assumed to be maintained. Ductile behaviour may be assumed if there is not expected to be a step change in resistance or the resistance is not expected to decrease by more than 20% over the extent of deformation expected.

When a step change is expected the “usable” deformation (probable deformation capacity) should be assumed to be 50% of the predicted deformation at the step change.

![Figure C4.2: Strength-deformation relationship for a generic geotechnical issue](image)

When a residual capacity is expected to be maintained after the step change, two approaches are possible. Either assume the probable resistance is the residual resistance from the outset or, if reliance is to be made of the pre-step change resistance, then the deformation capacity should be limited to 50% of the predicted deformation at the step change, as outlined above.

The geotechnical engineer should also nominate the limiting deflection/deformation, $\Delta_L$, 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 often the 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 representation shown in Figure C4.2 is of a monotonically loaded system. It is recognised that the step change behaviour may result from dynamic effects and this should be taken into account when evaluating when, and if, a step change is expected. For some geotechnical issues it may be more appropriate to express the relationship in terms of resistance and %ULS shaking. Refer to Section C4.4.4 for further discussion.

The evaluation of the score for a severe structural weakness (SSW) involving a geotechnical issue may be treated in a similar fashion to that outlined above for a step change but without the expectation of a residual capacity.

C4.4.3 Step change behaviours

As discussed in Section C4.3.1 the primary focus of the geotechnical engineer should be to identify non-ductile behaviour in the foundation/soils and in particular step change behaviour and to reflect this in the %NBS scoring as outlined in Section C4.4.4.

In its static condition and during lower levels of earthquake demand the ground can remain in a competent, stable state. With increasing earthquake demand the ground can gradually deform but at tolerable levels, with the “factor of safety” typically greater than unity.

In the range of earthquake demand being considered there can be a threshold point (or a narrow “bandwidth”) in earthquake demand (i.e. combinations of magnitude and peak ground acceleration) up to which gradual ground deformations may have occurred but suddenly, at further increasing demand, the ground or foundation performance abruptly deteriorates.

This is termed a “geotechnical step change” in 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 and 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.

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 seismic 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 are 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 seismic 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 C4.3 provides some examples of buildings and their sites and considers whether or not they have the potential for step change behaviour.

### Table C4.3: Examples considering the potential for geotechnical step change

<table>
<thead>
<tr>
<th>Description</th>
<th>Step change potential?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unreinforced masonry building on site subject to liquefaction and lateral (flow) spread</td>
<td>Likely to be step change behaviour unless the structure above is well tied together</td>
</tr>
<tr>
<td>Building on site subject to coseismic slope movements</td>
<td>Unlikely to be step change if the building and/or its foundation is well tied together</td>
</tr>
</tbody>
</table>
### Description

<table>
<thead>
<tr>
<th>Description</th>
<th>Step change potential?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light timber frame dwelling in a rockfall impact zone</td>
<td>Likely to be step change but <strong>not</strong> a seismic rating issue</td>
</tr>
<tr>
<td>Light timber frame building on a site subject to liquefaction</td>
<td><strong>Unlikely</strong> to be step change</td>
</tr>
</tbody>
</table>

### Note:

While many sites may be subject to 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 seismic 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.

### C4.4.4 Calculation of %NBS

The basis for the seismic rating for the structure is %NBS which is the ratio of the ultimate (probable/expected/mean) capacity of the lowest scoring element/member compatible with a significant life safety hazard, to the actions expected when the structure is subjected to the demands set for the ULS defined 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 concept of a mean capacity in relation to geotechnical soil parameters is not one that has much meaning to geotechnical engineers. Nor is the concept of an ULS demand for geotechnical issues, and what it is intended to achieve, entirely clear to either geotechnical or structural engineers. This is because there is currently not a design Standard for new buildings that establishes, in anything other than general terms, what is mean by this term for geotechnical considerations. As outlined in Part A, inherent in the approach for design of new and assessment of existing buildings, is the concept that specific demand and acceptance criteria points are checked (for practical purposes these are few in number) but the expectation is that the risk in earthquake shaking generally, when considered across all levels of shaking, will be acceptable.
Note:

Geotechnical analysis for new design is typically undertaken assessing behaviour (e.g. liquefaction analysis, slope displacement) at a specific shaking level. For a detailed assessment it is appropriate to relate the scoring of a geotechnical issue against this level but also have due regard for the effect of larger levels of shaking. The indication is that the majority of the life safety risk for typical (i.e. IL2) new buildings results from shaking between ULS shaking and twice this level.

The amount of analysis required for assessment could be reduced using judgement at the upper end of the range, i.e. assessment of the potential for step change mechanisms could be limited to shaking levels up to 2 x %NBS / 100 x 100%ULS shaking. Such an approach may require iteration to the final value of %NBS (either as existing or as a retrofit target) but nevertheless may prove to be useful in some situations.

The approach taken in these guidelines is to relate the ultimate capacity for geotechnical issues to the geotechnical ultimate capacity used as a basis for design. This is discussed further in Section C4.4.2 including the adjustments that need to be made to allow for step change behaviours and the required focus on deformation rather than on reliable strength/resistance.

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 styled 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 earthquake 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,

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 PGAs representing the capacity and the ULS shaking.

C4.4.5 Consideration of SSI effects

SSI effects may have significant influence on the seismic performance 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.

Assessors 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 C4.3 illustrates a simple example of the range of structural responses as a consequence of the soil strength/stiffness adopted.

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.

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.

### Figure C4.3: Influence of SSI on structural performance (figure adapted from Mahoney)

SSI effects can be complex, but may also be relatively simple to include in an assessment.

For example it 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. Typically, sensitivity is tested at one half and twice expected values.
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.

While the added flexibility of SSI is generally considered, the beneficial effects of foundation radiation damping and kinematic interaction should not be included in the SSI modelling.

![Diagram](image.png)

**Figure C4.4: 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. FEMA P-1050-1, 2015.
### Table C4.4: SSI analysis options

<table>
<thead>
<tr>
<th>SSI analysis option</th>
<th>When to use/not to use</th>
<th>Comments</th>
</tr>
</thead>
</table>
| 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/uptilift foundations  
  The use of tension only elements in dynamic analysis has risks around stiffness matrix spikes and loss of energy via over-damping | Example: 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/or site response analysis | Similar to simplified SFSI model with Winkler springs | 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 linearization 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 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 – 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 | A robust process of 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 C4.5) 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 peer reviewed.
C4.4.6 Coincidence of mechanisms

Consideration should be given to when in the earthquake shaking sequence the structural and geotechnical influences may become significant, and whether or not these interact (coexist) and compound the threat to the structure.

For example, the effects of liquefaction may manifest late in the earthquake shaking sequence or even after shaking has ceased. Therefore, it may not be appropriate to consider simultaneous application of reduced soil resistance from liquefaction and the full shaking intensity on the building. Further discussion on the timing of liquefaction is given in NZGS/MBIE module 3.

C4.4.7 Managing uncertainties

The collaborative and iterative process described in Figure C4.7 (refer to Section C4.5.3) takes into account the management of uncertainties, as discussed below.

The design of new building foundations has to allow for uncertainties associated with the nature and behaviour of soils. These uncertainties are allowed for by:

- selecting appropriate strength reduction factors and load factors
- testing the building analysis with a range of soil stiffness values (sensitivity analyses), and
- verification testing during construction.
However, the assessment of existing building foundations may involve considerably more uncertainties depending on the availability and quality of “as built” information and investigation data. These uncertainties could include:

- 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. 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 seismic rating for the building. Identified critical uncertainties related to the critical structural weakness (CSW) or other low scoring structural weaknesses (SWs) are likely to require further 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 performance of this foundation (e.g. bearing capacity, appropriate strength reduction factor, ground stiffness, foundation load-displacement)
- 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.

The uncertainties involved with quantitative assessment of geotechnical issues are mitigated to some extent by their inherent ductile behaviour. Often the over or underestimation of soil/foundation strength will be more than compensated for by the available deformation capacity. Although the deformation capacity will have its own set of uncertainties deformation of the foundation soils will rarely limit the seismic rating of a building unless a step change is indicated.

**Note:**
If the structural assessment concludes that the foundation performance (or site response; e.g. liquefaction triggering) is unlikely to be material to the building seismic rating as it relates to life safety, further investigation and assessment of that foundation is unlikely to be necessary. Unless a step change is indicated it is unlikely that a life safety issue will be caused by a geotechnical issue.
C4.5 Assessment Process

C4.5.1 General

As the seismic assessment of a building should consider the interaction of the soil, foundation and structure, it requires interaction between the geotechnical and the structural disciplines as previously noted in this section.

Figure C4.6 illustrates the three key stages in this process:

- project definition
- assessment (including the geotechnical desktop study and geotechnical analysis and assessment), and
- reporting within the DSA.

![Figure C4.6: Project definition, assessment and reporting stages](image-url)
C4.5.2 Stage 1 – Project definition

This first stage of the process outlined in Figure C4.6 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 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.

C4.5.3 Stage 2 – Assessment

The initial part of the assessment involves a geotechnical desktop study to identify potential geotechnical issues that could affect the building’s seismic performance. Refer to NZGS/MBIE module 2 for guidance.

As indicated in Figure C4.7 the assessment stage is an iterative process between the geotechnical and structural engineer identifying and refining the understanding of the issues (“Define”) and assessment (“Assess”).

Refer to Section C4.2 for guidance on the relative roles and responsibilities during this phase, and Section C4.2.4 in particular for the scope of assessment for each project category. Specific guidance on probable capacity, site characterisation, identifying and characterising geohazards, assessing foundation performance and considering SSI effects is contained in Sections C4.1 to C4.4.

