BUILDING PERFORMANCE

Earthquake geotechnical engineering practice

Module 1. Overview of the guidelines

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MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
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It is not mandatory to follow the guidance, but if followed:

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- users should consider taking appropriate professional advice prior to entering into a construction contract which incorporates all or parts of this document.

While the Ministry of Business, Innovation & Employment, Engineering New Zealand and the New Zealand Geotechnical Society have taken care in preparing this document, it is only a guide. It is not a substitute for legal advice or legislation.

All users should satisfy themselves as to the applicability of the content and should not act on the basis of any matter contained in this document without considering, and if necessary, taking appropriate professional advice.

The document may be updated from time to time and the latest version is available from the Ministry's website at www.building.govt.nz or the New Zealand Geotechnical Society's website at http://www.nzgs.org/

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The contents may be subject to changes, additions, and deletions. Submissions by the geotechnical community to the Society are encouraged, as it is intended that the guidelines will be updated from time to time.

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Preface

This document, Module 1, provides an overview of the series of guidelines for Earthquake Geotechnical Engineering Practice in New Zealand. It introduces the subject of earthquake geotechnical engineering, provides context within the building regulatory framework, provides guidance for estimating ground motion parameters for geotechnical design, and outlines the other modules in the series.

The series has been a collaborative exercise from the outset, originating from a panel discussion that occurred during the New Zealand Geotechnical Society (NZGS) Biennial Symposium in 2006 about the Loading Standard at the time not including the prediction of the effects of earthquakes on soil and thereby causing concern about variation in geotechnical practice. NZGS and the Ministry of Business Innovation and Employment (MBIE), have jointly developed this series to improve the standard of earthquake geotechnical engineering practice in New Zealand, promote consistency among the profession, and to address the lessons from the Canterbury and Kaikōura earthquakes and the Canterbury Earthquakes Royal Commission recommendations.

New Zealand is a high earthquake hazard region and the two very significant recent events (the 2010-11 Canterbury Earthquake Sequence and the 2016 Kaikōura Earthquake) have both underscored the importance of geotechnical considerations within the design of built environment in New Zealand, and helped to inform the guidance series.

This latest update and revision of Module 1 was undertaken with the support of Engineering New Zealand, incorporating feedback on the first revision, and provides updated hazard information as a result of recent investigation and research.

It should continue to be read in conjunction with the latest versions of the other modules:

› Module 2: Geotechnical investigations for earthquake engineering
› Module 3: Identification, assessment and mitigation of liquefaction hazards
› Module 4: Earthquake resistant foundation design
› Module 5: Ground improvement of soils prone to liquefaction
› Module 5A: Specification of ground improvement for residential properties in the Canterbury region
› Module 6: Earthquake Resistant Retaining Wall Design

The science and practice of earthquake geotechnical engineering is far from mature and is advancing at a rapid rate. It is intended that the *Guidelines* will be updated periodically to incorporate new advances in the field but 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 recommendations herein appropriately.

We would encourage you to make yourselves familiar with the series and apply it appropriately in practice.

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1 Introduction

New Zealand is a high earthquake hazard region and earthquake considerations are integral to the design of the built environment in New Zealand. The effects of earthquake shaking need to always be considered in geotechnical engineering practice and frequently are found to govern design.

Earthquake geotechnical engineering is a relatively young discipline of civil engineering that considers the geotechnical aspects of the wider discipline of earthquake engineering. Geotechnical conditions are critical to understanding the intensity and pattern of damaging ground shaking at a site. Ground failure from site instability, landslides, soil softening especially liquefaction, and lateral spreading are significant earthquake hazards. The design of foundations, retaining structures, horizontal and buried infrastructure to resist earthquake shaking, ground deformation and potential ground failure requires special consideration.

The high seismic hazard in New Zealand and profound relevance of earthquake geotechnical engineering were demonstrated by the 2010-2011 Canterbury Earthquake Sequence (CES). Christchurch and Canterbury were hit hard by a series of strong earthquakes generated by previously unmapped faults located in the vicinity or within the city boundaries. In the period between 4 September 2010 and December 2011, the intense seismic activity produced the magnitude (M_w) 7.1 Darfield event, the destructive 22 February 2011 M_w 6.2 earthquake, 12 other M_w 5 to 6 earthquakes, and over one hundred M_w 4 to 5 earthquakes. The 22 February 2011 earthquake was the most devastating causing 185 fatalities, the collapse of two multi-storey buildings, and the need for nearly total rebuild of the Central Business District.

The geotechnical aspects and impacts of the earthquakes were of economic and societal significance. The Canterbury earthquakes triggered widespread liquefaction in the eastern suburbs of Christchurch, as well as rock slides, rockfalls and cliff instabilities in the Port Hills affecting tens of thousands of residential buildings, and causing extensive damage to the lifelines and infrastructure over much of the city. About half of the total economic loss caused by CES could be attributed to the geotechnical impacts of the earthquake-induced liquefaction and rockslides.

More recently, in the 2016 Kaikōura earthquake, widespread surface fault ruptures and tens of thousands of landslides affected the transportation infrastructure, lifeline networks and farmland throughout a large source zone in the South Island. The magnitude 7.8 Kaikōura earthquake caused disproportionate impacts in Wellington (Cubrinovski et al., 2020), approximately 60 km from the source, including extensive liquefaction-induced damage in the reclamations of the port of Wellington (Cubrinovski et al., 2017). These most recent events provide clear evidence that the geotechnical impacts of strong earthquakes are significant and often dominant in New Zealand setting, and therefore they require careful considerations by national and regional government, stakeholders and practicing engineers.

1.1 Objective

While there is a substantial and rapidly growing body of published research on the subject of earthquake geotechnical engineering, most of this information is relatively dispersed in journal articles and conference proceedings making it difficult for practising engineers to keep abreast of developments and what may be considered 'state of practice'.

There are few comprehensive textbooks or monographs on the subject with some notable exceptions. (Kramer, 1996; Towhata, 2008; Idriss and Boulanger, 2008).

The objective of the Guidelines is to help summarise current practice in earthquake geotechnical engineering with a focus on New Zealand conditions, regulatory framework, and practice. The Guidelines are not intended to be a detailed treatise of latest research in earthquake geotechnical engineering, which continues to advance rapidly. Instead, this document is intended to provide sound guidelines to support rational design approaches for everyday situations, which are informed by latest research. Complex and unusual situations are not covered. In these cases, special or site-specific studies are considered more appropriate.

The main purpose of the Guidelines is to promote consistency of approach to everyday engineering practice in New Zealand and, thus, improve geotechnical-earthquake aspects of the performance of the built environment.

These Guidelines are not a book of rules — users are assumed to be qualified, practicing geotechnical engineering professionals with sufficient experience and knowledge to apply professional judgement in interpreting and applying the recommendations contained herein.

Neither are the Guidelines intended to be a primer on the subject of earthquake geotechnical engineering — readers are assumed to have a sound background in soil mechanics, geotechnical engineering, and earthquake engineering. A thorough foundation for earthquake geotechnical engineering is provided by Kramer (1996) and users of the Guidelines should be familiar with the material covered therein.

The science and practice of earthquake geotechnical engineering is advancing at a rapid rate. The users of this document should familiarise themselves with recent advances and interpret and apply the recommendations herein appropriately.

1.2 Intended audience

These Guidelines have been prepared, generally, for the use of qualified, practising geotechnical engineers with a sound background in soil mechanics, geotechnical engineering, and earthquake engineering.

Module 2: Site investigations

Is intended to be used by both qualified, practising geotechnical engineers and engineering geologists to guide planning and execution of geotechnical investigations.

Module 4: Foundations, and Module 6: Retaining walls

Will also be of interest to practising structural engineers although it is intended that they should work in close collaboration with geotechnical engineering professionals to develop designs for significant building foundations and retaining structures.

1.3 Professional collaboration

Geotechnical considerations are crucial to successful designs for any part of the built environment, especially in New Zealand's high earthquake hazard environment.

Successful outcomes require close collaboration among the key professionals (geotechnical engineers, engineering geologists and structural engineers) to properly consider the site geology, earthquake hazards, site response, soil response, foundation behaviour, structural interactions and soil-structure system response.

A proper understanding of the site geology is essential and requires collaboration between the geotechnical engineer and engineering geologist with inputs from the structural engineer to understand the site requirements for the proposed structure and any possible site-structure interactions.

A full consideration of the site response and soil response to shaking together with a sound understanding of the structural response including soil-structure interaction is essential to make appropriate selections of suitable foundation types or ground treatments, requiring close collaboration between the geotechnical and structural engineers. Geotechnical and structural engineers may have different performance objectives in mind, or simply do not clearly understand what each discipline contributes or is able to contribute to the design process, or what actually matters for design (Oliver et al, 2013). Good design solutions require that the geotechnical and structural engineers sit down together to share each professional's perspective of the project and coming to a shared understanding of all of the issues and interactions required for a successful outcome. The result would ideally be a joint report outlining the expected performance of the site, ground, foundations, and structure including their critical interactions and consideration of system response and effects. Furthermore, the collaboration should aim for a holistic approach involving the client and design team through different phases of the process from the understanding of client's brief and performance objectives to soil-structure interaction issues.

1.4 General assessment principles

Earthquake geotechnical engineering problems require adequate treatment in all phases of the assessment procedure, including evaluation of seismic loads, site investigations, hazard identification, site and soil characterisation, use of appropriate assessment methodology, analyses, interpretation and engineering judgement.

Consideration of uncertainties is critically important throughout the assessment process. The level of detail and particular features of the assessment procedure should be balanced across all phases. They also should be appropriate for the scale of the project, the importance of the facilities planned for the site, the level of risk associated with the hazard and potential consequences of failure in terms of loss of life, economic loss, and impacts on communities.

Geotechnical professionals increasingly rely on computer software to carry out analysis and design including liquefaction assessments, slope stability assessments, foundation design, and advanced numerical modelling using finite element and finite difference methods. The benefits include increased productivity and, when used properly, useful additional insights from parametric studies, system response considerations and rapid prototyping.

However, users need to have a sound understanding of the analysis methods being implemented within each software package including the inherent limitations and uncertainties of each, otherwise the results and their interpretation may be misleading and potentially dangerous. The quality and reliability of the outputs directly depends on the quality of the inputs — mainly soil parameters that are intrinsically variable and difficult to measure. Uncertainties in both input parameters and output results should be considered by use of parametric and sensitivity studies, and by use of multiple analysis methods or models. It remains the professional responsibility of the user to interpret and validate the results based on expertise and engineering judgement.

1.5 The Building Code and Guidance

This section provides an overview of how the Modules fit into the Building Code system.

1.5.1 BUILDING CODE SYSTEM

The New Zealand Building Code is regulation that sets out the performance criteria to be met for all new building work in New Zealand. The Building Code does not prescribe how work should be done but states how completed building work and its parts must perform.

The Stability section of the Code (Clause B1) includes requirements that buildings, building elements and site work shall have:

- a low probability of rupturing, becoming unstable, losing equilibrium or collapsing during construction or alteration and throughout their lives; and
- a low probability of causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives.

The guidance contained in this earthquake geotechnical engineering practice series is intended to support consideration of these performance requirements with a specific focus on the potential impacts of earthquakes on buildings, as well as the practices that have, and continue to be, developed to address these risks.

Figure 1 provides an overview of the design process and the interaction with legal requirements. Further details on the New Zealand building regulatory system are provided in Section 3.2 and Appendix B.

Getting a clear understanding of client requirements is always the starting point. Client requirements may well exceed the minimum performance levels set in the Building Code, depending on their objectives and purposes for the building, but they **cannot** be set at a lower level than the Building Code. Once the design brief is developed and agreed, and concept design is completed, decisions can be made about the design approach, the type of ground investigations needed, the process necessary for estimating ground motion parameters, and whether a simplified or more complex dynamic non-linear analysis is appropriate. The design is undertaken following a check for legal minimum requirements, with close collaboration of all other parties, including structural designers and the Building Consent Authority, BCA.

The regulatory triangle on the right-hand side of Figure 1 describes the Building Code system, providing the hierarchy for the minimum legal requirements and compliance pathways. The diagram also illustrates that Section 175 guidance can be issued by MBIE to better inform and provide greater clarity for designers and BCAs. Guidance can be targeted at three levels:

- Building Act provisions
- Building Code performance requirements; or
- for design solutions, including Alternative Solutions.

The Building Act: 2004, (B.A. s.17), requires that all building work must comply with the Building Code. The Building Code 'prescribes functional requirements for buildings and the performance criteria with which buildings must comply in their intended use' (B.A. s.16). The performance-based Building Code is detailed in Schedule 1 of Building Regulations 1992. Internationally there has been a strong move towards performance-based Building Codes as they focus on minimum outcomes required, not on the means to get there. They allow for innovation in methods and materials used, greater flexibility to meet client requirements and quicker uptake of new knowledge. The Objective, Functional Requirement, and Performance is stated in the Code and it is up to the designer to meet these outcomes whatever compliance pathway is followed.

Figure 1. Design process and regulatory context

START

Client requirements

Compliance with design brief and any special client requirements, consider:

› Attitude to risk (how critical is continuing functionality?)
› Agree performance criteria
› Type of hazard analysis needed?
› Level of investigations needed?
› Level of design methods used?
› Geotech/structural collaboration

CHECK

Legal requirements

Compliance with the:

› Building Act
› Building Code

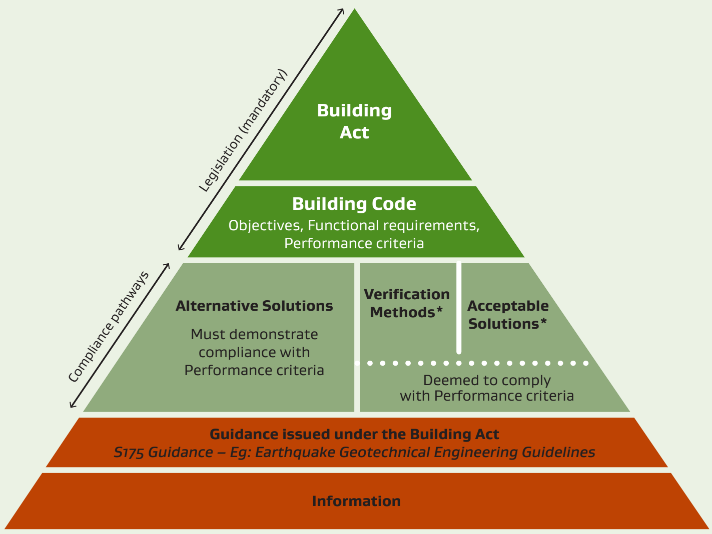
UNDERTAKE DESIGN

(per client requirements with Building Code as minimum)

Design solution options

Compliance with design with brief and any special client requirements, consider:

› Acceptable solutions eg B1/AS1 (deemed to comply)
› Verification method eg B1/VM1 (deemed to comply)
› Alternative solutions—demonstraion of performance using, for example:
  – New Zealand or International standards
  – s175 guidance, eg Geotechnical modules
  – First principle design
  – Expert knowledge
  – History of performance
  – MBIE determination, etc.



** may include cited standards and information*

There are different pathways for demonstrating compliance with the Code. Prescriptive Acceptable Solutions (AS) and Verification Methods (VM) provide a 'deemed to comply' pathway. This means designs following the Acceptable Solutions and Verification Methods published by MBIE, eg Verification Method B1/VM1, or Acceptable Solution B1/AS1, must be accepted by the Building Consent Authority, BCA, when making Building Consent decisions. Verification Methods and Acceptable Solutions often reference specific New Zealand or international standards, eg NZS 1170.5 is referenced in B1/VM1 and NZS 3604 is referenced in B1/AS1. However, geotechnical engineers need to be sure that the VM or AS being referenced is appropriate for their specific application. Alternative Solutions provide an opportunity for the designer to propose and demonstrate that their design fully meets the performance requirements set in the Code. Acceptable Solutions and Verification Methods are not fully comprehensive, and most projects have an Alternative Solution element, requiring additional analysis and considerations.

This is particularly the case for geotechnical engineering as considerable complexities are often encountered in projects but are not necessarily considered in following the prescriptive pathway. Caution is necessary if using the prescriptive ('deemed to comply') pathway, as the relevant Verification Method, B1/VM4, has a narrow scope of application, and some aspects are thought to no longer meet current design practice. This is an area being reviewed by MBIE.

The BCA must issue a Consent if it is 'satisfied on reasonable grounds that the provisions of the building code would be met if the building work were properly completed in accordance with the plans and specifications that accompanied the application' (B.A. s.49).

Following s.175 guidance that has a legal status, helps BCAs satisfy the 'reasonable grounds' test. Section 19(2)(b) of the Building Act specifically provides for BCAs to take into account Section 175 guidance when making consent decisions. However, it is important that the designer recognises the limits of their competence and experience and that they are capable of undertaking the project design requirements. This capability will be a factor the Building Consent Authority considers when assessing a building consent application and applying the 'reasonable grounds' test.

1.5.2 SECTION 175 GUIDANCE

Given the complexity and uncertainties associated Given the complexity and uncertainties associated with earthquake geotechnical engineering problems, as well as the unique features of each site and consequent large variety of soil characteristics and ground conditions encountered, geotechnical practice generally relies on guidance documents rather than prescriptive standards. They can provide more appropriate support for geotechnical engineers and designers, and hence are the accepted norm internationally. For this reason, it was decided that publishing s.175 guidance was the best means of getting better consistency and improving general practice in earthquake geotechnical engineering in New Zealand. This decision followed consultations and advice from practitioners and the New Zealand Geotechnical Society. Apart from some straightforward low-risk situations and limited applications, prescriptive 'deemed to satisfy' Verification Methods or Acceptable Solutions are not able to address complex issues requiring engineering evaluation and judgement.

As guidance, the Modules provide general principles using the latest research knowledge so that practitioners are aware of and focus on key issues in the assessment, rather than producing detailed calculation methods, often available in textbooks and journal publications. Some worked examples have been produced to provide more detail. Also, guidance can be updated periodically to incorporate new advances in the rapidly evolving field of earthquake geotechnical engineering.

The building process can be complex. There are numerous site conditions, products, design methods and building systems that can be used to carry out a construction project. Sound engineering decision making is required throughout the course of the project to provide the client with a robust outcome. This will include specifying the type and scope of field investigations and laboratory testing of the soils appropriate to the level of risk, interpretation of soils and the site, modelling, analysis and interpretation as well as on-going monitoring throughout construction to verify assumptions made in the design. Good design is not a 'box ticking' exercise. It is difficult to address all these aspects in prescriptive documents; sound and experienced engineering judgement is required throughout.

2 Scope

The material in this document relates specifically to earthquake hazards and should not be assumed to have wider applicability. It is intended to provide general guidance for earthquake geotechnical engineering practice in New Zealand.

The recommendations in this document are intended to be applied to everyday engineering practice by qualified and experienced geotechnical engineering professionals who are expected to also apply sound engineering judgement in adapting the recommendations to each particular situation. Complex and unusual situations are not covered. In these cases, special or site-specific studies are considered more appropriate and additional guidance sought.

Other documents may provide more specific guidelines or rules for specialist structures, and these should, in general, take precedence over this document.

Examples include:

› New Zealand Society on Large Dams — Dam Safety Guidelines

› NZ Transport Agency — Bridge Manual

› Transpower New Zealand — Transmission Structure Foundation Manual

Where significant discrepancies are identified among different guidelines and design manuals, it is the responsibility of the engineer to resolve such discrepancies as far as practicable.

The recommendations made in this document may seem excessive or burdensome for very small projects such as single unit dwellings. The intention is that earthquake hazards (and all geotechnical hazards) should be properly investigated and assessed at the subdivision stage of development when appropriate expenditures can be more easily justified. Simpler investigations and assessments would be then likely be adequate for individual sites. Professional judgement needs to be applied in all cases.

More specific guidance has been issued by MBIE for the repair and rebuilding of residential dwelling foundations in the Canterbury earthquake region (MBIE, 2012) and this should take precedence over these Guidelines. However, the MBIE Guidance is specifically for use within the Canterbury earthquake region only and it may not be appropriate to use it elsewhere.

3 Geotechnical considerations for the built environment

3.1 Introduction

This section considers the key geotechnical performance requirements for the built environment prior to, during, and after earthquake shaking in the context of the New Zealand building regulatory environment.

Clause B1 of the Building Code expands on the general purpose of the Building Act to ensure safety by including objectives to:

- safeguard people from injury caused by structural failure
- safeguard people from loss of amenity caused by structural behaviour
- protect other property from physical damage caused by structural failure.

Buildings, building elements and site-works are required to have a low probability of:

- rupturing, becoming unstable, losing equilibrium or collapsing during construction, alteration, and throughout their lives
- causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, during construction, alteration, or when the building is in use.

Account is required to be taken of various physical conditions including:

- earthquake
- earth pressure
- differential movement
- time-dependent effects such as creep and shrinkage
- removal of support.

Site-work is required to be carried out so as to provide stability for construction and to avoid the likelihood of damage to other property. It must achieve this while taking account of:

- changes in ground water level
- water, weather and vegetation
- ground loss and slumping.

Geotechnical considerations are clearly an essential part of the design and construction of any building development. Failing to demonstrate compliance with the above requirements because of geotechnical deficiencies would result in failure to obtain a building consent.

Issue of a building consent would also be dependent on the land generally meeting the stability requirements of the Resource Management Act. Section 106 gives a consenting authority the power to refuse a subdivision consent if the land is subject to erosion, subsidence, slippage or inundation. Section 220 refers to similar criteria.

Geotechnical considerations are crucial to successful design of any part of the built environment. There is a strong need to raise awareness of the importance of the application of geotechnical engineering skills and knowledge in every aspect of building development. This will involve the following:

› a review of the geological, seismological, and geotechnical context of the development site
› specific investigation and gathering of geotechnical and related data
› development of geotechnical design parameters appropriate to the building development and the site
› due account of geotechnical considerations in the design of the building development so that it meets the requirements of the building code
› due consideration of geotechnical factors, including overall land stability, prior to the issue of resource and building consents
› review of geotechnical conditions and modification of design details as necessary during construction.

While not explicitly stated, for each of these factors, due consideration of the effects of earthquakes (ground shaking, ground deformation and failure, and fault displacements) must clearly be included in every geotechnical assessment.

Geotechnical considerations are essential to achieve satisfactory performance of the built environment during earthquakes, including site stability and control of settlements and distortion of buildings and other structures.

Earthquake actions differ from other design actions in several important respects, such as:

a Earthquake actions are caused by ground accelerations with characteristics that vary greatly from one earthquake to another, and that cannot be accurately predicted. Instead, ground motions based on probabilistic analysis and holistic considerations are estimated for design. There is always a residual risk that the actual earthquake actions will be greater than the code-specified design actions, and therefore, structures (including their foundations) should be made sufficiently robust to accommodate such 'overloading' in a progressive manner, so as to avoid sudden collapse (a requirement of NZS1170.5).

b The ground motion is transmitted into the building through the foundations. Compliance and yielding of the foundations may alter the dynamic response of the building, eg by lengthening the natural period of vibration and increasing the damping of the system (ie soil-structure-interaction effects). In addition, the resulting relative displacements at the base of the structure may damage the foundations and building service connections, and foundation settlement and rotations may increase the building displacements and place additional demands on the superstructure (see Module 4, Section 3.8 and 3.9 for discussion of *soil-structure-interaction* effects and *performance-based design* in general).

c Earthquake shaking may reduce the strength and stiffness of the founding soils and the bearing capacity of the foundations. Certain soils may lose almost all of their strength and stiffness due to soil liquefaction. The degradation in foundation performance resulting from liquefaction may jeopardize the stability and integrity of the structure, and therefore it must be carefully considered in the assessment (ie within the site assessment discussed in Module 3, and foundation selection process, covered in detail in Module 4, Section 4).

d Earthquake shaking causes shear deformations within the ground below the surface that induce shear and bending strains in buried foundation elements, especially deep piles, including both time-dependant and permanent strains (ie *kinematic loads* in deep foundations). These strains are in addition to those caused by inertial loads from the building and may damage the piles such that they can no longer safely carry the weight of the building. Kinematic effects are most pronounced where deep piles pass through liquefied soil layers. (Kinematic effects on piles are discussed in detail in Module 4, Section 6).

e The inertial response of the building induces dynamic, cyclic loading of the foundations that increase settlements by a process of 'ratcheting'.

f Earthquake actions include lateral loads from building inertia applied at the foundation level (and moment loads), simultaneous with vertical load. The lateral and moment loads may reduce the bearing capacity of shallow foundations and cause structural damage to deep foundations.

These effects are considered in more detail in Module 4, Sections 5 and 6. The overturning forces may result in a net uplift load being applied to individual foundation elements. Deep foundations may be used to resist these uplift loads with details given in Module 4, Section 6. The possibility of not resisting these uplift forces, and thus permitting rocking of the building, is included in the discussion of *performance-based design* in Module 4, Section 3.9.

All of the above effects place demands on building foundation performance that are additional to those from the gravity and other load combinations and require careful evaluation and consideration in the design.

Performance of the site and site subsoils during earthquake shaking are critically important to meeting building performance objectives. Site assessment and foundation selection is discussed in detail in Modules 3 and 4.

3.2 Design requirements

All building work in New Zealand must comply with the New Zealand Building Code.

For most building works, compliance with the Building Code is established by conformance to *Verification Method* B1/VM1 for structural design published by the Ministry of Building, Innovation and Employment. However, New Zealand's Building Code is performance based and alternative methods of demonstrating compliance are possible (termed *alternative solutions* in the Building Regulations), as discussed in Section 1.5. A more detailed overview of the New Zealand building regulatory system is given in Appendix B.