![Figure C4.7: The iterative assessment process](image-url)
This iterative approach involves starting with a simple, conservative linear assessment and then following this, if warranted, by progressively more sophisticated analysis using more detailed information (refer to Figure C4.8 following for the steps involved).

During this process the geotechnical engineer will provide input to the structural engineer’s assessment. However, the geotechnical report cannot be finalised until the SSI assessment has been completed. Interaction between the structural and geotechnical engineer should occur after each stage of analysis and assessment. It may also be appropriate to include the client in some instances.

**Figure C4.8: Steps involved in the geotechnical analysis**

An outline of a typical meeting agenda is as follows:

- **Inputs:**
  - Conclusions of geotechnical desktop study and walkover inspection
  - Results of geotechnical and structural review and analysis, and assessment to date.
• **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.
  - Consider what further analysis and assessment is required and how best to undertake this.
  - Consider the current uncertainties, how they are likely to impact on the reliability of the setting of the seismic rating and, if appropriate, the cost/benefit of further investigations to reduce these uncertainties.

• **Output:**
  - Updated list of identified geotechnical issues noting whether or not they impact on the building’s %NBS
  - Agreement as to which category describes the %NBS assessment of the building: structurally dominated, interactive, or geotechnically dominated
  - Agreement of the analyses that will be carried out
  - Agreement of what, if any, site investigations will be undertaken
  - Agreement of the geotechnical parameters required as input to the structural analysis and the form in which these parameters will be provided.

### C4.5.4 Stage 3 – Reporting and peer review

The geotechnical report should include:

- results and conclusions of the desktop study and any site investigations
- identification of geotechnical issues and their importance to the seismic rating of the building
- geotechnical parameters to be adopted by the structural engineer in analysis and assessment
- any identified geohazard influences from outside the site that are not included in the assessment of the seismic rating, and
- risks and uncertainties.

Refer to Section C4.8 for more details on report content and when peer review is recommended.

If the assessment is geotechnically dominated or interactive, the geotechnical engineer and structural engineer should present their conclusions jointly to the client.
C4.6 Site Characterisation

C4.6.1 General

Refer to NZGS/MBIE module 2 for general guidance on site characterisation including data acquisition, site definition, ground modelling, ground motion, and the scoping and implementation of a site investigation. Refer to Section C4.1 for guidance on SSI effects.

C4.6.2 The ground model

The ground model is a cornerstone of geotechnical site assessment. An appropriately detailed ground model will efficiently relay the geotechnical and geohazard information relevant to the project.

The ground model can be an annotated drawing or cross section that 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).

Refer to Appendix C4B for further guidance on developing the ground model.

C4.6.3 Site investigation

The purpose of the investigation of an existing structure is to characterise the ground conditions that the building is founded within. Physical investigation of foundations is sometimes necessary to confirm foundation dimensions and geometry. This may include local excavation around foundations or piles/pile caps. 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.

Where more reliable information on foundation capacity or response 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).

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)
- Pile Integrity Test methods to estimate the length and condition of a pile.

These methods can offer a relatively convenient and cost-effective investigation method. However, calibration against independent (preferably physical) methods is recommended, particularly where structure performance is sensitive to results.
C4.7  Geohazard Identification and Characterisation

C4.7.1  General

Refer to NZGS/MBIE module 1 and ASCE 41-13 (2014) Chapter 8 for general guidance on geohazards.

Geohazards are also described in Section C4.3.3 of these guidelines. As noted in that section, the influence of geohazards in a structural assessment can be wide ranging and may vary markedly over the spectrum of earthquake demand considered in a DSA. It is the understanding of the consequences of the geohazard(s) on the structure’s behaviour and its (%NBS) seismic rating that are the goals of the geotechnical input to a DSA.

Note:

Typically, for routine assessments it is not possible to reliably quantify earthquake-induced land or foundation deformations. Impressions of susceptibility and qualitative appraisal of deformation risk and severity are normally what the geotechnical engineers provides. Accordingly, the assessment of earthquake performance of existing buildings requires a high degree of experience, competence, local knowledge and engineering judgement.

Assessors should address the following questions. This should be done initially via a screening process conducted using existing information acquired from a site inspection and desk study:

- What are the potential geohazards the site could be affected by?
- What are the consequences to the structure if the geohazard occurred? In particular, is the geohazard material to the life safety performance of the structure or the occupants?
- What are the effects of combinations of geohazards?
- Does the geohazard lead to a step change in soil and/or foundation performance?
- Is there enough information available to answer the above questions satisfactory? If not, what is the scope of work required to obtain the essential information?
- Is the geohazard likely to affect the seismic rating?

Note

It is not intended that all geohazards likely to affect a site be identified as part of a DSA. However, geohazards that that could influence the seismic rating will require specific consideration. If, in the process of carrying out the DSA, other geohazards are identified these should be reported as potential hazards with a note to record their affect and that they have not influenced the seismic rating.

C4.7.2  Fault rupture

Refer to NZGS/MBIE module 1 for guidance on fault rupture hazard. Lateral and vertical differential deformations and their likelihood should be estimated for buildings crossing faults.
Note:
Experience in earthquakes suggest that fault traces can be diverted by the presence of buildings so that the full differential effect of any fault displacement may not be seen by a building. Also the position 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. Buildings located close to known faults, but not across them, are unlikely to be significantly affected by fault displacements.

C4.7.3 Bearing capacity exceedence
Refer to Section C2 and Section C4.4.

C4.7.4 Liquefaction, cyclic softening and lateral spread
Refer to NZGS/MBIE module 1 for general comments on liquefaction, cyclic softening and lateral spread hazard, and Modules 2 and 3 for detailed guidance.

C4.7.5 Retaining walls
Appendix C4C: Assessment of retaining walls (interim guidance) is provided as interim guidance on the assessment of existing retaining walls.

Note:
NZGS/MBIE module 6 Retaining walls, in preparation, will cover earthquake resistance design of retaining walls.

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).

C4.7.6 Slope instability
Refer to NZGS/MBIE module 1 for general comments on landslide and rockfall hazard. More information on the assessment and mitigation of slope instability and rockfalls may be provided in a future NZGS/MBIE module.

The information in Appendix C4D is provided as interim guidance on assessing slope instability hazard.

Note:
GNS Science’s wealth of reporting on the Port Hills soil and rock slope stability in the Canterbury earthquake sequence is also informative for landslide and rockfall hazard assessment in other parts of New Zealand. The reports can be accessed on the Christchurch City Council website at www.ccc.govt.nz (search for “Port Hills geotechnical”).

C4.7.7 Tsunami
Refer to NZGS/MBIE module 1 for general comments on tsunami. NZGS has no present plans to include assessment or mitigation of tsunami hazard within the current module.
series. The effects of Tsunami are not intended to be considered in the assessment of the seismic rating for the building.

**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 a peer review where applicable (refer to Section C4.8.4).

**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 C4.1.1 for characteristics of the three project categories and Section C4.2.4 for the expected differences in reporting scope).

**C4.8.3 Report content**

All geotechnical reports should document the following:

- the desk study and site inspection records
- the ground model and any site investigations compiled/completed as part of the assessment
- results and conclusions of any interpretation undertaken
- any further recommended investigation/analysis/monitoring, and
- risks and uncertainties.

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

- potential geohazards identified and the basis of their non-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 provided to the structural engineer for use in analysis and assessment including bearing capacities and, where required, ground linear 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 performance of the building. For geohazards that potentially influence the ultimate performance of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard)
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- 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 ultimate capacity of the structure the report should provide, as a minimum, probable resistance/deformation, and/or resistance/%ULS shaking relationships (to suit the geohazard) and 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 peer review requirements are summarised in Table C4.5. The peer reviewer’s comments and the assessor’s responses should be summarised separately and appended to the geotechnical report.

Table C4.5: Suggested peer review requirements

<table>
<thead>
<tr>
<th>Case</th>
<th>Peer review recommended</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structurally dominated project (in the absence of any other considerations described below)</td>
<td>X</td>
</tr>
<tr>
<td>Interactive project (in the absence of any other considerations below)</td>
<td>X</td>
</tr>
<tr>
<td>Interactive project IL3* and above</td>
<td>✓</td>
</tr>
<tr>
<td>Geotechnically dominated project IL3 and above</td>
<td>✓</td>
</tr>
<tr>
<td>Site response analysis</td>
<td>✓</td>
</tr>
<tr>
<td>Studies that provide geotechnical input to multiple structures simultaneously</td>
<td>✓</td>
</tr>
<tr>
<td>Studies that define geohazard risks for multiple sites, e.g. regional liquefaction, tsunami, rockfall studies</td>
<td>✓</td>
</tr>
<tr>
<td>Studies where the outcome of the structural assessment is sensitive to one or more of the following:</td>
<td>✓</td>
</tr>
</tbody>
</table>

- soil-structure interaction
- geophysical investigations
- numerical modelling
- time-history analyses

**Note:**

* IL = Building importance level as defined in NZS 1170.0:2002
References and Bibliography


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Taciroglu, E. (2013). Development of Improved Guidelines for Seismic Analysis and Design of Earth Retaining Structures July 2013, California Department of Transportation (CDOT) 2013/02


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Appendix C4A: Modelling of SSI Effects

C4A.1 General

Soil-structure interaction (SSI) can be very complicated and difficult to model with great reliability and accuracy. This appendix outlines some general principles, followed by possible approaches using a flexible base model to account for linear and nonlinear foundation deformations.