Verification method B1/VM1 is essentially a strength-based design procedure, where loads to be resisted by the foundations are determined by the structural engineer after analysis of the building and using structural actions and combinations of actions specified in AS/NZS 1170.0:2002. Combinations of self-weight, live load, wind, snow, earthquake, static liquid pressure, ground water, rainwater ponding, and earth pressure are considered. The resulting design actions to be applied to the foundation elements include vertical, horizontal, and moment loads.

Earthquake structural design actions for buildings in New Zealand are specified in NZS1170.5:2004 according to location, subsoil conditions, building period, and earthquake return period. NZS1170.5:2004 also includes more specific requirements for methods of structural analysis for earthquake actions. (Note that these structural design actions are not intended to be used for geotechnical assessment or design; see Section 5 for more information).

NZS1170.5:2004 includes a requirement that ultimate limit state deformations be limited so that the structural system continues to safely perform its load bearing function, contact with neighbouring buildings is avoided, parts continue to be supported, and non-structural systems necessary for emergency evacuation of the building continue to function. Foundation movements will contribute to the building deformations and need to be considered.

Two limit states for the building are required to be considered separately by designers under NZS 1170:

› the serviceability limit state (SLS), corresponding to specified service criteria for a building (for foundation design these are settlements, especially differential settlements criteria), and
› the ultimate limit state (ULS) corresponding to specified strength and stability criteria together with a requirement for robustness (an ability to withstand overload without collapse).

3.3 Serviceability limit state (SLS)

According to NZS 1170.0:2002, '*Serviceability limit states, are states that correspond to conditions beyond which specified service criteria for a structure or structural element are no longer met.*
(Note: The criteria are based on the intended use and may include limits on deformation, vibratory response, degradation or other physical aspects.)'

SLS design actions and combinations of actions are considered highly likely (ie with probability of 86 percent) to occur during the lifetime of the building. Two serviceability limit states are considered by NZS 1170.0:2002: SLS1 and SLS2.

- SLS1 is a requirement for all buildings of Importance Level IL2 or above.
- SLS2 is a requirement only for buildings of Importance Level 4 (ie structures with special post-disaster functions).

The annual probability of exceedance for each is given as:

- SLS1 1/25 (except for buildings of low importance, IL 1, which have no SLS requirement)
- SLS2 1/500 (for buildings with *normal*, 50-year design life, see NZS1170.0:2002 for other cases).

The specified service criteria for earthquake shaking for both SLS1 and SLS2 are described in NZS1170.5:2004 as follows:

Deformation shall be limited at the serviceability limit state so that:

1. *At the SLS1 level, structural system members and parts of structures shall not experience deformations that result in damage that would prevent the structure from being used as originally intended without repair.*
2. *At the SLS2 level, for structures of importance level 4, all parts of the structure shall remain operational so that the structure performs the role that has resulted in it being assigned this importance level.*

Classification of importance levels for building types (New Zealand Structures) is given in Table 3.2 of AS/NZS 1170.0:2002, whereas annual probability of exceedance for serviceability and ultimate limit states are given in Table 3.3 of AS/NZS 1170.0:2002, for structures with design working life in the range between 6 months and 100 years.

Foundation performance is critical to meeting these building performance criteria. Settlement limits, both total and differential, need to be agreed with the structural engineer and architect (since these will be critical to limiting damage to the structure and fabric of the building) and with the owner (since these will affect the continuance of the intended use of the building). Lateral movement of the foundation elements relative to the ground should be limited to tolerable values to prevent damage to buried service connections unless special flexible design details are used with greater movement capability.

Tolerable settlements at the SLS are highly dependent on the type of structure and its intended use. Guidance for different types of structures is given in Table C1 of AS/NZS 1170.0:2002.

Strong earthquake shaking will almost always increase building settlements to some degree because of the addition of dynamic, inertial loads from the building and cyclic ratcheting effects. For sites comprised of loose or soft soils, where liquefaction or cyclic softening of the soils are expected at the SLS level of shaking, the increase in settlement may be intolerable and far exceed the SLS threshold.

Comment

The selection of tolerable settlements for the foundations of buildings, at the serviceability limit state, is a complex topic beyond the intended scope of these Guidelines. The entire building may settle vertically or rotate as a rigid body by significant amounts without causing any structural or architectural distress (although there are limits beyond which aesthetic and serviceability considerations would be of concern). It is *differential* settlements, below the tilt line (see Figure 3.1 in Module 4), inducing structural deformation of the building, that will be the cause of distress to the building fabric.

Traditionally, for gravity loading, the assumption has been made [Terzaghi & Peck, 1967] '*that most ordinary structures such as office buildings, apartment houses, or factories, can withstand a differential settlement between columns of three quarters of an inch*' (20 mm) and that such a differential settlement would not be exceeded if the largest footing were designed to settle no more than 1 inch (25 mm) on the loosest part of the soil deposit. (Note: this guidance was intended for gravity loading and earthquake loading may induce quite different patterns of settlements in a building.)

For certain buildings and for certain uses, 20 mm differential settlement might cause significant loss of amenity, while for other cases much larger movements would be tolerable. The linkage between *loss of amenity*, as intended by the NZ Building Code, *serviceability*, as defined by NZS1170.0:2002, and *allowable bearing pressure*, as used in common practice to design shallow footings, needs careful consideration.

The selection of settlement criteria for building foundations at the SLS should not be a decision of the geotechnical engineer in isolation, but should be agreed and documented with the structural engineer, architect and owner, as appropriate, depending on the structural form, building fabric, and intended use and performance of the building. (Note: the codes stipulate minimum requirements; more rigorous criteria should be used when an improved performance above the minimum code requirement is sought by the owner.)

It is also important to accept that on deep alluvial sites, some permanent ground deformation must be expected in a large earthquake which will clearly impact on shallow foundations and may also result in problems for deep foundations.

3.4 Ultimate limit state (ULS)

ULS design actions and combinations of actions are considered much less likely to occur during the lifetime of the building, but are required to be resisted with a very low risk of structural collapse or failure of parts relevant to life safety.

For buildings of normal importance (Importance Level 2) with a normal (50 year) design life, earthquake shaking with a 500-year return period is considered for ULS. Return periods for buildings of other importance levels and design life duration are defined in NZS 1170.0:2002.

Building damage should be limited and controlled when subjected to the ULS earthquake shaking so that the risk of building collapse is very low and so that evacuation of the building occupants may be safely carried out. The building design should be robust (ie able to resist greater loads and displacements than those defined for ULS without collapse) because of the possibility of earthquake loading in excess of the ULS design level.

The ULS is centred on life safety and accepts the possibility of significant damage to the building, even resulting in its demolition. Some building owners may want better performance, with the ability to readily repair damage and to continue using the building. In effect, this approach requires customised design-criteria exceeding those based on the importance level of the building using low damage design principles. As foundation damage is frequently difficult or even impossible to repair, acceptable deformation and damage levels for the foundation relative to above ground structural members need to be clearly defined in consultation with the building owner and structural engineer, at the outset of the design.

The foundations, and ground supporting them, form an essential component of the overall building system, and their ability to continue carrying the weight of the building prior to, during, and after an earthquake is critical. Failure or excessive settlement of the foundation elements may threaten the stability of the building, prevent the intended lateral resistance mechanisms from developing, and cause excessive ductility demands on building elements, thereby increasing the risk of collapse.

On the other hand, resistance to **lateral** seismic loading (ie sliding) is not necessarily critical to the safe performance of buildings in cases of foundation systems well tied together. Lateral deformations are 'self-limiting' in the sense that lateral acceleration pulses are of short duration and are cyclic in nature, ie act in both (all) directions. (Note: caution is required for situations in which biased lateral loading may occur such as for buildings located on slopes or in the near-source region where strong unilateral pulses of loading may occur.)

For buildings on shallow foundations, relative lateral displacement (sliding) may be acceptable in many cases, provided these are within tolerable limits and building access and critical service connections are detailed accordingly.

For buildings on deep pile foundations, some relative lateral movement and foundation compliance may be beneficial in reducing the dynamic response of the building. However, care must be taken to ensure that the resulting lateral displacements do not damage the piles or reduce their ability to safely carry the building weight (in combination with kinematic effects).

The foundations for capacity designed buildings must be capable of resisting the over-strength actions from the building structure, otherwise the intended response of the superstructure cannot eventuate. The foundations of a building should not fail or deform excessively prior to the building developing its full intended structural response, including member over-strengths.

Comment

Eurocode 8 specifically requires foundation design loads for 'dissipative structures' (ie structures where ductile yielding is being used to dissipate energy and reduce building response) to account for the development of possible over-strength of the building.

Under NZS1170.5:2004, *deformations* of the foundations under the ULS loads (including over-strength loads) should be considered as well as *ultimate resistance*. While there is no requirement to achieve the same low level of deformation as for the SLS case, the foundation deformations should be accounted for in the structural design, and should not be so great that they add significantly to the ductility demand of the structure or prevent the intended structural response from developing.

Tolerable limits for foundation deformation at the ULS will depend on the structural form of the building and the building response mechanism intended by the structural engineer. Deformation limits should be agreed and documented between the geotechnical engineer and the structural engineer.

At present, there is little detailed guidance available to be able to predict foundation settlements with earthquake loading at the ULS, especially where liquefaction or cyclic softening of the founding soils is expected to occur (refer to Bray & Dashti 2014 for more information). For important projects, advanced numerical procedures show promise and may be justified but for most everyday situations a pragmatic, conservative approach of limiting plastic deformations of foundations under the calculated loads and avoiding situations where liquefaction and cyclic softening effects may be significant is recommended.

3.5 Other limit states

Under verification method B1/VM1 and NZS 1170.0-2002, there is no a specific requirement to consider earthquake events intermediate between the SLS and ULS levels of shaking, the assumption being that there would be a continuum of performance of the structure between the SLS and ULS limit states (except SLS2 for IL4 buildings).

However, the behaviour of soils and geotechnical systems under earthquake shaking may be highly non-linear and even exhibit a pronounced 'step change' in performance with increasing intensity of shaking. Typical examples include sites affected by liquefaction or slope instability. For such cases, only considering performance at the SLS and ULS levels of shaking would fail to identify potentially poor and unacceptable performance at intermediate return periods of shaking.

Consider two hypothetical cases shown in Figure 2. Case A shows a system for which large ground response (eg liquefaction, lateral spreading, or slope instability) is triggered for a ground motion intensity corresponding to a 40-year return period, whereas in Case B the triggering of ground failure occurs for a 400-year return period motion. Importantly, both cases show small response and acceptable performance for SLS level of shaking (25-year return period), and large, damaging, but tolerable response for ULS level of shaking (500-year return period). Hence, if one evaluates these two systems by discrete calculations for SLS and ULS alone, then the conclusion would be that both systems would exhibit similar seismic performance.

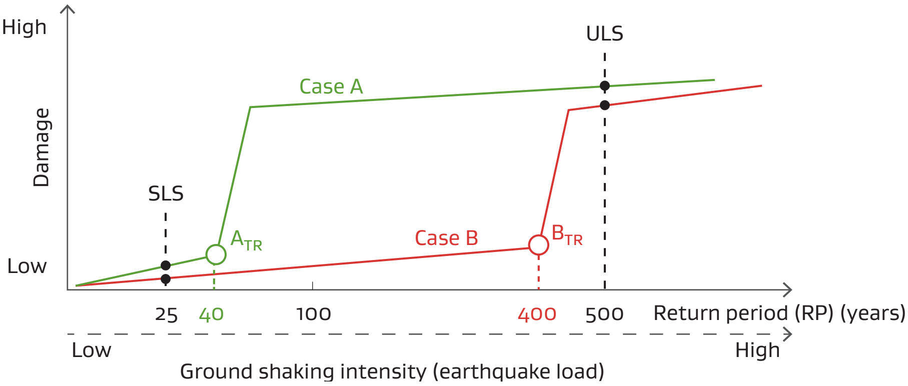
However, Figure 2 clearly depicts a significant difference in the performance between Case A and Case B, for intermediate limit states between SLS and ULS. Considering a normal 50-year lifespan of a structure, for Case A there is approximately 70 percent probability that triggering (or poor seismic performance due to significant damage) will occur during the life of the structure. The respective probability for poor performance (triggering of ground failure) during the life of the structure for Case B is only about 10 percent. Thus, Case A is seven times more likely to exhibit poor seismic performance (significant damage) than Case B during the life of the structure. Importantly, for Case A poor performance is highly likely to occur for relatively frequent earthquakes, with $40 < RP < 100$ years.

Where triggering of such degraded performance is likely at a modest, intermediate return period (eg less than a 100-year return period for a building of normal importance), the resulting level of damage may be excessive and inappropriate for such a high likelihood of occurrence. Tolerable impact limits for such intermediate cases should depend on the return period for triggering and the level of resulting damage to the facility.

Comment

While current code requirements specifically identify only the SLS and ULS criteria to be met, the Canterbury and Kaikoura earthquakes have demonstrated that these performance criteria alone may not be adequate to protect the building stock of an important urban centre in the context of community expectations. It is important to discuss performance expectations with the Client at the outset, and in the assessment intermediate and higher than ULS states may need to be considered to provide a robust design that will control building damage as well as meeting life safety requirements.