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 2012).

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 old fear of possible 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.

- When the stiffness of foundation soils is included in a global analysis building model it is important that the probable/expected design values are used. In lieu of an explicit evaluation of the uncertainties in foundation characteristics it is recommended that the upper bound stiffness is taken as two times the expected design value and the lower bound stiffness as one half of the expected design value (NIST, 2011).

- The assessment should seek to understand the probable behaviour of the soil, foundation and structure. As such, typical nominal geotechnical parameters which are conservative for design can be non-conservative for assessment and lead to an incorrect mechanism being identified.

- Sensitivity to variations in assumptions should always be checked. Care is required in assumption of variability. “Nominal” values provided by a geotechnical engineer are often design values which are typically close to lower bound. By undertaking a typical sensitivity analysis (e.g. 50%/200%) the analysis may be biased towards a soft response which may underestimate the loads on the structure and foundation.

- 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.

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. The assessing engineer should be cautioned on adopting the various “benefits” of SSI if considering possible mechanisms that may reduce seismic demand on the structure.

**C4A.3 SSI Modelling Approaches**

**C4A.3.1 Simplified hand analysis to determine influence of ground**

The assessor 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 assessor should use conservative geotechnical parameters and undertake relevant sensitivity analyses.

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 bound of 50% to 200% times the spring flexibility should be carried out, unless specialist geotechnical advice on the soil flexibility is available.

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, conservative 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 or $D$ spacing, where $D =$ diameter of piles or depth of ground beams. In some analysis packages, line or area springs can be applied.
- 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 (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 Appendix 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, 2012 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, $G_{sec} = \frac{\tau_c}{\gamma_c}$ where $\tau_c$ and $\gamma_c$ are the expected amplitudes of shear stress and shear strain respectively
- an equivalent viscous damping ratio, $\xi_{soil}$ that is directly proportional to the hysteretic energy dissipated.

\[
\xi_{soil} = \frac{1}{2\pi} \frac{A_{loop}}{G_{sec} \gamma_c^2}
\]  

...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. $\xi_{soil} = 5\%$ is recommended to be used with the structural performance factor, $S_p$ as per the building structural ductility capability.

C4A.3.5 Nonlinear pseudostatic analysis with explicit nonlinear soil spring

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), 2010 and Cubrinovski and Bradley, 2009).

Two approaches for shallow and deep foundations are illustrated in the following figures.
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, $\xi_{\text{soil}}$ is limited to 10% and $\xi_{\text{soil}} = 5\%$ 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 $\xi_{\text{soil}}$) 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 the direct approach, in which the entire SSI system is analysed in a single model/step. SSI using the direct model 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 direct FE methods, including the definition of critical input parameters (e.g. a constitutive model for various soil types), the available geotechnical information of the underlying soil, the definition of boundary conditions and the complexity of such a complex nonlinear model.

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)](image-url)
Appendix C4B: The Ground Model

The ground model is a composite of: the terrain at and around the structure, the geological model, the regional seismic hazard model, the geotechnical model (soil/rock properties), and the groundwater regime. Every site has these components. Where appropriate, the ground model is augmented with information on relevant geohazards and foundation details to provide an at-a-glance summary drawing/sketch of the soil-foundation domain (refer to Figure C4B.1 for a graphic representation of the ground model's possible components).

The resulting ground model provides a valuable tool for discussions between the geotechnical and structural engineer, and can help illuminate potential geotechnical influences on the structural behaviour.

The level of detail presented in the ground model and the degree to which it covers all of the aspects shown in Figure C4B.1 should reflect the use to which it is to be put. There may be little benefit in having a very detailed and complete ground model if it is determined that the geotechnical issues are not material to the seismic rating of the building.

Note:
NZGS/MBIE module 4: Guide to Site Investigation (which was still to be published at the time these guidelines were issued) will provide more detail on the development of an appropriate ground model.
Table C4B.1 presents common sources of information that may be useful. Local knowledge and experience will be invaluable.

**Table C4B.1: Ground model information sources**

<table>
<thead>
<tr>
<th>Component</th>
<th>Relevant information</th>
<th>Initial sources (where available)</th>
<th>Additional/detailed sources</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Terrain</strong></td>
<td>• Landform at and around the site</td>
<td>• Site inspection</td>
<td>• LiDAR</td>
</tr>
<tr>
<td></td>
<td>• Topography</td>
<td>• Contour plans</td>
<td>• Geomorphic mapping</td>
</tr>
<tr>
<td></td>
<td>• Geomorphology</td>
<td>• Aerial photographs</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Google Earth</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Note: Where visible, the geomorphology can be invaluable to understand the geological processes that led to its current form and may influence its behaviour in the future.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Geology</strong></td>
<td>• Structure of the soil and rock deposits underlying the site and the land around the site</td>
<td>• Site inspection</td>
<td>• Ground investigation factual records</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geological maps, memoirs and reports</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• GNS Science’s online geological maps</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geotechnical reports</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• National Geotechnical Database (in development)</td>
<td></td>
</tr>
<tr>
<td><strong>Seismic hazard</strong></td>
<td><strong>Site period</strong></td>
<td>• NZGS/MBIE module 1</td>
<td>• Site-specific seismic hazard assessment</td>
</tr>
<tr>
<td></td>
<td>• Intensity and duration of shaking</td>
<td>• NZS 1170.5:2004</td>
<td>• Topographic features that may amplify seismic energy</td>
</tr>
<tr>
<td></td>
<td>• Damping potential</td>
<td>• GNS Science’s online active faults database</td>
<td>• Site response studies</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• NZTA Bridge Manual (2013)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Existing site-specific seismic hazard assessments</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geological maps, memoirs and reports</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Council resources and historic records</td>
<td></td>
</tr>
<tr>
<td><strong>Soil/rock properties</strong></td>
<td><strong>Geotechnical properties, e.g. strength, density, stiffness</strong></td>
<td>• Geotechnical maps, memoirs and reports</td>
<td>• Ground investigation factual records and laboratory records</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geotechnical reports</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Presumptive values</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geotechnical databases</td>
<td></td>
</tr>
<tr>
<td><strong>Groundwater regime</strong></td>
<td><strong>Depth to groundwater level; seasonal fluctuation</strong></td>
<td>• Site inspection (refer to page points, standing water, lake/river levels)</td>
<td>• Ground investigation factual records</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geological maps, memoirs and reports</td>
<td>• Monitoring records</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geotechnical reports at and around the site</td>
<td>• Water well bore logs and water level records</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Council records</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Water abstraction resource consents</td>
<td></td>
</tr>
<tr>
<td><strong>Geohazards</strong></td>
<td><strong>The site’s exposure to geohazards; their relevance to life safety</strong></td>
<td>• District Plan – commentary on natural hazards</td>
<td>• Site-specific evaluation</td>
</tr>
</tbody>
</table>
Where relevant information is not available via the initial review (desk study and site inspection), it is expected that work would be commissioned to fill in any knowledge gaps. A common knowledge gap is the detailed site characterisation, which warrants specific site investigation (e.g. boreholes, CPTs, laboratory testing, etc). This investigation should be scoped and managed by the geotechnical engineer.

Further reading


Appendix C4C: Assessment of Retaining Walls
(Interim Guidance)

C4C.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:
As NZGS/MBIE module 6: Guidance for retaining wall design was at a preliminary stage of development when these guidelines were drafted, a relatively high level of detail is provided here compared to other parts of this section.

C4C.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 (MBIE, 2014).

Wood (2014) reviewed wall damage descriptions in the Stronger Christchurch Infrastructure Rebuild Team (SCIRT) database and generated the following summary of general observations for walls over 1.5 m in height:

- Engineered retaining walls performed well in these earthquakes, even though these were unlikely to have been designed to the levels of ground shaking experienced (many may not have been designed for any earthquake loading).
- A significant number of retaining walls at residential properties suffered damage. Many of these were poorly designed and/or constructed (e.g. lack of reinforcement or grouting, or low quality backfilling).
- Walls that retained fill often did not perform as well as those that retained undisturbed loess soil.
- Significant settlement of retained fill was commonly observed at flexible walls as well as more rigid walls.
- Many non-engineered rock facings (which are generally quite old structures) collapsed exposing stable, near vertical faces of undisturbed loess. This indicates that undisturbed dry loess typically has high apparent cohesion under short term loading conditions.
- Several retaining wall failures appeared to be initiated by slope instability either above or below the wall.

Note:
Other useful references include Anderson et al. (2015) and Kendall-Smith (2015).

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.

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 $\phi$ (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 states that: “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”.

C4C.3 Identification of Retaining Walls requiring Assessment

C4C.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 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? (E.g. 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 (e.g. 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 C4.4, 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.

### C4C.3.2 Loss of emergency access/egress to the building

Table C4C.1 gives some examples where poor performance in a wall may impact on emergency access/egress and hence on the building’s seismic performance.

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>As designed</th>
<th>Potentially unacceptable performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Instability in a retaining wall supporting building egress</td>
<td><img src="image1.png" alt="Diagram" /></td>
<td><img src="image2.png" alt="Diagram" /></td>
</tr>
<tr>
<td>Instability in a retaining wall supporting ground that provides building egress</td>
<td><img src="image3.png" alt="Diagram" /></td>
<td><img src="image4.png" alt="Diagram" /></td>
</tr>
<tr>
<td>Instability in a wall supporting ground above a building egress</td>
<td><img src="image5.png" alt="Diagram" /></td>
<td><img src="image6.png" alt="Diagram" /></td>
</tr>
</tbody>
</table>
C4C.3.3 Loss of support to foundation soil

Table C4C.2 gives an example where poor performance in a retaining wall providing support to the building foundations may impact on the building’s seismic rating.