Figure 2: Comparative evaluation of the seismic performances of two geotechnical systems



3.6 Performance-based design considerations

In performance-based design (PBD), owners and engineers work together to achieve an optimal balance between construction costs and building performance.

The New Zealand Building Code is performance based and it is permitted to use alternative design procedures (*alternative solutions*) other than Verification Method B1/VM1 to demonstrate compliance with the Building Code performance requirements.

With performance-based design, codified strength-based design (eg B1/VM1) is replaced by a more holistic appraisal of the building performance under various loading scenarios. Performance-based design may require more sophisticated modelling of building response to earthquake loading. Modelling of the foundation system and soil response, including the effects of soil non-linearity, needs to be included in a rigorous way, otherwise the results may be inaccurate and misleading. Structural and geotechnical engineers need to work together closely on such studies to achieve realistic and reliable results.

Performance-based design remains to be the principal approach (design philosophy) for earthquake resistant design of build environment. It is still an evolving approach that undergoes continuous adjustments to accommodate its implementation in the engineering practice. There are several important objectives that PBD is trying to achieve:

a To estimate deformations and consequent level of damage of the structure (ie building-foundation-soil system), for a specific level of earthquake loading (eg RP=25 y, RP=100 y, . . . RP≥ 500 y seismic load)

b To estimate the level of earthquake loading that will cause a marked deterioration in performance (ie to identify damage thresholds; eg damaging liquefaction is triggered at RP=200 y; yielding of piles occurs at RP=500 y; substantial (difficult to repair) damage to a building occurs at RP=300y seismic load).

c To estimate the above while considering the uncertainties in earthquake loads, material behaviour and system response and performance (through the use of parametric and sensitivity studies)

Clearly, the performance-based design philosophy aims for a holistic approach in the evaluation of the seismic performance of a building, in which the performance of key components and the system as a whole are evaluated in the context of the seismic hazard at the site, and specific earthquake scenarios. Such evaluation would provide a sound basis for decision-making on the intended performance by the owner while understanding the associated risks. While current seismic codes are performance-based design in principle, they typically refer to the 'minimum requirements' and selectively identify only specific aspects in the assessment from those mentioned above (eg performance at SLS and ULS levels) to allow for a gradual change of design processes and implementation of PBD in engineering practice.

The main limitation for the implementation of PBD in practice is the lack of readily available practical tools and methods to reliably predict the performance (ie deformation) of a building (-foundation-soil system) including the uncertainty and variability in its performance. Challenges in the implementation also include the ability of practitioners to perform more complex analyses, to consider uncertainty in the models used and soil properties required as inputs in such analyses. While these challenges should be acknowledged, it is important to emphasise that one of the key contributions of the PBD approach is the holistic nature of the assessment of complex systems including the understanding how the seismic performance would evolve with increasing levels of shaking, and for specific earthquake scenarios.

The New Zealand Building Code prescribes minimum performance requirements including safety and reliability of building systems and these need to be addressed explicitly in performance-based design. Key principles from the design philosophy of NZS 1170 should be followed including:

› Uncertainty in the earthquake loading must be accounted for. For methodologies based on response spectra, the hazard spectra derived from NZS1170.5 should be the basis for design. For dynamic time history modelling, uncertainty is considered by using a suite of relevant earthquake records, selected and scaled to match the hazard spectra derived from NZS 1170.5.

› Uncertainty in foundation performance and soil response should be accounted for. (Usually by means of a parametric study including a wide range of key soil strength and stiffness parameters.)

4 Earthquake geotechnical hazards

Earthquakes are sudden ruptures of the earth's crust caused by accumulating stresses (elastic strain-energy) resulting from internal processes of the planet.

Ruptures propagate over approximately planar surfaces called faults releasing large amounts of strain energy. Energy radiates from the rupture as seismic waves. These waves are attenuated, refracted, and reflected as they travel through the earth, eventually reaching the surface where they cause ground shaking. Surface waves (Rayleigh and Love waves) are generated where body waves (p-waves and s-waves) interact with the earth's surface.

The principal geotechnical hazards associated with earthquakes are:

- Fault rupture
- Ground shaking
- Liquefaction and lateral spreading
- Landslides and rockfalls
- Tsunami.

4.1 Fault rupture

For shallow earthquakes, the fault rupture may extend to the ground surface often generating scarps and lateral offsets of several metres.

The extent of surface deformation is dependent on the type of fault, size of fault movements, and the depth and nature of surface soils. Fault rupture induced deformations may be very damaging to buildings and lifelines such as buried services, roads, dams and railways. Light structures may be torn apart if the surface fault rupture dissects the building footprint. For heavier, stronger structures (eg reinforced concrete buildings of more than three storeys on thick soil deposits), the surface fault rupture may locally deviate around the building footprint because of the effect of the additional soil confining pressure and strength of the building foundation relative to the ground beneath it (Bray, 2009). Note: however that such rupture deviation due to presence of strong and robust structures does not always occur, and that faults have ruptured through large dams.

Ground subsidence or uplift induced by fault rupture or global tectonic movement involving relatively large areas may occur during strong earthquakes. Subsidence is often accompanied by inundation and damage to engineering structures over extensive areas, particularly in coastal regions.

The location of known active faults in New Zealand should be obtained from the latest available geological mapping for a site. Active fault locations are also usually shown on the planning maps of Territorial Local Authorities. Many active faults are shown in the GNS active faults database (http://data.gns.cri.nz/af/). The accuracy of such maps varies and the source data (trenches, geophysics, aerial photographs, etc.) should be consulted wherever possible.

Wherever doubt exists, trenching or other means (geophysics, penetration tests and boreholes) should be used to establish the location (or locations) of an active fault trace near to or on a site. It is important to recognise that there are many unknown faults in addition to the mapped faults. Such unknown (unmapped) faults are incorporated through specific considerations and assumptions in the seismic hazard analysis.

Refer
Planning for Development of Land on or Close to Active Faults — A guideline to assist resource management planners in New Zealand, a report published by the Ministry for the Environment.

4.2 Ground shaking

Ground shaking is one of the principal seismic hazards that can cause extensive damage to the built environment and failure of engineering systems over large areas.

Earthquake loads and their effects on structures are directly related to the intensity, frequency content, and duration of ground shaking. Similarly, the level of ground deformation, damage to earth structures and ground failures are closely related to the severity of ground shaking.

In engineering evaluations, three characteristics of ground shaking (ie ground motion) are typically considered:

› amplitude
› frequency content
› duration of significant shaking (ie time over which the ground motion has significant amplitudes)

These characteristics of the ground motion at a given site are affected by numerous complex factors such as the earthquake magnitude, source-to-site distance, effects of local soil and rock conditions, topographic and basin effects, rupture directivity, source mechanism, and propagation path of seismic waves. There are many unknowns and uncertainties associated with these factors which in turn result in significant uncertainties regarding the characteristics of the ground motion and earthquake loads. Hence, special care should be taken when evaluating the characteristics of ground shaking including due consideration of the importance of the structure and particular features of the adopted analysis procedure.

Information on estimating ground motion parameters for earthquake geotechnical engineering purposes is provided in Section 5 of this Module.

4.3 Liquefaction and lateral spreading

Soil liquefaction is one of the principal seismic hazards for urban communities and critical infrastructure in New Zealand.

Liquefaction is associated with significant loss of stiffness and strength in the liquefied soil and consequent large ground deformation as a result of the development of large excess pore water pressures within the soil. Particularly damaging for engineering structures are cyclic ground movements during the period of shaking and excessive residual deformations such as settlements of the ground and lateral spreads.

Ground surface disruption including surface cracking, dislocation, ground distortion, slumping and permanent deformations, such as large settlements and lateral spreads, are commonly observed at liquefied sites. Sand boils, including ejected water and fine particles of liquefied soils, are also typical manifestations of liquefaction at the ground surface. In cases of massive sand boils, gravel-size particles and even cobbles can be ejected on the ground surface due to seepage forces caused by high excess pore water pressures.

Note: sediment (silt, sand, gravel) ejecta are clear evidence of soil liquefaction, however they do not always occur at liquefied sites.

In sloping ground and backfills behind retaining structures in waterfront areas, liquefaction often results in large permanent ground displacements in the down-slope direction or towards waterways (lateral spreads). In the case of very loose soils, liquefaction may affect the overall stability of the ground leading to catastrophic flow failures. Dams, embankments and sloping ground near riverbanks where certain shear strength is required for stability under gravity loads are particularly prone to such failures.

Clay soils may also suffer some loss of strength and exhibit 'cyclic softening' during shaking but are not subject to boils and other 'classic' liquefaction phenomena. However, for weak normally consolidated and lightly over-consolidated clay soils the demand may exceed the undrained shear strength during shaking leading to accumulating shear strain and damaging ground deformations. If sufficient shear strain accumulates, sensitive soils may lose significant shear strength leading to slope failures, foundation failures, and settlement of loaded areas. Ground deformations that arise from cyclic failure may range from relatively severe in natural quick clays (sensitivity greater than eight) to relatively minor in well-compacted or heavily over-consolidated clays (low sensitivity). Studies by Boulanger and Idriss (2006, 2007), and Bray and Sancio (2006) provide useful insights for such soils. The summary in Idriss and Boulanger (2008) is helpful in clarifying issues regarding soil liquefaction and cyclic softening of different soil types during strong ground shaking.

For intermediate soils, the transition from 'sand-like' to 'clay-like' behaviour depends primarily on the mineralogy of the fine-grained fraction of the soil and the role of the fines in the soil matrix. The fines content of the soil is of lesser importance than its clay mineralogy as characterised by the soil's plasticity index (PI). Engineering judgement based on good quality investigations and data interpretation should be used for classifying such soils as liquefiable or non-liquefiable. Bray and Sancio (2006), Idriss and Boulanger (2008), and other studies provide insights on the liquefaction susceptibility of fine-grained soils such as low plasticity silts and silty sands with high fines contents. If the soils are classified as 'sand-like' or liquefiable, then triggering and consequences of liquefaction should be evaluated using procedures discussed in this document and Module 3. On the other hand, if the soils are classified as 'clay-like' or non-liquefiable, then effects of cyclic softening and consequent ground deformation should be evaluated using separate procedures.

Reclaimed land is particularly prone to liquefaction when constructed from liquefiable soils.
This vulnerability was demonstrated during the 2016 Kaikōura earthquake, in which relatively moderate levels of shaking in Wellington caused severe liquefaction in thick reclamations of gravelly soils sourced from quarries and in hydraulic fills.

> Information on the identification, assessment and mitigation of liquefaction hazards is provided in Module 3 of the Guidelines.

4.4 Landslides and rockfalls

Landslides are a familiar geotechnical hazard in many parts of New Zealand.

The rate of incidence of landslides is high during or following high rainfall intensity events, but strong earthquakes also trigger many landslides, including very large, dangerous rockslides. Ground accelerations caused by earthquake shaking can significantly reduce the stability of inclined masses of soil and rock. Even though the acceleration pulses may be of short duration, they may be sufficient to trigger rockfalls or initiate an incipient failure, especially where the soil or rock is susceptible to strain softening or brittle failure respectively. Following strong earthquakes, many slopes may remain in a marginally stable state over a relatively large region, and this often leads to a prolonged period of instability during which a substantially higher rate of incidence of landslides and ground failures may occur.

Earthquake-induced landslides usually affect large areas in the source zone, or even greater areas beyond the immediate source zone in the case of large magnitude earthquakes. As demonstrated in the Canterbury earthquakes, rockfalls, slope instabilities, and associated hazards are very difficult to deal with, and are particularly challenging in an urban setting. This is because they involve large volumes of marginally stable fractured rocks that are difficult to approach, stabilise and mitigate in a cost-effective manner.

Geotechnical evaluation of seismic stability of slopes and rockfalls typically involves assessment of stability under earthquake loading (triggering issues), permanent displacements of slides and rockfalls (run-out distance), and engineering mitigation measures.

> Information on the assessment and mitigation of slope instability and rockfalls may be provided in a future Module of the Guidelines.

4.5 Tsunami

Tsunami has not been recognised as a principal geotechnical hazard.

However, in the 2011 Great East Japan (Tohoku) Earthquake, a mega-tsunami triggered a large number of geotechnical failures of sea walls, breakwaters, river dikes and buildings causing tremendous physical damage and loss of life. In this context, due consideration of potential tsunami hazard must clearly be included in the geotechnical evaluation of structures that are exposed to tsunami hazard in coastal regions.

> NZGS has no present plans to include assessment or mitigation of tsunami hazard within the Guidelines.

5 Estimating ground motion parameters

Earthquakes occur on faults with a recurrence interval that depends on the rate of strain-energy accumulation. Intervals vary from hundreds to tens of thousands of years.

There is much uncertainty over the variability of the strain rate over time, the recurrence interval, the time since the last rupture, the activity of a fault, the location of active faults, and the degree of interaction between various fault segments during rupture.

This section first provides a brief overview of the ground motion characteristics, and then presents the seismic hazard of New Zealand which is recommended for geotechnical assessment. It is an interim hazard until results of the national seismic hazard update are made available for use.

Due to the uncertainty in predicting earthquake events, a probabilistic approach with strong physics-based considerations is commonly adopted to assess the seismic hazard at any location. The level of hazard varies significantly across New Zealand with very high levels near to the Australia/Pacific plate boundary where high rates of tectonic displacement occur. The seismic hazard generally decreases with distance from this zone.

For evaluation of liquefaction phenomena and other problems in earthquake geotechnical engineering, the amplitude of ground shaking caused by the earthquake (commonly represented by the peak horizontal ground acceleration, a_{max}) and the duration of strong shaking (related to the earthquake magnitude, M_w) are the key input parameters to most common design procedures, with no direct consideration of the frequency content of the earthquake loading (which is commonly considered via response spectra).

As incoming seismic waves travel from relatively stiff bedrock into much softer soils towards the ground surface at a site, they slow down and the amplitude of shaking increases. Certain frequencies may be amplified depending on the stiffness, thickness, density and geometry of the soil deposit at the site, as well as the amplitude of the incoming earthquake excitation. For very strong shaking at bedrock level, there may be attenuation of a_{max} and increased displacement amplitude at the ground surface of the site caused by yielding of weak soils and filtering of high frequencies because of the non-linear, strain-dependent stiffness and damping of soil.

Fault rupture in large earthquakes may reach the ground surface and extend over tens or hundreds of kilometres in length. Rupture typically initiates at a 'point' and then propagates along the fault surface at a velocity similar to that of seismic wave propagation. When rupture propagates toward a site, the energy released by the fault rupture can build-up and produce intense ground motions with distinctive velocity pulses. Such forward-directivity near-fault motions have relatively short durations, but high intensity. Backward-directivity motions are less intense, but longer in duration.

The ground shaking hazard at a site depends on the following parameters:

› Amplitude, frequency content and duration of shaking at bedrock beneath the site (which are largely controlled by the magnitude of the earthquake and source-to-site distance).
› Thickness and properties of soil strata beneath the site and overlying the bedrock, as well as bedrock properties themselves (site characteristics).
› Proximity of the site to active faults (including possible directivity and near-fault effects).
› Three-dimensional relief both of the surface contours and sub-strata (ie topographic, sedimentary basin and basin-edge effects).

Site effects, including sedimentary basin and topographic effects can substantially alter ground motion characteristics (ie amplitude, frequency content and significant duration) over short distances, in a given earthquake event. Such examples from recent New Zealand earthquakes are shown in Figure 3, where response spectra of ground motions recorded at nearby rock and soft soil sites are compared. Figure 3a shows records at two Lyttelton sites, which are in proximity to each other and very close to the source of the Christchurch earthquake (~ 2 km source-to-site distance). Because of the short distance from the source, a_{max} at the rock site is very high (~ 0.8 g). The strong ground shaking caused a significant nonlinear ground response (large strains) and softening at the nearby soft-soil site, which in turn resulted in a large reduction of high-frequencies (including a_{max}), but amplification of longer period components (around 1-2 seconds).

Figure 3b shows equivalent records at two Wellington sites, which are close to each other, but at a relatively large distance from the source of the Kaikoura earthquake (~ 60 km source-to-site distance). In this case, the rock ground motion shows low spectral accelerations across all periods including a low a_{max} value (~ 0.09 g), while the soft soil site significantly amplifies the spectral amplitudes across all periods including the a_{max} value (by a factor of three, to ~ 0.30g). These comparisons clearly demonstrate profound site effects on the ground motion characteristics including their strong dependence on the intensity of the earthquake excitation at bedrock.

The ground shaking hazard at a given site should be evaluated through consideration of the above factors, for all potential earthquake sources that may affect the site. The cornerstone methodology for providing such hazard information is the probabilistic seismic hazard analysis (PSHA), which allows to estimate probabilities of earthquake shaking intensities at a given site while considering key contributing factors to the hazard and associated uncertainties.

For problems in earthquake geotechnical engineering including liquefaction hazard assessment, the ground motion parameters at a site may be evaluated using one of the following methods:

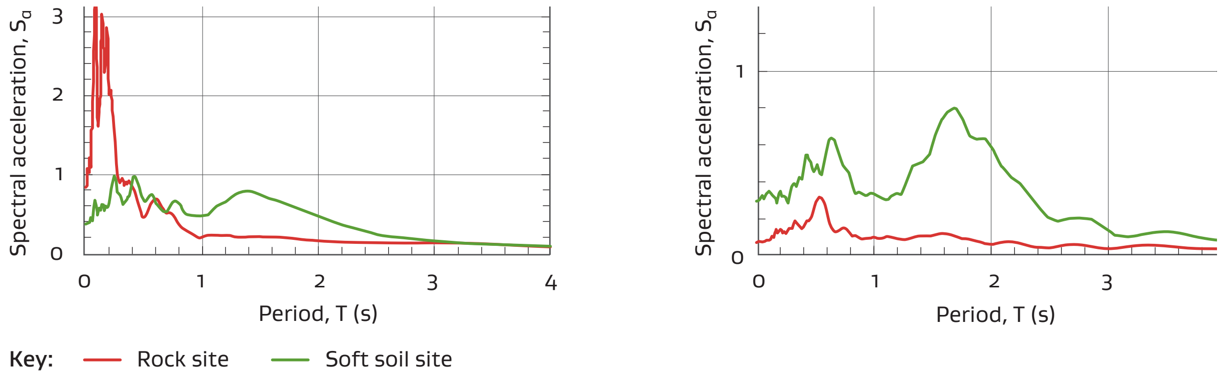
› *Method 1: Estimates based on the National Seismic Hazard Model of New Zealand obtained from comprehensive, but generic PSHA (summarized in this guideline document).*
› *Method 2: Site-specific probabilistic seismic hazard analysis.*
› *Method 3: Site-specific site response analysis.*

Method 1 is appropriate for routine engineering design projects. Methods 2 and 3 are preferred for more significant projects, more complex sites, or other cases where advanced analysis can be justified. Note that Method 3 is practically an extension and enhancement of Method 2. Dismuke and Fraser (2020) provides a useful reference on the appropriateness of site-specific studies for estimating ground motion parameters.

Figure 3. Site effects on ground motion characteristics represented via 5 percent damped response spectra:

a** **Near-fault records (approximately 2 km from the source) obtained at two nearby sites in Lyttelton, one on rock and the other on soft soil site (2011 Christchurch earthquake)

b** **Far-field records in Wellington (approximately 60 km from the source; 2016 Kaikoura earthquake) at two adjacent sites, one on rock and the other on soft soil site (reclaimed land)



5.1 Method 1: Estimates of hazard parameters (a_{max} and M_w) based on generic PSHA and the seismic hazard model of New Zealand

The peak ground acceleration (a_{max}) and earthquake magnitude (M_w) are used to define the earthquake loading in simplified liquefaction assessment and geotechnical engineering evaluations.

In Method 1, estimates of a_{max} and M_w can be obtained from a generic PSHA using the *National Seismic Hazard Model of New Zealand*. Such hazard outputs and estimates of ground motion parameters are provided in NZS1170.5 and NZTA Bridge Manual (2018). The Canterbury and Kaikoura earthquakes have led to further scrutiny of the New Zealand seismic hazard characterization, and several issues with the seismic hazard presented in NZS170.5 and NZTA Bridge Manual have been identified. These include:

› compatibility issues between the magnitude weighting factors embedded in the hazard evaluation of NZS1170.5 and the magnitude scaling factors employed in the liquefaction evaluation procedures, which are recommended in Module 3 of the Guidelines

› the use of an 'effective earthquake magnitude', without clear definition, in NZTA Bridge Manual, and

› the need for updates in the seismic hazard model.

Considerations of elevated seismicity due to the Canterbury earthquake sequence and the consequent MBIE interim guidance for the Canterbury Earthquake Region (CER) also adds to the complexity of the hazard evaluation.

The hazard presented in this document is a step forward from the NZS1170.5 and NZTA Bridge Manual approaches that will provide greater consistency for routine geotechnical engineering projects until a comprehensive update of the NSHM is completed. Reference should be made to the MBIE and NZGS websites for the latest hazard information, as an update of the NSHM and New Zealand seismic hazard is currently in progress.

Further information and details on the recommended seismic hazard for geotechnical assessment presented in this module can be found in Cubrinovski et al. (2021), which includes:

a a brief background on the implementation of the New Zealand seismic hazard in the MBIE-NZGS guidelines;

b detailed comparative analyses between the PGA hazards of NZS1170.5, NZTA-BM and recent site-specific PSHA for 24 locations in New Zealand, and

c an in-depth analysis, discussion and justification of the recommended interim seismic hazard for geotechnical assessment presented in this document.

Recommended interim seismic hazard of New Zealand for geotechnical assessment

The seismic hazard of New Zealand summarized in Table A1 (Appendix A) is recommended for use in Method 1. Table A1 provides a_{max} and M_w values for all of New Zealand, for 25, 50, 100, 250, 500, 1000 and 2500-year return periods. Remarks and footnotes of Table A1 provide calculation details and background of the data used.

The same ground motion parameters are recommended for all site classes. In other words, no scaling of a_{max} is needed for different site classes. (Note: site class is referred to as 'site subsoil class' in NZS1170.5).

The recommendation to use Site Class C peak ground acceleration (a_{max}) for all site classes is based on the following evidence and considerations:

a Results from most recent site-specific hazard analyses (site-specific PSHA) of 24 locations across New Zealand (Bradley et al., 2021; Cubrinovski et al., 2021) show that a_{max} for Site Class D is similar to a_{max} for Site Class C. As illustrated in Figure 4, a consistent trend is seen in results from site-specific PSHA for all sites reflecting the effects of soil nonlinearity on a_{max}. The results show that if a simple form of correlation between $a_{max(Vs200)}$ (Site Class D) and $a_{max(Vs300)}$ (Site Class C) is adopted, then a ratio of $a_{max(Vs200)} / a_{max(Vs300)} = 1.0$ is the best approximation over a wide range of accelerations from 0.1 g to 0.8 g. On this basis, it is recommended to use a_{max} for Site Class C (summarized in Table A1; Appendix A) also for Site Classes D and E.

b Current definition of site classes is provided in NZS1170.5, in which Site Class B ('Rock') is defined for rock sites with '*an average V_{s30} > 360 m/s*'. In the abovementioned site-specific hazard analyses (Bradley et al., 2021), the a_{max} values for Site Class C were estimated assuming V_{s30} = 300 m/s, which is very close to the current V_{s30} definition for Site Class B. On this basis, it is recommended to use a_{max} for Site Class C (summarized in Table A1; Appendix A) also for Site Classes A and B.

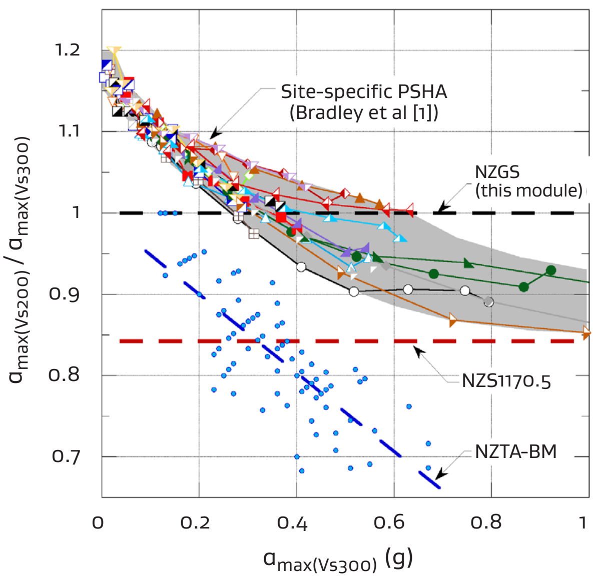
c Note: the recommendations presented in this module apply only to the a_{max} values. They do not imply any modification of spectral values and spectral shape factors presented in NZS1170.5, as such adjustments of spectra for different site classes, is beyond the scope of these geotechnical Guidelines.

Additional clarification and justification of the recommendation adopted in these *Guidelines* to use Site Class C peak ground acceleration (a_{max}) for all site classes is provided in Cubrinovski et al. (2021). It is anticipated that the definition of site classes, amplification factors and spectral shapes will be addressed in the update of the NSHM/New Zealand seismic hazard that is currently in progress.

Note: the a_{max} values in Table A1 are estimates for level ground conditions, ie they do not include geometry effects such as basin-edge and topographic features. Method 3 (site-specific study) is recommended for evaluation of complex geometry effects on ground motion parameters.

Figure 4. Comparison of a_{max} for V_{s30} = 200m/s (Site Class D) and a_{max} for V_{s30} = 300m/s (Site Class C) obtained from site specific PSHA (RP = 500 yr) at sites across New Zealand; the ratio $a_{max\ (Vs200)} / a_{max\ (Vs300)}$ is shown as a function of the intensity of ground shaking.