Table C4C.2: Loss of support to foundation soil

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>As designed</th>
<th>Potentially unacceptable performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Instability in a retaining wall below building foundations removing vertical support</td>
<td><img src="image1.png" alt="Diagram" /></td>
<td><img src="image2.png" alt="Diagram" /></td>
</tr>
</tbody>
</table>

C4C.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 seismic rating are set out in Table C4C.3.

Table C4C.3: Lateral loading or loss of lateral support to foundation soil

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>As designed</th>
<th>Potentially unacceptable performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Instability in a retaining wall impacting on building. Does not influence the seismic rating of the building</td>
<td><img src="image3.png" alt="Diagram" /></td>
<td><img src="image4.png" alt="Diagram" /></td>
</tr>
<tr>
<td>Instability of a retaining wall generating lateral loading on foundations supported at a deeper level</td>
<td><img src="image5.png" alt="Diagram" /></td>
<td><img src="image6.png" alt="Diagram" /></td>
</tr>
<tr>
<td>Instability in a basement retaining wall</td>
<td><img src="image7.png" alt="Diagram" /></td>
<td><img src="image8.png" alt="Diagram" /></td>
</tr>
</tbody>
</table>
C4C.4  Modes of Failure

Checks on retaining wall performance should consider potential failure modes as set out in Table C4C.4.

Table C4C.4: Potential failure modes

<table>
<thead>
<tr>
<th>Mode of failure</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing</td>
<td>The assessment of bearing capacity is generally a key consideration for gravity or cantilever walls, but may also extend to tied back embedded walls in assessing the tie back or where the tie is steeply inclined and generates a significant vertical load. Bearing capacity can be checked using conventional formula: refer to Bowles (1988), B1/VM4 (2011) or NZGS/MBIE module 2 (under preparation). Particular care should be taken in the consideration of inclined loading of foundations arising from horizontal static and seismic soil actions.</td>
</tr>
<tr>
<td>Sliding</td>
<td>The sliding/lateral resistance of a retaining wall is primarily a consideration for non-embedded walls, e.g. gravity/counterfort. Sliding is usually checked considering sliding resistance along the soil/wall interface or through intact soil. Friction between the wall/soil should be carefully evaluated. In some cases a polyethylene DPC membrane may result in low friction; in others concrete poured directly on in-situ ground can yield a sliding resistance approaching that of in-situ ground.</td>
</tr>
<tr>
<td>Overturning</td>
<td>This is applicable to most retaining wall types. Overturning may be checked using factored soil strengths following the approach of CIRIA C580 or Eurocode 8 (2003) or using unfactored soil properties and factored demands as per MBIE - retaining wall guidance (2014) and AS 4678 (2002).</td>
</tr>
<tr>
<td>Global stability</td>
<td>Global stability should be checked for all walls unless embedded in rock. For simple analyses the use of stability charts such as those published by Hoek and Bray (1974) may be appropriate, although for complex geometry or more detailed analyses the use of limit equilibrium analyses software may be more appropriate.</td>
</tr>
<tr>
<td>Structural</td>
<td>Checks should be undertaken of the performance of structural elements of retaining walls. These checks should typically be undertaken against loads derived from unfactored acceleration values (or yield values determined based on upper bound soil properties) but may incorporate structural ductility.</td>
</tr>
</tbody>
</table>
C4C.5 Seismic Loads

C4C.5.1 Seismic loads and resistance

Seismic loads for assessment of retaining walls should be derived assuming an importance level/design life as per the structure they potentially influence. Limited amplification is expected in most retaining walls: consequently, peak ground acceleration (PGA) is used for assessment. Note that this does not necessarily apply to the assumed response of structures which act through the wall. The potential for coincidence of structural loading and retained soil loading should be considered (refer to Table C4C.5).

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.

The assessment of these effects is best undertaken by well-calibrated dynamic numerical analysis using multiple appropriately selected and scaled time-history records. However, such analyses may not be justified and a simple sensitivity analysis should be undertaken.

Table C4C.5: Factors to be considered for loading on retaining walls

<table>
<thead>
<tr>
<th>Details</th>
<th>Structure/soil loading likely coincident</th>
<th>Use conservative assumption or undertake specific analysis</th>
<th>Structure/soil loading unlikely to be coincident</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basement retaining wall</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall retaining a building platform. Structure &gt;3(H) behind.</td>
<td></td>
<td></td>
<td>x</td>
<td>Consider slope of land between wall and structure and the presence of sensitive/liquefiable ground</td>
</tr>
<tr>
<td>Wall retaining building platform. Structure between 1 and 3(H) behind wall.</td>
<td></td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall retaining building platform. Structure &lt;1(H) behind wall.</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Where liquefaction derived pressures or lateral spread flow loads are already accounted for in design</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Inertial load from wall elements excluding MSE</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: \(H\) = wall height
Seismic loads act on retaining walls in a number of ways. Table C4C.6 provides some guidance on seismic loads and soil restraint.

**Table C4C.6: Seismic loads**

<table>
<thead>
<tr>
<th>Load case</th>
<th>Diagram of load application</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Inertial loads</strong></td>
<td></td>
</tr>
<tr>
<td>These are seismic loads developed</td>
<td></td>
</tr>
<tr>
<td>from acceleration of the wall</td>
<td></td>
</tr>
<tr>
<td>structure itself. They may vary</td>
<td></td>
</tr>
<tr>
<td>from insignificant (a soil nail or</td>
<td></td>
</tr>
<tr>
<td>sheet pile wall) to significant (a</td>
<td></td>
</tr>
<tr>
<td>gravity retaining wall).</td>
<td></td>
</tr>
<tr>
<td><strong>Seismic aActive earth thrust</strong></td>
<td></td>
</tr>
<tr>
<td>For the purpose of seismic</td>
<td></td>
</tr>
<tr>
<td>assessments earth pressures are</td>
<td></td>
</tr>
<tr>
<td>typically considered to act</td>
<td></td>
</tr>
<tr>
<td>pseudostatically. Retaining walls</td>
<td></td>
</tr>
<tr>
<td>are normally designed to resist</td>
<td></td>
</tr>
<tr>
<td>earthquake loading by considering</td>
<td></td>
</tr>
<tr>
<td>a pseudostatic horizontal</td>
<td></td>
</tr>
<tr>
<td>acceleration applied to the wall</td>
<td></td>
</tr>
<tr>
<td>and the retained soil. Refer to</td>
<td></td>
</tr>
<tr>
<td>NZGS/MBIE module 1 for guidance</td>
<td></td>
</tr>
<tr>
<td>on the determination of design</td>
<td></td>
</tr>
<tr>
<td>acceleration.</td>
<td></td>
</tr>
</tbody>
</table>

**Seismic passive earth thrust resistance**

**C4C.5.2 Derivation of retaining wall seismic loads**

**General**

A number of different methods can be adopted in the assessment of seismic loads acting on a retaining wall. Four methods are outlined below. The most suitable method will generally depend on the level of refinement in design and the level of displacement that can be accommodated by the wall.

Retaining walls may be checked against earthquake loading by considering a pseudostatic horizontal acceleration. Flexible walls are treated differently to stiff walls and tied-back or propped walls. Flexible walls assume the development of active earth pressures behind the wall, while stiff walls consider higher pressures derived from the inertia of the retained soil mass.
Guidance on the presumed flexibility of common retaining walls is presented in Table C4C.7. Where critical, a more rigorous assessment should be undertaken in consultation with a structural engineer.

### Table C4C.7: Presumed flexibility of common wall types

<table>
<thead>
<tr>
<th>Flexible (plastic) wall</th>
<th>Intermediate/stiff walls</th>
<th>Rigid (elastic) or stiff walls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity walls – gabion, crib, mass concrete, modular concrete block, 'L’ shaped cantilever, many embedded cantilever pole/pile walls founded in soil</td>
<td>Soil nail walls</td>
<td>Basement walls</td>
</tr>
<tr>
<td></td>
<td>Walls anchored back to deadman</td>
<td>Some propped walls</td>
</tr>
<tr>
<td></td>
<td>Some embedded cantilever pole/pile walls founded in strong ground, e.g. cemented soil/rock. counterfort walls</td>
<td>Some anchored walls</td>
</tr>
</tbody>
</table>

### Method A – Force-based design, rigid or stiff wall seismic response

The seismic earth pressure acting on walls that deflect less than 0.1% of their height and are restrained against outward displacement (e.g. buttressed concrete basement walls) will be greater than given by the M-O equation (Wood et al, 1990).

**Rigid**

The earthquake component of the pressure force on a rigid or elastic wall that deflect less than 0.1% of their height can be taken as:

\[
\Delta P_E = 1.0 k_h \gamma H \quad \text{(Equation 3-1 from RRU 84) \ldots \text{C4C.1}}
\]

where:

- \( k_h \) = earthquake acceleration design coefficient (calculated using \( W_d = 1 \))
- \( H \) = wall height
- \( \gamma \) = unit weight of the backfill

The seismic thrust is assumed to act at 0.58\( H \). For slopes up to 20 degrees the point of action will rise to around 0.7\( H \). Note that yielding of soil support and/or structural elements means that in practice very few walls (in the context of a seismic assessment which allows for significant structural yielding) are truly rigid.