(Note: each solid line represents the results for one New Zealand location); the dashed lines indicate the amplification factors stipulated in NZS1170.5, NZTA-BM and the recommended value of 1.0 in this guideline.



Several sources of data were used to define the seismic hazard presented in Table A1 (Appendix A):

1. The hazard definition in NZTA-Bridge Manual (2018) was adopted for the majority of New Zealand locations, with the exception of the locations and regions listed under Items 2 to 4 below.
2. For six principal locations (ie Gisborne, Napier, Palmerston North, Wellington, Whanganui and Blenheim), and their associated neighbouring areas listed in Table A1 (Appendix A; Group ID Number, table right hand column: 13, 16, 18, 19, 20 and 23), site-specific hazard definition was adopted based on results from the hazard study commissioned for this guidelines series, NZGS-2020 (Bradley et al., 2021; Cubrinovski et al., 2021).
3. For Auckland and Northland regions, two sets of values are provided for the hazard for return periods RP ≥ 500 yr (ie ULS level and above):
   - one based on the NZTA-Bridge Manual (2018) hazard, and
   - the other (given in brackets in Table A1) based on the lower bound ULS load requirements stipulated in NZTA Bridge Manual (2018; Section 6.2; Table 6.3, p6–6), as described in the respective section below.
4. Region-specific hazard definition for the Canterbury Earthquake Region (CER) was used based on the interim guidance provided by MBIE following the 2010–2011 Canterbury earthquakes.

Important characteristics of the seismic hazard presented in Table A1 (Appendix A) are outlined below. Further details including an in-depth discussion of the hazard analyses and their interpretation are given in Bradley et al. (2021) and Cubrinovski et al. (2021).

Hazard estimates for Method 1 based on NZTA Bridge Manual

For most of the New Zealand regions (*excluding* the Canterbury Earthquake Region and six locations (regions) covered by NZGS-2020), the procedure outlined in NZTA-BM (2018) was followed to determine respective a_{max} and M_{eff} values.

> Following NZTA-BM (2018), peak horizontal ground acceleration (a_{max}) was calculated as:
>
> $a_{max} = C_{0,1000}\, R/1.3\ f\ g$
>
> in which:
>
> $C_{0,1000}$ = Unweighted peak ground acceleration coefficient corresponding to a 1000-year return period, for Class A, B, (rock), from Table C.6.1 (NZTA, 2018)
>
> R = return period factor as given by NZS 1170.5:2004 Table 3.5
>
> g = acceleration from gravity
>
> f = 1.33 site response factor for site Class C (shallow soil)
>
> The earthquake effective magnitude (M_{eff}) depends on the earthquake return period being considered, as specified in Table C.6.1 (NZTA, 2018).

Guidance on selection of appropriate return periods for a particular facility is given in NZS 1170.0 Table 3.3. Typically, for buildings of normal use (Importance Level 2) earthquake motions with a return period of 500 years (R = 1) are used for the ultimate limit state (ULS) and 25 years (R = 0.25) are used for the serviceability limit state (SLS).

Hazard estimates for Method 1 based on site-specific analyses (NZGS-2020)

For six locations (and their geographic areas), the hazard computed in NZGS-2020 (Bradley et al., 2021; Cubrinovski et al., 2021), as summarized below and in Table A.1 (Appendix A), should be used.

For six locations (ie Gisborne, Napier, Palmerstone North, Wellington, Whanganui and Blenheim), and their adjacent regions, the peak ground acceleration (a_{max}) and earthquake magnitude (M_w) values were obtained from site-specific PSHA assuming V_{s30} = 300m/s (Bradley et al., 2021).

For these six locations (regions), it was found that the hazard presented in NZTA Bridge Manual (2018) was substantially lower than the site-specific hazard (Bradley et al., 2021; Cubrinovski et al., 2021), and therefore the hazard values from the site-specific PSHA were adopted for this interim guidance.

Peak ground acceleration (a_{max}) values were explicitly determined for each return period. Earthquake magnitudes (M_w) were computed for 25-year and 500-year return periods; M_w values between 25-year and 500-year return periods were interpolated (Cubrinovski et al., 2021), while M_w values for 500-year return period were adopted for RP > 500 yr.

a_{max} and M_w values for the six locations and their associated areas are summarized in Table A1 (Group ID Number: 13, 16, 18, 19, 20 and 23).

Detailed analysis, interpretation and justification of the recommended interim PGA (Peak Ground Acceleration) hazard for these locations is provided in Cubrinovski et al. (2021).

Hazard estimates for Method 1 for Auckland and Northland regions

For RP ≤ 250 yr, a_{max} and M_w values are adopted based on the NZTA Bridge Manual hazard (2018), as summarized in Table A1. For RP ≥ 500 yr, the higher or more critical load from:

1. a_{max} and M_w values based on the NZTA Bridge Manual hazard (2018), or
2. a_{max} = 0.19 g and M_w = 6.5 (ie lower bound ULS load recommended in NZTA, 2018), shown in brackets in Table A1 (for Groups ID #1 and ID #2 in Table A1), is recommended for use.

NZTA (2018) specifies a 6.5 magnitude earthquake at 20 km distance, as the lower bound ULS load for design. On this basis, it determines a_{max} = 0.19 g and M_w = 6.5 as the minimum ULS load for site Class C (NZTA, 2018; Section 6.2; Table 6.3, p6–6), for RP ≥ 500 yr.

This load definition (ie a_{max} = 0.19 g and M_w = 6.5) may govern the design load for RP ≥ 500 yr and is provided in brackets in Table A1, for Auckland, Waiuku, Warkworth, Pukekohe and Manakau (shown as Group ID #2, Table A.1, Appendix A)[1] and for Northland (ie Group ID #1 in Table A1).

1 In the NZTA Bridge Manual, the lower bound ULS load definition is not associated with a specific return period, and hence it applies generally to RP ≥ 500 yr. Consequently, the same values of amax = 0.19 g and Mw = 6.5 are specified, for example, for RP = 500 yr and RP = 2500 yr. In other words, there is no additional increase in amax for high return periods, which is typically seen in the PGA hazard. As high return periods are relevant for ULS load definition to high importance structures.

Hazard estimates for Method 1 for the Canterbury Earthquake Region

For locations within the Canterbury Earthquake Region, the following procedure is required for the purpose of assessing liquefaction hazard:

Canterbury Earthquake Region

Following the Canterbury Earthquake Sequence (CES), interim guidance by MBIE (2012;2014) was provided for the Canterbury Earthquake Region in which a_{max} values and earthquake magnitude, M_w, were recommended. The annual probability of exceedance is estimated as the average over the period of 50-years following CES (ie the average for the period 2011–2061), considered appropriate for Importance Level 2 buildings.

The recommended values of a_{max} and earthquake magnitude, M_w, are given below. They apply only to deep or soft soil (Class D) sites within the Canterbury Earthquake Region, for liquefaction analysis.

SLS a_{max} = 0.13 g, M_w = 7.5, and
a_{max} = 0.19 g, M_w = 6
ULS a_{max} = 0.35 g, M_w= 7.5

For the SLS, both combinations of a_{max} and M_w must be analysed and the worst-case scenario should be adopted.

For Class D sites outside of Christchurch City and still within the Canterbury Earthquake Region, especially sites closer to the Southern Alps and foothills, it is recommended by MBIE that design a_{max} values be taken as the greater of either the above values or those from NZS1170.5.

Note: the above values have been classified as interim guidance by MBIE. The Ministry has advised that further, more comprehensive guidance may be given as a result of on-going model refinement. Reference should be made to the MBIE website for the latest updates.

Hazard estimates for Method 1 from generic PSHA

Note:

Results from a generic site-specific PSHA might be available from city and regional councils for some regions and urban centres of New Zealand which are currently covered by the NZTA Bridge Manual hazard. It is recommended to make use of such studies when evaluating the hazard parameters for geotechnical assessment.

However that generic site-specific probabilistic seismic hazard assessments should only be carried out by an experienced specialist, and such studies should be subject to a rigorous external peer review.

5.2 Method 2: Site-specific probabilistic seismic hazard analysis

Method 2 is preferred to Method 1 for important structures.

Method 2 allows site specific peak ground accelerations and/or spectra to be developed for the location of interest and specific site characteristics. It also allows for updating of the seismic hazard study based on the best available science at the time of application.

The justification for performing a Method 2 analysis is based on the reasoning that:

› site-specific hazard analysis will provide more accurate modelling of the earthquake loading, site effects, and seismic response

› disaggregation of the site-specific seismic hazard will provide essential input for scenario earthquake analyses, and also for performance-based evaluations for various limit states (see Section 3.5 in this module and Section 10 in Module 3); and

› site-specific hazard analyses could incorporate new information and updated modelling of the hazard using most recent studies and data.

Where a site-specific seismic hazard analysis has been carried out, multiple scenarios using different combinations of a_{max} and effective M_w could be made available for liquefaction triggering assessment, in the form of a disaggregated hazard. In the disaggregated hazard, sources (faults) with the largest contribution to the hazard of the site are identified (ie their percentage contribution to the total hazard for a given RP), and their earthquake magnitudes and source-to-site distance are provided. Thus, a_{max} for the considered RP together with the earthquake magnitude of the specific source, would define the parameters for a particular earthquake scenario associated with that source.

Given the hazard uncertainty and its implication in design, the lower bound values provided in Method 1 (refer to Section 5.1 and Table A1) and stipulated in NZTA Bridge Manual: 2018 (The effects to be designed for shall not be less severe than those due to the lower bound event of a magnitude 6.5 earthquake at 20km distance) should be observed and not be reduced by site-specific probabilistic hazard analyses (Method 2).

Comment

The effect of earthquake magnitude in assessing the risk of liquefaction triggering has received increased significance in the latest update of the simplified procedure [eg Boulanger and Idriss, 2014]. Earthquakes of higher magnitude may trigger liquefaction at significantly lower values of a_{max} than lower magnitude events, and hence, the highest value of a_{max} estimated for the site and its corresponding earthquake magnitude M_w may not represent the critical (worst) case.

Method 2 site-specific probabilistic seismic hazard assessments should only be carried out by an experienced specialist, and such site-specific studies should be subject to a rigorous external peer review.

5.3 Method 3: Site-response analysis

Method 3 involves site-specific amplification considerations through detailed site-response analyses and hence potentially provides more realistic values for site effects than Methods 1 or 2, which both use generic site-response factors according to the site subsoil class.

Method 3 in essence is an extension of Method 2 as it combines site-specific PSHA and site-specific site response analyses. Method 3 is appropriate for more significant projects, more complex sites, or other cases where more analysis can be justified.

Method 3 entails specific modelling of the soil profile of the site requiring more geotechnical information than Methods 1 or 2 including small-strain soil stiffness (eg from shear wave velocity, V_s, profiles) and non-linear soil stress-strain characteristics for each of the modelled soil units.

Site-specific ground response analysis can be carried out to varying levels of detail:

- **1–D analysis**: Various software programs are available to perform this analysis but require good judgement and a good knowledge of the soil properties and profile to bedrock (or sufficiently stiff soil) for the result to be meaningful. Non-linear soil response may be modelled either through an equivalent-linear analysis or a fully non-linear analysis. When using non-linear analysis, particular care should be taken that the adopted stress-strain model accurately represents the stress-strain curve of the soil across the entire range of relevant strains including stiffness, damping and strength of the soil (ie the shear stress at failure or large strains should correspond to the dynamic shear strength of the soil).

  Note: some widely available non-linear models have been calibrated at small to moderate strains only, and they generally provide poor representation of soil stress-strain behaviour at strains greater than 0.5 percent or 1.0 percent. The reports by Stewart et. al. (2008; 2014) provide some guidance for the application of non-linear ground response analysis procedures.
- **2–D and 3–D analyses**: Useful for sites with significant geometry effects where focussing of incoming seismic waves or superposition effects (such as at the edge of a basin or topographic features; eg 1995 Kobe, 2010-2011 Christchurch and 2016 Kaikoura earthquakes) may occur. The direction of incoming seismic waves may significantly affect the result, and therefore care in performing these analyses is required. These are highly specialised analyses for which no generally accepted guidance is available.

Site-specific ground response analyses should carefully address uncertainty in critical soil parameters by including sensitivity studies across relevant parameter values. Effective stress analysis is encouraged to be used in cases where effects of excess pore pressures are significant and where such analysis can be justified.

Given the hazard uncertainty and its implication in design, the lower bound values provided in Method 1 (refer to Section 5.1 and Table A1) and stipulated in NZTA Bridge Manual: 2018 (The effects to be designed for shall not be less severe than those due to the lower bound event of a magnitude 6.5 earthquake at 20km distance) should be observed and not be reduced by site response analyses (Method 3).

Method 3 (site-response analysis) should only be carried out by experienced specialists, and such site-specific studies should be subject to a rigorous external peer review.

6 Guideline modules

This section gives a brief description of the objective and contents of each of the individual modules.

Refer to either the New Zealand Geotechnical Society's website www.nzgs.org/publications/guidelines.htm or to MBIE's website www.building.govt.nz for the latest edition and current status of each module.