**Stiff**

Where walls can deflect between 0.1% to 0.2% of their height they are classified as “stiff” and earth pressure can be taken as:

\[
P_E = 0.75 k_h \gamma H^2 \quad \text{(Equation 3-2 from RRU 84) \ldots \text{C4C.2}}
\]

The seismic thrust is assumed to act at 0.5\( H \). At deflections beyond 0.2% the earthquake pressure force component on a stiff wall reduces in an approximately linear manner to the M-O earthquake force component at a wall deflection of about 0.4% of the wall height as shown in MBIE (2014).
Method B – Force-based design, flexible wall seismic response

For flexible walls the pseudostatic earth pressure may be calculated using $K_{AE}$ from the Mononobe-Okabe (1926, 29) (M-O) equations (refer to NCHRP 611 (2008) for a detailed description of the M-O method plus equations).

Charts giving values of $KE$ for various levels of seismic acceleration, soil parameters, wall slope, wall interface friction angle and back-slope angle are presented in CDOT 2013/2 and Taciroglu (2013): Appendix A (active) and Appendix B (passive).

For walls where no significant permanent deformation is acceptable, the full design acceleration should be used to calculate $K_{AE}$. The inertial effect resulting from the mass of the wall under horizontal seismic acceleration, including the mass of any soil located above the heel, should be added to the calculated lateral earth pressure in all cases. The calculation of lateral earth pressure should include the effect of any surcharge applied to the retained ground. The seismic active earth pressure may be assumed to act at a height $H/3$ above the base of the wall.

Refer also to MBIE (2014).

Method C – Displacement-based design

Displacement-based design is commonly adopted in the design of new retaining walls and the assessment of existing retaining walls. A displacement-based design approach can be taken in a number of ways:

For simple analyses such as initial assessments of retaining walls it is common practice [reference] to adopt a reduction in the design acceleration where some lateral displacement of the structure and retained ground is acceptable. A design seismic acceleration coefficient of between 0.33 and 0.5 of the peak ground acceleration is commonly applied for retaining structure design using pseudostatic procedures (Kramer, 1997; Wood, 2008).

Permanent displacements in the order of 100 mm might be anticipated if the wall is tested to the full design acceleration. Consequently, this approach is only recommended for walls anticipated to have a relatively ductile response and for soils that are not subject to significant strength loss on shearing. As shearing within the retained/founding soil is implicit in this design method soil parameters consistent with this displacement should be selected.

For more detailed analyses, methods with a more explicit treatment of wall displacement and changes in soil strength are recommended. These methods use the result of Newmark sliding block regression analyses published by researchers such as Bray and Travasaro (2007) and Jibson (2007). Note that a similar approach to uncertainty should be adopted for displacement as with soil properties, i.e. two values derived and provided to the assessor (50th percentile and 15th percentile probable values (analogous to design values).

Method D – 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
A minimum of seven records is recommended to address the potential for earthquake shaking variability.

**C4C.5.3 Modelling seismic loads**

**Irregular retained ground**

Ground retained by retaining walls is rarely as level as that found in textbook examples and simple methods developed for finding the equivalent height of backfill are potentially unconservative when considering seismic loads (they do not consider the enlargement of the critical active wedge due to seismic inertia). In such cases a trial wedge can be employed; however this iterative procedure can be time consuming (CDOT 2013/02).

An alternative is to use limit equilibrium slope stability software to determine the required stabilising force. This can be entered into the retaining wall analysis as a shear force appropriately distributed on the back face of the wall above the slip plane. For more detail refer to NCHRP 611: Appendix F.

**Surcharge loads**

It is often necessary to simplify the seismic component of berms, surcharges or sloping backfills in analyses. Typically seismic loads are modelled either as a localised horizontal load acting on the back of the wall or as a surcharge applied to the retained soil behind the wall (surcharge = equivalent horizontal seismic load/active earth pressure coefficient).

Careful consideration is necessary to ensure that the approximation applied is distributing the load realistically on the back of the retaining wall. Sensitivity analyses are recommended and where walls are stacked, a check may be necessary using numerical methods.

Further guidance on the treatment of surcharge loads is provided in CDOT 2013/02.

**C4C.5.4 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. 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, 2003) provides methods of estimating of the magnitude of retained ground settlement and potential consequences for a range of structural types.
C4C.5.5 Soil properties

Assessment soil parameters should be selected for the assessment of retaining walls adopting the principles set out in Section C4.4.

A brief discussion on the selection of key parameters is provided below.

Wall friction

Generally, friction between the wall and active or passive wedges should be adopted – with the exception of the case where the wall may move with (and hence not develop restraint against) the soil wedge. For example, it is generally not appropriate to adopt wall friction (for the calculation of active forces) where the wall is expected to settle significantly or be subject to large vertical deflections (bearing capacity failure). Wall-soil friction on the active wedge side of a wall is generally limited to $\frac{2}{3} \phi$. Wall-soil friction on the passive side is generally limited to $\frac{1}{2} \phi$ when using the M-O equation, as above this level passive pressure can become un-conservative.

Retained soil cohesion

The presence of even modest levels of cohesion within retained soil can significantly reduce the earth pressure (static and seismic) acting on a retaining wall, particularly for walls of modest height. However, cohesion should be used with caution as the presence of fissures, tension cracks or backfilled trenches within retained soil is common. Allowance should also be made for relaxation following excavation and loss of soil suction during periods of high groundwater level. Seismic earth pressure design charts assuming a range of cohesion values are available (CDOT 2013/02). The uncertainty in retained soil cohesion should be reflected in derived loads/deflections for consideration in structural performance. Where a probable (50th percentile) assessment is undertaken using cohesion, a conservative (15th percentile) assessment should also be provided which will generally ignore the effects of cohesion.

Foundation soil cohesion

The assumption of undrained conditions during earthquake shaking for retaining wall foundations is a common approach, particularly in the design (and checking) of embedded pole walls. For embedded cantilever walls, assuming undrained conditions ($S_u$) as opposed to drained conditions ($\phi$) can produce a considerably more favourable result. Care should be taken. The engineer must consider whether the relevant soil can be depended on to behave in an undrained manner and, if in doubt, check assuming drained/frictional ($\phi$) conditions. Care should also be taken to consider the potential effect of infilled trenches and the like in front of the retaining wall. In a similar manner to retained soil, the consequences of ground potentially responding in a drained manner (e.g. from a backfilled trench) could be considered alongside a probable performance.

C4C.5.6 Groundwater

The effect of water on the seismic earth pressure on a retaining wall needs consideration of three aspects: hydrostatic pressure, hydrodynamic pressure and excess pore pressure generated by dynamic loading (Taylor, 2009). These are discussed below.
Hydrostatic pressure

Where the water table is above the base of the backfill, the active earthquake thrust should be divided into static and dynamic components for computing the lateral forces. Buoyant soil weight is used for computing the static component below the water table. The hydrostatic force is added and saturated soil weight is used for computing the dynamic component. This process can significantly increase the calculation complexity, particularly in conjunction with a sloping or irregular retained soil geometry or surcharges. In such instances it is recommended to make use of limit equilibrium slope stability software to determine the active (and, if necessary, passive) earth pressure.

Hydrodynamic pressure

In high permeability soils (gravels/coarse sand) or seawalls, hydrodynamic pressures may be estimated utilising the Westergaard free water solution (1931). In low permeability soils (i.e. clay/silt) hydrodynamic pressures may be disregarded. Matsuzawa et al. (1985) provides a solution for intermediate soils.

Seismically generated excess pore pressure

The potential for excess pore pressure to be generated dynamically from soil contraction should be assessed in accordance with the recommendations in NZGS/MBIE module 3 and accounted for in analyses.

C4C.5.7 Strength reduction factors

Refer to DBH 2011 (B1/VM4) and AS 2159:2009 for determination of appropriate strength reduction factors.
Appendix C4D: Slope Instability Hazard (interim guidance)

C4D.1 Introduction

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.

C4D.1.1 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.

C4D.1.2 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.

C4D.2 Guiding Principles

While slope instability 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.

C4D.2.1 Influence of ground conditions

Examples of where seismically induced slope instability is potentially 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 35°), 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.

C4D.3 Assessment Process

Stage 1 - Initial assessment of stability
A great deal of information on slope stability can usually be obtained via desk study and/or site inspection by a suitably skilled person. Where the stability of the slope may be influenced by the geology 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 (Clause 8.2.2.4) is a useful guide for screening purposes.

Assessors 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 modules 2 and 4.

Stage 2 - Site inspection
Relevant geohazard information 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 from Google Earth). 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).
• 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.

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 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 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 Bardour and Krahn (2004) for insights and guidance on numerical modelling.

C4D.4 Defining Seismic Accelerations for Slope Stability Analysis

Refer to NZGS/MBIE module 1.

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 NZS 1170.5:2004 or NZGS/MBIE module 1 (2016). MBIE (2014) and Eurocode EN 1998-5:2004 provide information on topographic amplification factors.
Appendix C4E: General Commentary on Seismic Performance of Foundations

C4E.1 Observations from the Canterbury Earthquake Sequence

The following commentary outlines the generic seismic performance for a variety of foundation types and is based on New Zealand experience and observations (particularly following the Canterbury earthquake sequence of 2010-11).

C4E.1.1 Shallow foundations

Foundation elements are considered to be shallow when the depth to breadth ratio is less than 5 (i.e. $D/B < 5$), generally including the following:

- **Isolated pads**
  Isolated pads are seldom appropriate for building foundations subject to seismic actions, especially in Christchurch where the ground conditions are known to be variable and mostly unsuitable. These could well have suffered from differential settlement and differential lateral movement, especially in areas of liquefaction.

- **Strip/beam footings**
  Continuity of foundation elements is important to ensure integrity of a structure subject to differential ground movements. Where differential movements are excessive, the footings should be checked for structural damage.

- **Pad and tie beam foundations**
  Similar to above

- **Mat foundations**
  Mat foundations are continuous structural slabs spanning between columns and walls, etc. Their resistance to differential ground movements will vary according to their strength and stiffness. The level of damage will also depend on the extent of differential movements, both vertical and lateral.

- **Raft foundations**
  Raft foundations are similar to mat foundations but have sufficient strength and stiffness to behave essentially as a rigid body when accommodating differential ground movements. True rafts are rare as the required levels of strength and stiffness are prohibitive.

A summary of common shallow foundation issues, fixes and potential limitations is presented in Table C4E.1.
Table C4E.1: Summary of common shallow foundation issues, fixes and potential limitations

<table>
<thead>
<tr>
<th>Problem</th>
<th>Fix</th>
<th>Potential limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Excessive settlement</td>
<td>a. If settlement tolerable but structure at risk if similar settlement occurred in future earthquake, and bearing capacity of ground suitable for shallow foundations, then widen foundations.</td>
<td>Difficult with boundary walls – may require offset foundation and crossbeams to take out eccentricity</td>
</tr>
<tr>
<td></td>
<td>b. Underpin with piles. This may also allow re-levelling.</td>
<td>May not have sufficient access for piling rig, both to perimeter and internal foundations. Consider the type of pile carefully and check compatibility with existing foundations for both vertical and lateral actions.</td>
</tr>
<tr>
<td></td>
<td>c. Compaction grouting can relevel foundations and stiffen soils to reduce settlement in a future earthquake.</td>
<td>Not suitable in all soils, may require drilling through floor</td>
</tr>
<tr>
<td>2. Lateral spread</td>
<td>a. Consider external damming or buttressing of soils in order to restrain future spread.</td>
<td>Not likely to be practical in many cases – only applicable when there is sufficient access and work can be achieved on site</td>
</tr>
<tr>
<td></td>
<td>b. External sheet pile wall or piles, with ground anchors to restrain lateral load.</td>
<td>Requires access for plant and suitable ground conditions for toe of sheet piling and anchorage</td>
</tr>
<tr>
<td></td>
<td>c. Underpinning may be installed to perimeter foundations. May also need addition of foundation ties across the building to counteract future spread.</td>
<td>Relatively simple to install provided clear access available. May still be vulnerable to future damage if lateral spread not addressed externally.</td>
</tr>
<tr>
<td>3. Basement with uplift</td>
<td>Grout under the floor to fill any voids.</td>
<td>May compromise any tanking. Uncertainty as to how effective grouting may have been.</td>
</tr>
</tbody>
</table>

A key generic issue relevant to all types of shallow foundations is to decide whether or not they remain appropriate for the structure or underpinning with deep foundations is required. This decision should not be based solely on the performance of the foundation to date, but on the risks of damaging settlement from future events, based on proper analysis of the ground conditions. While differential settlements as measured post the February 2011 Canterbury earthquake may be within tolerable limits for the structure, another earthquake could produce similar or greater differential movement cumulative to the first, which could then lead to severe structural damage or failure.

Settled footings may be the result of liquefaction or soil response at depth, or may simply have been overloaded by the earthquake induced axial loads. DBH 2011 B1/VM4 permits use of a generic geotechnical strength reduction factor of $\phi_g = 0.8 – 0.9$ for load combinations including earthquake “overstrength”, which is much higher than factors typically used for other load combinations. This results in a high risk that the ultimate capacity of the footing will be exceeded at the design load. In reality, the bearing capacity of shallow foundations is reduced by inertial effects during shaking as well as from
increased pore water stresses, which in combination with high seismic loading from the structure can induce large deformation.

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 parts of a shallow foundation only. 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.

Where gapping occurred adjacent to footings, the gaps should be filled with sand-bentonite grout to restore the full passive resistance of the soil.

Where rocking of foundations has occurred (or suspected to have occurred) gaps may exist underneath foundation elements or under the edges of elements. Locate and fill such gaps.

Note that foundation-related problems for shallow footings may have a “binary” aspect: i.e. if there has been excessive movement there may be no effective repair solution, even if the super structure is relatively undamaged.

C4E.1.2 Deep foundations

Foundation elements are considered to be deep when the depth to breadth ratio is greater than 5 (i.e. $D/B > 5$). Generally, in Christchurch, the following deep foundation types are in use:

**Driven concrete piles**

Typically these are 6 m to 15 m long, with some as short as 2-3 m and rare buildings with piles in excess of 20 m. These are driven to found onto a dense gravel stratum. Few buildings in Christchurch have been founded on driven piles larger than 150 mm square sections in the last 15 years due to resource consent issues to do with noise and vibration during driving. They are typically designed as end bearing, although a contribution from side friction may be included. Both compression and uplift capacity from side resistance may be lost with liquefaction. Lateral capacity may also be affected if adequate embedment has not been achieved into the dense soils.

**Driven steel piles**

These types of piles are not widely used in Christchurch but may be driven to found onto the more dense gravel strata at depth. Uplift capacity from friction may be lost with liquefaction unless adequate embedment has been achieved into a dense (non-liquefiable) soil.
Driven timber piles

Typically, these tend to be shallower than other pile types and may be vulnerable to both bearing and lateral capacity strength loss within or underneath the bearing stratum. These are not common for commercial buildings.

Bored cast-in-place piles

These are usually 6-15 m deep and 0.6 to 1.2 m diameter; occasionally up to 1.5 m diameter and up to 20 m deep. Typically, these are excavated in water filled steel casing which is withdrawn during concreting. Although often designed as end bearing with some contribution from side resistance, in reality, for many of them, the gravity loads will have been carried since construction by the side resistance mechanism. Loss of side resistance from porewater pressure effects during shaking may lead to settlement from gravity loads (refer to discussion below).

Uplift in bored piles in Christchurch is resisted by side resistance. There is no knowledge of belling or under-reaming of any piles in Christchurch. The cohesionless sands and gravels below the water table do not allow under cutting or even any excavation outside a fully cased hole without bentonite slurry support.

Bulb (Franki) piles

These are common on many buildings between about 1970 and the late 1980s. Steel casings were bottom driven to depth, a cement-gravel plug was driven out to form the bulb, and the casing withdrawn as the shaft was concreted. These are typically 450-600 mm diameter shafts on nominal 1 m diameter bulbs and are less than 10-12 m depth. The bulbs are below the reinforcing cage, so there is no reliable uplift capacity except on the shaft – unless there is a second bulb driven out through the reinforcing cage above the compression bulb. Piles may have limited fixity at the base affecting lateral capacity.

Screw piles

Typically, these are 10 m to 20 m long and are screwed into a dense stratum. Capacity comes from end bearing onto the screw flanges. Uplift capacity comes from “upside down” bearing which may fail if the overlying materials liquefy. There is minimal side resistance along the stem.

Continuous Flight Auger (CFA) piles

There are only included here for completeness as this is a relatively new technology in Canterbury and there are few currently in use. Continuous Flight Auger piles are essentially bored piles installed without casing, so most of the comments above on bored piles apply. The maximum length and diameter is limited by available equipment but is in the order of 600 mm diameter and 15 m length. Using specially adapted equipment, an auger is screwed into the ground and then withdrawn as concrete is pumped down the centre of the flight under pressure, displacing the soil. Once withdrawn, a reinforcing cage is placed into the concrete. This technique is relatively quick but is technically challenging: it requires good quality assurance procedures as well as experienced operators.
Common issues for deep foundations that need to be considered are as follows (refer also to Table C4E.2):

- **Loss of side resistance (skin friction)** in piles 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.

- **Settled piles** may simply have been overloaded by the earthquake induced axial loads. The Building Code B1/VM4 document permits use of a generic geotechnical strength reduction factor of $\phi_g = 0.8 - 0.9$ for load combinations including earthquake “overstrength” loads, which is much higher than factors typically used for other load combinations, resulting in a high risk that the pile capacity will be exceeded at the design load. Strength reduction factors for pile design, including earthquake load cases, should be selected based on a proper risk assessment procedure such as that given in AS 2159-2009.

- **Pile settlement** may also be from liquefaction of sand layers below the founding layer. 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. Guidance for selecting the appropriate level of investigation is given in Table C4E.2.
Table C4E.2: Summary of common deep foundation issues, fixes and potential limitations

<table>
<thead>
<tr>
<th>Problem</th>
<th>Fix</th>
<th>Potential limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Excessive pile settlement</td>
<td>a. May be possible to cut piles and re-level building. (However, this will not increase the pile capacity which may be inadequate.)</td>
<td>Careful consideration should be given to temporary stability, or the building may be vulnerable to even small earthquakes during implementation.</td>
</tr>
<tr>
<td></td>
<td>b. May be possible to use compaction grouting below the pile tips and either lift the piles themselves, or the whole soil block in which the piles are embedded.</td>
<td>Requires access for drilling in grout pipes; probably requires offshore expertise.</td>
</tr>
<tr>
<td>2. Structural damage</td>
<td>In many situations it should be possible to access and repair flexural damage if it is close to the pile caps. Damage here signals the possibility of damage at depth. This would need to be checked; possibly by drilling down the centre of the pile if not under the wall or column, or by angle borehole from alongside.</td>
<td>Difficulties in determining whether additional damage at depth exists may mean pile integrity cannot be relied on. An indirect approach is to assess pile damage at depth by analysis of the pile-soil kinematic interactions.</td>
</tr>
</tbody>
</table>

C4E.2 Lessons from Previous Earthquakes

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 C4E.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 C4E.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, 1991).
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 C4E.2 illustrates the example of a five storey building damaged in the Christchurch earthquake of 22 February 2011 (Kam et al., 2012). 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 settle 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 C4E.2(c)) which is likely to be a consequence of both seismic shaking and induced vertical displacement demand from the wall’s foundation failure. The building’s lateral load system was severely comprised due to the foundation-wall system failure and it partially collapsed in a subsequent aftershock.
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.