6.1 Module 1: Overview of the guidelines

Module 1 (this document) provides an introduction to the Guidelines and the subject of earthquake geotechnical engineering.

The objective for the Guidelines is discussed together with the intended audience. The scope of the Guidelines as a whole is described together with their status within the context of the New Zealand regulatory framework. Procedures for estimating ground motion parameters for use with problems in earthquake geotechnical engineering including liquefaction hazard assessment are provided.

6.2 Module 2: Geotechnical investigation for earthquake engineering

Sites to be developed as part of the built environment must be thoroughly investigated to allow identification and assessment of all geotechnical hazards, including liquefaction related hazards. Identification of liquefaction hazard at a site firstly requires a thorough investigation and sound understanding of the site geology, recent depositional history and geomorphology.

The level of investigation should be appropriate to the geomorphology of the site, the scale of the proposed development, the importance of the facilities planned for the site, and the level of risk to people and other property arising from structural failure and loss of amenity.

Module 2 explains the importance of developing a geotechnical model for a site and describes the key issues to be considered. Guidance is given on planning of geotechnical site investigations. The various techniques available for sub-surface exploration are described in detail and the advantages and disadvantages of each discussed.

Guidance is provided on the preparation of geotechnical reports including appropriate matters to consider in the geotechnical factual report, geotechnical interpretive report, geotechnical design report, and geotechnical construction observation report.

Appropriate densities for site coverage of sub-surface exploration and sampling is discussed and recommendations made. The appropriate depth for sub-surface exploration is also discussed.

Some common problems encountered with site investigation works are discussed.

6.3 Module 3: Identification, assessment, and mitigation of liquefaction hazards

This module introduces the subject of soil liquefaction and describes the various liquefaction phenomena, including lateral spreading.

Guidance is given on identification of liquefaction hazards, including a strategy for appropriate investigations, soil compositional criteria, and geological criteria. Different methodologies for assessing the risk of liquefaction triggering are discussed and recommendations made. Detailed guidance is given on the use of the 'simplified procedure' for assessing risk of liquefaction triggering considered appropriate for everyday engineering situations, together with an explanation of the limitations of this procedure.

Sources of liquefaction induced ground deformation are described and available procedures for assessing ground deformation are outlined. The residual strength of liquefied soils is discussed together with the effects of liquefaction on structures. An overview of ways and means to mitigate the effects of liquefaction and lateral spreading is provided. Numerous references are provided.

A discussion on clay soils and volcanic soils is included. Reference to NZ-specific soils and ground conditions is made, with in-depth discussion on field observations and research findings from well-documented case histories from the 2010-2011 Canterbury earthquakes and 2016 Kaikoura earthquake.

6.4 Module 4: Earthquake resistant foundation design

Module 4 discusses foundation performance requirements during earthquakes within the context of the New Zealand Building Code requirements.

The different types of foundations in common use are described together with a strategy for selecting the most suitable type based on necessary site requirements for each. The particular issues affecting the performance of shallow foundations during earthquakes are explained and guidance on suitable design procedures given. The specific issues affecting the earthquake performance of the various types of deep foundations are discussed together with the advantages and disadvantages of each type. Guidance on analysis and design requirements for deep foundations with earthquake loading is given.

6.5 Module 5: Ground improvement

Module 5 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 is provided on assessing both the need for ground improvement and the extent of improvement required to achieve satisfactory performance.

The various mechanisms for ground improvement are explained, including densification, reinforcement, drainage, chemical modification, solidification, replacement, and lowering of water table. The main techniques for ground improvement are described and discussed in some detail, including dynamic compaction, deep vibratory compaction, stone columns, reinforcement piles, lattice structures, vertical drains, and permanent dewatering.

A matrix summarising the advantages and disadvantages of each technique is presented to provide guidance in selecting the most appropriate method. The reliability and resilience of each technique is discussed and relative cost information presented.

Guidelines for designing ground improvement schemes are presented for the different techniques, together with a discussion of construction and verification considerations.

Several case studies of ground improvement projects both within New Zealand and overseas are presented together with information about actual earthquake performance. Six examples of ground improvement design are presented to demonstrate application of the key design principals outlined in the guidelines. The examples cover common ground improvement techniques used in New Zealand for light weight residential structures, industrial and heavy buildings.

6.6 Module 5A: Specification of ground improvement for residential properties in the Canterbury region

Module 5a provides 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.

The guidance is intended to be limited in use to small scale ground improvement works as typically required for single residential sites (eg 500 m² plan area). A preliminary and general specification is included together with specifications for testing, general earthworks, and technical specifications for the four ground improvement techniques. Guidance is given on how to incorporate site specific technical specifications into a construction contract for the works.

The technical specifications are based on a substantial science and research programme to test residential scale ground improvement options and to identity affordable and practical ground improvement solutions to mitigate the effects of liquefaction for residential properties by the Earthquake Commission, the US National Science Foundation, and MBIE.

The guideline was written originally for immediate use with the Canterbury earthquake recovery but is also considered generally useful for other areas within New Zealand prone to soil liquefaction.

The document does not replace the need for site specific geotechnical investigations or for the design input from a suitably experienced geotechnical engineer.

6.7 Module 6: Retaining walls

Module 6 provides guidance for earthquake resistant design of routine retaining structures in New Zealand practice.

It identifies situations where seismic design of retaining walls should be considered, providing the necessary seismic parameters and identifying key issues relating to seismic design.

Simplified approaches for everyday design cases are provided along with worked examples for common cases.

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Appendix A.

Table A1: Peak Ground Acceleration (a_{max}) and Earthquake Magnitude (M) values recommended for Geotechnical Assessment, for Site Classes A, B, C, D and E, for level ground conditions

LOCATION ID NUMBER[a]	TOWN/CITY	PEAK GROUND ACCELERATION (a_{max})[b] AND EARTHQUAKE MAGNITUDE (M)[c],[d],[e] VALUES RECOMMENDED FOR USE FOR ALL SITE CLASSES (A, B, C, D AND E) — WITHOUT MODIFICATION[f]														BASIS OF DATA (REFER NOTES BELOW FOR DETAIL)[g]	GROUP ID NUMBER[h]
		RETURN PERIOD															
		25-YEAR		50-YEAR		100-YEAR		250-YEAR		500-YEAR		1000-YEAR		2500-YEAR			
		a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M		
1	Kaitaia	0.03	5.8	0.05	5.8	0.07	5.8	0.10	5.8	0.13 (0.19)	5.8 (6.5)	0.17 (0.19)	5.8 (6.5)	0.24 (0.19)	5.8 (6.5)	(1)*	1
2	Paihia/Russell																
3	Kaikohe																
4	Whāngarei																
5	Dargaville																
6	Auckland	0.05	5.9	0.06	5.9	0.09	5.9	0.14	5.9	0.15 (0.19)	5.9 (6.5)	0.20 (0.19)	5.9 (6.5)	0.28 (0.19)	5.9 (6.5)	(1)*	2
7	Warkworth																
8	Manukau City																
9	Waiuku																
10	Pukekohe																
11	Huntly	0.06	5.8	0.08	5.8	0.12	5.8	0.18	5.8	0.24	5.8	0.31	5.8	0.42	5.8	(1)	3
12	Ngāruawāhia																
13	Hamilton	0.06	5.9	0.09	5.9	0.12	5.9	0.18	5.9	0.25	5.9	0.32	5.9	0.44	5.9	(1)	4
14	Te Awamutu																
15	Otorohanga																
16	Thames	0.07	5.9	0.10	5.9	0.14	5.9	0.21	5.9	0.28	5.9	0.36	5.9	0.50	5.9	(1)	5
17	Morinsville																
18	Cambridge																
19	Matamata																
20	Te Kuiti																
21	Paeroa	0.07	5.9	0.10	5.9	0.15	5.9	0.22	5.9	0.30	5.9	0.39	5.9	0.53	5.9	(1)	6
22	Te Aroha																
23	Tauranga																
24	Mount Maunganui																
25	Waihi																
26	Te Puke	0.08	6	0.11	6	0.16	6	0.24	6	0.32	6	0.41	6	0.57	6	(1)	7
27	Putāruru																
28	Tokoroa																
29	Mangakino																
30	Taumarunui																

LOCATION ID NUMBER[a]	TOWN/CITY	PEAK GROUND ACCELERATION (a_{max})[b] AND EARTHQUAKE MAGNITUDE (M)[c],[d],[e] VALUES RECOMMENDED FOR USE FOR ALL SITE CLASSES (A, B, C, D AND E) — WITHOUT MODIFICATION[f] — RETURN PERIOD														BASIS OF DATA (REFER NOTES BELOW FOR DETAIL)[g]	GROUP ID NUMBER[h]
		25-YEAR		50-YEAR		100-YEAR		250-YEAR		500-YEAR		1000-YEAR		2500-YEAR			
		a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M		
31	Rotorua	0.09	6	0.13	6	0.18	6	0.27	6	0.36	6	0.47	6	0.64	6	(1)	8
32	Whakatāne																
33	Kawerau	0.11	6.1	0.15	6.1	0.22	6.1	0.33	6.1	0.44	6.1	0.57	6.1	0.79	6.1	(1)	9
34	Ōpōtiki																
35	Murupara	0.11	6.3	0.15	6.3	0.21	6.3	0.32	6.3	0.43	6.3	0.56	6.3	0.77	6.3	(1)	10
36	Taupō	0.10	6.1	0.14	6.1	0.19	6.1	0.29	6.1	0.39	6.1	0.51	6.1	0.70	6.1	(1)	11
37	Tūrangi	0.09	6.25	0.13	6.25	0.18	6.25	0.28	6.25	0.37	6.25	0.48	6.25	0.66	6.25	(1)	12
38	**Gisborne**															**(3)**	
39	Wairoa	**0.12**	**6.3**	**0.18**	**6.6**	**0.28**	**6.8**	**0.46**	**7.2**	**0.65**	**7.5**	**0.87**	**7.5**	**1.20**	**7.5**	(4)	**13**
40	Ruatoria																
41	New Plymouth	0.07	6	0.10	6	0.14	6	0.21	6	0.29	6	0.37	6	0.52	6	(1)	14
42	Waitara																
43	Inglewood																
44	Stratford																
45	Ōpunake	0.07	6.2	0.10	6.2	0.14	6.2	0.21	6.2	0.28	6.2	0.36	6.2	0.50	6.2	(1)	15
46	Hāwera																
47	Pātea																
48	**Whanganui**	**0.09**	**6.2**	**0.13**	**6.4**	**0.18**	**6.5**	**0.27**	**6.8**	**0.36**	**6.9**	**0.46**	**6.9**	**0.62**	**6.9**	**(3)**	**16**
49	Taihape																
50	Raetihi																
51	Ohakune	0.09	6.3	0.13	6.3	0.18	6.3	0.27	6.3	0.36	6.3	0.47	6.3	0.64	6.3	(1)	17
52	Waiouru																
53	Marton																
54	Bulls																
55	**Napier**															**(3)**	
56	Hastings	**0.12**	**6.4**	**0.18**	**6.5**	**0.26**	**6.7**	**0.42**	**7**	**0.58**	**7.1**	**0.78**	**7.1**	**1.11**	**7.1**		**18**
57	Waipawa															(5)	
58	Waipukurau																
59	**Palmerston North**															**(3)**	
60	Fielding																
61	Dannevirke																
62	Woodville	**0.13**	**6.4**	**0.18**	**6.6**	**0.27**	**6.9**	**0.41**	**7.3**	**0.55**	**7.5**	**0.72**	**7.5**	**1.00**	**7.5**		**19**
63	Pahiatua															(6)	
64	Foxton																
65	Levin																
66	Ōtaki																