Figure C4E.3 presents several examples of significant building residual deformations due to foundation “failure” observed in the Christchurch CBD (Kam et al., 2012). 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 would exacerbate the inelastic demand on the superstructure’s plastic hinges such as those shown in Figure C4E.3(b) but may not necessarily result in a uncontrolled displacement response.

![Building foundation “failure”](image)

**Figure C4E.3: Building foundation “failure” (Cubrinovski and McCahon, 2012)**

**C4E.3 Further 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.

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Potential Strengths</th>
<th>Potential Weaknesses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basement and Raft</td>
<td>Resistance to lateral movement</td>
<td>Limited ductility, potential for differential settlement</td>
</tr>
<tr>
<td>Shallow Foundation</td>
<td>Cost-effective</td>
<td>Limited depth, potential for piping and piping</td>
</tr>
<tr>
<td>Deep Foundation</td>
<td>Higher load-bearing capacity</td>
<td>Higher cost, limited access to site</td>
</tr>
</tbody>
</table>

**Note:**

This information is for general guidance only. Each site and structure should undergo site-specific engineering assessment.
Table C4E.3: Summary of traditional foundation types

<table>
<thead>
<tr>
<th>Foundation type</th>
<th>Era</th>
<th>Brief description</th>
<th>Likely strengths</th>
<th>Likely weaknesses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driven timber piles</td>
<td>1890 - 1920</td>
<td>Round poles top driven to a set</td>
<td>• Durable when quality hardwood used, especially when submerged</td>
<td>• Degradation/rot, especially at top</td>
</tr>
<tr>
<td>Driven timber tip armoured</td>
<td>1890 - 1920</td>
<td>End tapered and protected with steel to penetrate stiffer layers</td>
<td>• Consistent capacity</td>
<td>• Poor engagement into foundation</td>
</tr>
<tr>
<td>Driven steel piles</td>
<td>Typical post 1970s</td>
<td>Commonly bare steel, sometimes galvanised or coated.</td>
<td>• Consistent capacity</td>
<td>• Rusting/degradation potentially very significant</td>
</tr>
<tr>
<td>Railway irons</td>
<td>1890s - 1910</td>
<td>Cast iron prior to ~1910</td>
<td>• Base bearing capacity consistent</td>
<td>• Variable engagement into foundation</td>
</tr>
<tr>
<td>Driven concrete piles</td>
<td>1915-1990s</td>
<td>Drilled pile, concrete poured at base and driven to provide consolidated end bearing and spread</td>
<td>• Base bearing capacity consistent</td>
<td>• Shear failure. Existing piles often have few stirrups and can fail in a brittle manner during ground lurch or lateral spreading.</td>
</tr>
<tr>
<td>Precast</td>
<td>1960s-1980s</td>
<td>Drilled pile with precast pile driven out through base</td>
<td>• Side friction variable dependent upon installation technique, but should be calculated considering it as a displacement pile</td>
<td>• Franki/Bulb piles are likely to have poor curtailment of reinforcement into the consolidated base, and so little tension capacity.</td>
</tr>
<tr>
<td>Franki/bulb piles</td>
<td>1970s-1980s</td>
<td>Driven tube with sacrificial steel base, casing withdrawn during casting</td>
<td>• Very old (&lt;1910) piles may have high quantities of non-Portland cement and hence be very durable</td>
<td>• Top fixity: does this work in both directions? Is it truly fixed at the top?</td>
</tr>
<tr>
<td>Bottom driven steel tube</td>
<td>1980s-1990s</td>
<td>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.</td>
<td>• Reinforcing easy to curtain into foundation beams</td>
<td></td>
</tr>
<tr>
<td>Bored piles</td>
<td>1860-1990</td>
<td>Multiple drilling techniques</td>
<td>• Often large robust sizes</td>
<td></td>
</tr>
<tr>
<td>Straight</td>
<td>1960-1990</td>
<td>Sides grooved with special tool after drilling</td>
<td>• Reinforcing easy to curtain into foundation beams</td>
<td></td>
</tr>
<tr>
<td>Straight grooved</td>
<td>1960-1990</td>
<td>Specialist technique</td>
<td>• Be careful for distribution between skin friction and end bearing (relative stiffness and strength)</td>
<td></td>
</tr>
<tr>
<td>Belled</td>
<td>1960-1990</td>
<td>Specialist technique</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel screw piles</td>
<td>1990-1990</td>
<td>Specialist technique</td>
<td>• Records correlate capacity with depth</td>
<td>• Helices very flexible: vertical displacements often govern for seismic loads (soil/structure interaction)</td>
</tr>
<tr>
<td>Ground anchors</td>
<td>1960-1990</td>
<td>Drilled and grouted hole, bar or strand anchors.</td>
<td>• Testing results should be available</td>
<td>• Small contribution to base-shear resistance</td>
</tr>
<tr>
<td>Drilled and inserted</td>
<td>1960-1990</td>
<td>Drilled and grouted hole, bar or strand anchors.</td>
<td>• High capacity can be installed in small space</td>
<td>• Poorer performance under cyclic load</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Free length gives controlled plastic</td>
<td>• Limited compression capacity: critical if</td>
</tr>
<tr>
<td>Foundation type</td>
<td>Era</td>
<td>Brief description</td>
<td>Likely strengths</td>
<td>Likely weaknesses</td>
</tr>
<tr>
<td>---------------------------------</td>
<td>--------</td>
<td>------------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Pressure grouted/drilled</td>
<td>1990-</td>
<td>Proprietary bar drilled specialist technique</td>
<td>elongation if required</td>
<td>building settles due to liquefaction</td>
</tr>
<tr>
<td>Deadman</td>
<td>18??-</td>
<td>Relies on steel bars back to mass or reinforced concrete passive acting blocks</td>
<td>• Testing records may be available</td>
<td>• Little to no shear capacity; vulnerable to lurch or lateral spreading</td>
</tr>
<tr>
<td>Mechanical expansion</td>
<td>1970-</td>
<td>Rock bolts with expansive ends</td>
<td>• Can often be re-tested to prove capacity</td>
<td>• Durability critical, especially around anchorages (esp. for both ends of deadman anchorages)</td>
</tr>
<tr>
<td>Grout expansion</td>
<td>1990-</td>
<td>Proprietary grouted tubes which &quot;unroll&quot;</td>
<td></td>
<td>• Potential “brittle” behaviour (reduced grout to country bond with strain)</td>
</tr>
<tr>
<td>Mechanical tip</td>
<td>1990-</td>
<td>Proprietary bearing engagement e.g. &quot;Duckbill/Manta Ray&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shallow</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brick strip</td>
<td>1840-</td>
<td>Nominal widening, sometimes incorporating site concrete</td>
<td>• Predictable well tested behaviour in “good ground”</td>
<td>• Affected significantly by liquefaction</td>
</tr>
<tr>
<td>Concrete strip</td>
<td>1840-</td>
<td>Reinforced or unreinforced</td>
<td>• Pads often oversized for older buildings</td>
<td>• Strip footings often undersized/highly stressed under brick walls.</td>
</tr>
<tr>
<td>Ground beam</td>
<td>1950-</td>
<td>Reinforced, likely spreading point loads</td>
<td>• Rafts can mitigate differential displacement</td>
<td>• Pre-1930s footings may not have continuous reinforcement</td>
</tr>
<tr>
<td>Isolated pad caisson</td>
<td>1840-</td>
<td>Reinforced</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Raft</td>
<td>1970-</td>
<td>Reinforced</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Domestic</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Timber ordinary</td>
<td>1840-</td>
<td>Rounds or squares excavated and concreted in place</td>
<td>• Typically small loads per unit</td>
<td>• Degradation of timber with time</td>
</tr>
<tr>
<td>Timber anchor</td>
<td>1980-</td>
<td>Square excavated and concreted in place</td>
<td></td>
<td>• Often lack of distributed resistance</td>
</tr>
<tr>
<td>Timber driven</td>
<td>1960-</td>
<td>Round or square</td>
<td></td>
<td>• Ensure structure fixed to foundations</td>
</tr>
<tr>
<td>Concrete ordinary</td>
<td>1920-</td>
<td>Precast, sometimes cast in “kerosene tins”</td>
<td></td>
<td>• “Ordinary” and predominant number of individual piles have little or no cantilever capacity</td>
</tr>
<tr>
<td>Concrete strip</td>
<td>1930-</td>
<td>Typical subfloor walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brick strip</td>
<td>1860-</td>
<td>Single or two wythe, sometimes in site concrete</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix C4F: Initial Screening of Ground Profile for Liquefaction Susceptibility

Soils susceptible to liquefaction may lose all 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 seismic forces during the earthquake.

ASCE 41-13 Table 8-1, reproduced below in Table C4F.1, provides a means of initial screening only of the ground profile to determine the site’s liquefaction susceptibility. Refer to ASCE 41-13 Section 8.2.2 for further details.

Table C4F.1: Estimated susceptibility to liquefaction of surficial deposits during strong ground shaking (ASCE 41-13, Table 8-1)

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>General distribution of cohesionless sediments in deposits</th>
<th>Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Modern &lt;500 years</td>
<td>Holocene &lt;11,000 years</td>
</tr>
<tr>
<td>(a) Continental Deposits</td>
<td></td>
<td></td>
</tr>
<tr>
<td>River channel</td>
<td>Locally variable</td>
<td>Very high</td>
</tr>
<tr>
<td>Flood plan</td>
<td>Locally variable</td>
<td>High</td>
</tr>
<tr>
<td>Alluvial fan, plain</td>
<td>Widespread</td>
<td>Moderate</td>
</tr>
<tr>
<td>Marine terrace</td>
<td>Widespread</td>
<td>-</td>
</tr>
<tr>
<td>Delta, fan delta</td>
<td>Widespread</td>
<td>High</td>
</tr>
<tr>
<td>Lacustrine, playa</td>
<td>Variable</td>
<td>High</td>
</tr>
<tr>
<td>Collovinium</td>
<td>Variable</td>
<td>High</td>
</tr>
<tr>
<td>Talus</td>
<td>Widespread</td>
<td>Low</td>
</tr>
<tr>
<td>Dune</td>
<td>Widespread</td>
<td>High</td>
</tr>
<tr>
<td>Loess</td>
<td>Variable</td>
<td>High</td>
</tr>
<tr>
<td>Glacial till</td>
<td>Variable</td>
<td>Low</td>
</tr>
<tr>
<td>Tuff</td>
<td>Rare</td>
<td>Low</td>
</tr>
<tr>
<td>Tephra</td>
<td>Widespread</td>
<td>High</td>
</tr>
<tr>
<td>Residual soils</td>
<td>Rare</td>
<td>Low</td>
</tr>
<tr>
<td>Sebka</td>
<td>Locally variable</td>
<td>High</td>
</tr>
<tr>
<td>(b) Coastal Zone Deposits</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Delta</td>
<td>Widespread</td>
<td>Very high</td>
</tr>
<tr>
<td>Esturine</td>
<td>Locally variable</td>
<td>High</td>
</tr>
<tr>
<td>Beach, high energy</td>
<td>Widespread</td>
<td>Moderate</td>
</tr>
<tr>
<td>Beach, low energy</td>
<td>Widespread</td>
<td>High</td>
</tr>
</tbody>
</table>
### (c) Fill Materials

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>General distribution of cohesionless sediments in deposits</th>
<th>Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by geologic age)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Modern &lt;500 years</td>
</tr>
<tr>
<td>Lagoon</td>
<td>Locally variable</td>
<td>High</td>
</tr>
<tr>
<td>Foreshore</td>
<td>Locally variable</td>
<td>High</td>
</tr>
<tr>
<td>Uncompacted fill</td>
<td>Variable</td>
<td>Very high</td>
</tr>
<tr>
<td>Compacted fill</td>
<td>Variable</td>
<td>Low</td>
</tr>
</tbody>
</table>

**Note:** Adapted from Youd and Perkins 1978
Appendix C4G: Initial Screening for Static Bearing Capacity

BS 8004:1996 Table 1, reproduced below in Table C4G.1, can be used for initial screening only for the static bearing capacity of different ground conditions.

This table should not be used for determining post-liquefaction bearing capacity.

Refer to BS 8004:1996 Section 2.2 for further details.

Table C4G.1: Presumed allowable bearing values under static loading (refer to 1.2.3 and 1.2.4) (BS 8004:1996 Table 1)

<table>
<thead>
<tr>
<th>Category</th>
<th>Types of rocks and soils</th>
<th>Presumed allowable bearing value</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rocks</td>
<td>Strong igneous and gneissic rocks in sound condition</td>
<td>10,000</td>
<td>These values are based on the assumption that the foundations are taken down to unweathered rock. For weak, weathered and broken rock, see 2.2.2.3.1.12.</td>
</tr>
<tr>
<td></td>
<td>Strong limestones and strong sandstones</td>
<td>4,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Schists an slates</td>
<td>3,000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strong shales, strong mudstones and strong siltstones</td>
<td>2,000</td>
<td></td>
</tr>
<tr>
<td>Non-cohesive</td>
<td>Dense gravel, or dense sand and gravel</td>
<td>&gt;600</td>
<td>Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation. For effect of relative density and groundwater level, refer to 2.2.2.3.2.</td>
</tr>
<tr>
<td>soils</td>
<td>Medium dense gravel, or medium dense sand and gravel</td>
<td>&lt;200-600</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose gravel, or loose sand and gravel</td>
<td>&lt;200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compact sand</td>
<td>&gt;300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense sand</td>
<td>100-300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose sand</td>
<td>&lt;100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Value depending on degree of looseness</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesive soils</td>
<td>Very stiff boulder clays and hard clays</td>
<td>300-600</td>
<td>Group 3 is susceptible to long-term consolidation settlement (refer to 2.1.2.3.3). For consistencies of clays, refer to Table 5.</td>
</tr>
<tr>
<td></td>
<td>Stiff clays</td>
<td>150-300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Firm clays</td>
<td>75-150</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soft clays and silts</td>
<td>&lt;75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Very soft clays and silts</td>
<td>Not applicable</td>
<td></td>
</tr>
<tr>
<td>Peat and organic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>soils</td>
<td></td>
<td>Not applicable</td>
<td>See 2.2.2.3.4</td>
</tr>
<tr>
<td>Made ground or</td>
<td></td>
<td>Not applicable</td>
<td>Refer to 2.2.2.3.5</td>
</tr>
<tr>
<td>fill</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note:
1. These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation (refer to 2.1.2.3.2 and 2.1.2.3.3).

* 107.25 kN/m² = 1.094 kgf/cm² = 1 tonf/ft²
Appendix C4H: Influence of Shaking Levels on Ground Stability and Liquefaction Susceptibility

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).

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 (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 C4H.1 below for examples of the correlation. The MMI-PGA correlation is extracted from Saunders and Berryman (2012).

An important step is for the assessor 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.

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 the Canterbury earthquake experience (comments by Dowrick et al., 2008 are retained in strikethrough for reference).

Table C4H.1: Approximate correlation between PGA, MMI and land damage descriptors

<table>
<thead>
<tr>
<th>PGA, g</th>
<th>MMI</th>
<th>Land descriptors (from Dowrick et al. (2008), with additional comment)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.03g</td>
<td>&lt;MM5</td>
<td>Land/slope issues are unlikely.</td>
</tr>
<tr>
<td>0.03-0.08</td>
<td>MM5</td>
<td>Loose boulders may occasionally be dislodged from steep slopes.</td>
</tr>
<tr>
<td>0.08-0.15</td>
<td>MM6</td>
<td>Loose material may be dislodged from sloping ground, e.g. existing slides, talus and scree slopes.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A few very small ($\leq 10^3$ m$^3$) soil and regolith slides and rockfalls from steep banks and cuts.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>A few minor cases of liquefaction (sand boil) in highly susceptible alluvial and estuarine deposits.</td>
</tr>
<tr>
<td>PGA, g</td>
<td>MMI</td>
<td>Land descriptors (from Dowrick et al. (2008), with additional comment)</td>
</tr>
<tr>
<td>--------</td>
<td>-----</td>
<td>----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>0.15-0.25</td>
<td>MM7</td>
<td>Small slides such as falls of sand and gravel banks, and small rock-falls from steep slopes and cuttings common. Instances of settlement of unconsolidated, or wet, or weak soils. Very small (≤10^3 m³) 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 m³), mainly rockfalls on steeper slopes (&gt;30°) 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.</td>
</tr>
<tr>
<td>0.25-0.45</td>
<td>MM8</td>
<td>Cracks appear on steep slopes and in wet ground. Significant landsliding likely in susceptible areas. Small to moderate (10^2-10^5 m³) 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 m³) landslides from coastal cliffs, and possibly large to very large (≥10^6 m³) 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.</td>
</tr>
<tr>
<td>0.45-0.60</td>
<td>MM9</td>
<td>Cracking of ground conspicuous. Landsliding widespread and damaging in susceptible terrain, particularly on slopes steeper than 20°. Extensive areas of shallow regolith failures and many rockfalls and disrupted rock and soil slides on moderate and steep slopes (20°-35° or greater), cliffs, escarpments, gorges, and man-made cuts. Many small to large (103-106 m³) failures of regolith and bedrock, and some very large landslides (106 m³ 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.</td>
</tr>
</tbody>
</table>

**Added comment:** Widespread damaging liquefaction in alluvial soils experienced across Christchurch and environs including lateral spread.
<table>
<thead>
<tr>
<th>PGA, g</th>
<th>MMI</th>
<th>Land descriptors (from Dowrick et al. (2008), with additional comment)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.60-0.80</td>
<td>MM10</td>
<td>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.</td>
</tr>
<tr>
<td>0.80-0.90</td>
<td>MM11</td>
<td>Environmental response criteria have not been suggested for MM11 as that level of shaking has not been reported in New Zealand or (definitively) elsewhere.</td>
</tr>
<tr>
<td>&gt; 0.90</td>
<td>MM12</td>
<td>As above.</td>
</tr>
</tbody>
</table>

**Note:**
* Refer to Dowrick et al. (2008), for full descriptors of building damage
Additional comments in bold are based on the Canterbury earthquake experience (original Dowrick et al., 2008 comments are retained in strikethrough for reference).