LOCATION ID NUMBER[a]	TOWN/CITY	PEAK GROUND ACCELERATION (a_{max})[b] AND EARTHQUAKE MAGNITUDE (M)[c],[d],[e] VALUES RECOMMENDED FOR USE FOR ALL SITE CLASSES (A, B, C, D AND E) — WITHOUT MODIFICATION[f] — RETURN PERIOD														BASIS OF DATA (REFER NOTES BELOW FOR DETAIL)[g]	GROUP ID NUMBER[h]
		25-YEAR		50-YEAR		100-YEAR		250-YEAR	250-YEAR	500-YEAR		1000-YEAR		2500-YEAR			
		a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M		
67	**Wellington**	**0.13**	**6.5**	**0.19**	**6.8**	**0.28**	**7.1**	**0.47**	**7.5**	**0.68**	**7.7**	**0.91**	**7.7**	**1.27**	**7.7**	**(3)**	**20**
68	Paraparaumu															(7)	
69	Porirua																
70	Lower Hutt																
71	Upper Hutt																
72	Eastbourne — Point Howard																
73	Wainuiomata																
74	Masterton																
75	Motueka	0.11	5.9	0.15	5.9	0.21	5.9	0.32	5.9	0.43	5.9	0.56	5.9	0.77	5.9	(1)	21
76	Tākaka																
77	Nelson	0.10	6.1	0.14	6.1	0.20	6.1	0.31	6.1	0.41	6.1	0.53	6.1	0.74	6.1	(1)	22
78	**Blenheim**	**0.12**	**6.4**	**0.18**	**6.6**	**0.26**	**6.8**	**0.39**	**7.1**	**0.52**	**7.3**	**0.67**	**7.3**	**0.90**	**7.3**	**(3)**	**23**
79	Picton															(8)	
80	St Arnaud	0.12	6.1	0.16	6.1	0.24	6.1	0.35	6.1	0.47	6.9	0.61	6.9	0.85	6.9	(1)	24
81	Westport	0.14	6	0.19	6.0	0.28	6.0	0.41	6.0	0.55	6.0	0.72	6.0	0.99	6	(1)	25
82	Reefton																
83	Murchison																
84	Hanmer Springs	0.14	6.5	0.20	6.5	0.28	6.5	0.42	6.5	0.56	7.0	0.73	7.0	1.01	7	(1)	26
85	Kaikōura	0.14	6.1	0.20	6.1	0.28	6.1	0.42	6.1	0.56	6.7	0.73	6.7	1.01	6.7	(1)	27
86	Cheviot	0.11	6.6	0.15	6.6	0.22	6.6	0.33	6.6	0.44	6.6	0.57	6.6	0.79	6.6	(1)	28
87	Hokitika	0.13	6.5	0.19	6.5	0.27	6.5	0.40	6.5	0.53	6.7	0.69	6.7	0.96	6.7	(1)	29
88	Greymouth																
89	Otira	0.17	6.4	0.23	6.4	0.33	6.4	0.50	6.4	0.67	7.1	0.86	7.1	1.20	7.1	(1)	30
90	Arthurs Pass																
91	Darfield (CER)–M6	0.19	6							0.41	6.3					(2) & (1)	31
	Darfield–M7.5	0.13	7.5							0.35	7.50					(2) & (1)	
92	Christchurch[i] (CER)–M6	0.19	6													(2)	32
	Christchurch[i] (CER)–M7.5	0.13	7.5							0.35	7.5						
93	Rangiora (CER)[i]–M7.5																
94	Akaroa[i] (CER)–M7.5																
95	Ashburton	0.06	6.1	0.09	6.1	0.13	6.1	0.19	6.1	0.26	6.1	0.33	6.1	0.46	6.1	(1)	33
96	Geraldine																
97	Aoraki/Mount Cook	0.12	6.2	0.16	6.2	0.23	6.2	0.35	6.2	0.46	6.9	0.60	6.9	0.83	6.9	(1)	34
98	Wānaka	0.10	6.1	0.14	6.1	0.20	6.1	0.30	6.1	0.40	6.1	0.52	6.1	0.72	6.1	(1)	35
99	Twizel																

LOCATION ID NUMBER[a]	TOWN/CITY	PEAK GROUND ACCELERATION (a_{max})[b] AND RECOMMENDED FOR USE FOR ALL SITE CLASSES EARTHQUAKE MAGNITUDE (M)[c],[d],[e] VALUES (A, B, C, D AND E) — WITHOUT MODIFICATION[f]																	BASIS OF DATA (REFER NOTES BELOW FOR DETAIL)[g]	GROUP ID NUMBER[h]
		RETURN PERIOD																		
		25-YEAR		50-YEAR		100-YEAR		250-YEAR		250-YEAR	500-YEAR		1000-YEAR		2500-YEAR					
		a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)		M	a_{max}(g)	M	a_{max}(g)	M	a_{max}(g)	M				
100	Cromwell	0.08	6.2	0.12	6.2	0.17	6.2	0.25		6.2	0.34	6.2	0.44	6.2	0.61	6.2	(1)	36		
101	Fairlie																			
102	Alexandra	0.07	6.3	0.10	6.3	0.15	6.3	0.22		6.3	0.30	6.3	0.39	6.3	0.53	6.3	(1)	37		
103	Queenstown	0.10	6.5	0.14	6.5	0.20	6.5	0.31		6.5	0.41	6.5	0.53	6.5	0.74	6.5	(1)	38		
104	Arrowtown																			
105	Milford Sound	0.16	6.1	0.22	6.1	0.32	6.1	0.48		6.1	0.63	7.1	0.82	7.1	1.14	7.1	(1)	39		
106	Dunedin	0.06	6	0.08	6	0.11	6	0.17		6	0.23	6	0.29	6	0.41	6	(1)	40		
107	Temuka																			
108	Timaru																			
109	Waimate																			
110	Oamaru																			
111	Palmerston																			
112	Mosgiel																			
113	Balclutha																			
114	Te Anau	0.11	6.4	0.15	6.4	0.22	6.4	0.33		6.4	0.44	6.4	0.57	6.4	0.79	6.4	(1)	41		
115	Riverton	0.07	6.2	0.09	6.2	0.13	6.2	0.20		6.2	0.27	6.2	0.35	6.2	0.48	6.2	(1)	42		
116	Winton																			
117	Gore																			
118	Mataura																			
119	Invercargill	0.05	6.1	0.08	6.1	0.11	6.1	0.16		6.1	0.21	6.1	0.28	6.1	0.39	6.1	(1)	43		
120	Bluff																			
121	Oban																			

Footnotes

(a) Numbering for locations, refer to Table A2 for alphabetical list of locations (Note: not identical numbering of locations in Table 3.3 of NZS 1170.5)
(b) a_{max} estimated for Class C shallow soil (NZTA-BM; 2018) or V_{s30} = 300m/s (NZGS-2020)
(c) Meff—effective magnitude is used for all data based on (NZTA-BM; 2018)—ie Basis of Data (1) in right hand column of Table A1
(d) M_w—Mean moment magnitude is used for all data based on (NZGS-2020)—ie Basis of Data (3) to (8) in right hand column of Table A1
(e) M_w—Mean moment magnitude for RP=500 years (NZGS-2020) are adopted for use for RP>500 years for Basis of Data (3) to (8) in right hand column of Table A1
(f) a_{max} and M values listed in the table apply to all site classes without any scaling or modification; a_{max} estimated for Class C shallow soil (NZTA-BM, 2018; Locations (1)) or V_{s30} = 300m/s (NZGS-2020; Locations (3) to (8));
Note: a_{max} estimates are for level ground conditions (ie effects of basin edge and topographic features are not included)
(g) Origin of data, refer Notes (g) below
(h) Grouping of locations, towns and cities sharing very similar hazard
(i) Canterbury Earthquake Region (CER)—MBIE Guidance for M = 7.5
(j) Canterbury Earthquake Region (CER)—MBIE Guidance for M = 6.0

Notes [g]—Origin of Data presented

(1) a_{max} and Meff values for subsoil Class C based on NZTA Bridge Manual (2018), Table C6.1; R-value based on NZS1170.5, Table 3.5;
(1)* (1) a_{max} and M_w values (not in brackets) for subsoil Class C based on NZTA Bridge Manual (2018), Table 6.3; R-value based on NZS1170.5, Table 3.5;
(2) For R ≥ 500yr: a_{max} = 0.19g and M_w=6.5 (values in brackets) based on lower bound ULS load (6.5 earthqauke magnitude at 20km distance) specified in NZTA Bridge Manual (2018)
(2) Parameters based on MBIE 2014 Canterbury Earthquake Region guidance
(3) Parameters based on NZGS-2020 Hazard study
(4) Location associated with NZGS-2020 hazard for Gisborne
(5) Location associated with NZGS-2020 hazard for Napier
(6) Location associated with NZGS-2020 hazard for Palmerston North
(7) Location associated with NZGS-2020 hazard for Wellington
(8) Location associated with NZGS-2020 hazard for Blenheim

Table A2: Alphabetical list of locations

TOWN/CITY	LOCATION ID NO.
Akaroa	94
Alexandra	102
Aoraki/Mount Cook	97
Arrowtown	104
Arthurs Pass	90
Ashburton	95
Auckland	6
Balclutha	113
Blenheim	78
Bluff	120
Bulls	54
Cambridge	18
Cheviot	86
Christchurch	92
Cromwell	100
Dannevirke	61
Darfield	91
Dargaville	5
Dunedin	106
Eastbourne—Point Howard	72
Fairlie	101
Fielding	60
Foxton	64
Geraldine	96
Gisborne	38
Gore	117
Greymouth	88
Hamilton	13
Hanmer Springs	84
Hastings	56
Hāwera	46
Hokitika	87
Huntly	11
Inglewood	43
Invercargill	119
Kaikohe	3
Kaikōura	85
Kaitaia	1
Kawerau	33
Levin	65

TOWN/CITY	LOCATION ID NO.
Lower Hutt	70
Manukau City	8
Mangakino	29
Marton	53
Masterton	74
Matamata	19
Mataura	118
Milford Sound	105
Morinsville	17
Mosgiel	112
Motueka	75
Mount Maunganui	24
Murchison	83
Murupara	35
Napier	55
Nelson	77
New Plymouth	41
Ngāruawāhia	12
Oamaru	110
Oban	121
Ohakune	51
Ōpōtiki	34
Ōpunake	45
Ōtaki	66
Otira	89
Otorohanga	15
Paeroa	21
Pahiatua	63
Paihia/Russell	2
Palmerston	111
Palmerston North	59
Paraparaumu	68
Pātea	47
Picton	79
Porirua	69
Pukekohe	10
Putāruru	27
Queenstown	103
Raetihi	50
Rangiora	93
Reefton	82

TOWN/CITY	LOCATION ID NO.
Riverton	115
Rotorua	31
Ruatoria	40
St Arnaud	80
Stratford	44
Taihape	49
Tākaka	76
Taumarunui	30
Taupō	36
Tauranga	23
Te Anau	114
Te Aroha	22
Te Awamutu	14
Te Kuiti	20
Te Puke	26
Temuka	107
Thames	16
Timaru	108
Tokoroa	28
Tūrangi	37
Twizel	99
Upper Hutt	71
Waihi	25
Waimate	109
Wainuiomata	73
Waiouru	52
Waipawa	57
Waipukurau	58
Wairoa	39
Waitara	42
Waiuku	9
Wānaka	98
Warkworth	7
Wellington	67
Westport	81
Whakatāne	32
Whanganui	48
Whāngarei	4
Winton	116
Woodville	62

Appendix B. New Zealand building regulatory system

Note
Appendix B does not represent a complete reproduction of the Building Code regulation and readers should regularly website check www.building.govt.nz for the most up-to-date information on the Building Code Regulations.

B.1 Overview of the Regulatory System

The regulation and performance of buildings in New Zealand sits under the following three-part framework.

- The Building Act (2004), which is the legislation that contains the provisions for regulating building work. It sets out the legal requirements for ensuring all new building designs, repairs, alterations, demolition and removal will comply with the supporting Building Regulations and the New Zealand Building Code.
- The various building regulations, in particular the Building Regulations (1992) which in its Schedule 1 contains the New Zealand Building Code. The Building Code establishes a performance-based system in that it sets performance standards that all new building work must meet, covering aspects such as stability, durability, protection from fire, access, moisture, safety of users, services and facilities, and energy efficiency.
- Verification methods and acceptable solutions, which are provided for all Building Code clauses. These provide one way (but not the only way) of complying with the Building Code. The performance-based Building Code system allows Alternative Solutions provided the design can be shown to meet the performance criteria of the Building Code, to the satisfaction of the Building Consent Authority.

B.1.1 BUILDING ACT

The Building Act principles include:

- All building work needs to comply with the Building Code, whether or not a building consent is required (s 17)
- Buildings need to be durable for their intended purpose (s 4 (2) (c)).
- The whole of life costs of a building (including maintenance) need to be considered (s 4 (2) (e))
- The importance of standards of building design and construction in achieving compliance with the Building Code (s 4 (2) (f))
- Other property needs to be protected from physical damage resulting from the construction, use and demolition of a building (s 4 (2) (j))
- Owners, designers, builders and building consent authorities each need to be accountable for their role in obtaining consents and approvals, ensuring plans and specifications for building work will meet the Building Code (s 4 (2) (q))
- The Building Consent Authority must have ‘reasonable grounds’ to grant a building consent (s 49)
- Buildings with specified intended lives (s 113).

Note: refer to section 4 of the Building Act for a full list of principles.

The following table summarises the normal interpretation of B1:

Table B.1: Building Code Performance Requirements of Clause B1 (Structure)

CLAUSE B1 REFERENCE	PERFORMANCE CRITERIA	
	SERVICEABILITY / AMENITY	STABILITY
B1.3.1 — low probability of instability. Relates to ULS events	NA	Gross deformation of foundations that could lead to collapse to be avoided eg bearing failure, sliding
B1.3.2 — low probability of loss of amenity. Relates to SLS events	Avoid undue deformation of foundations and structure. Building must be readily usable after the event	NA
B1.3.3 — lists physical conditions likely to affect building stability	NA	Includes earthquake, differential movement and adverse effects on buildings such as temporary loss of geotechnical bearing capacity due to liquefaction

Note: Other sub clauses listed under B1 (structure) also need to be satisfied, eg B1.3.4 to B1.3.7. Refer to Building Code Clause B1 (structure) for full requirements.

B.1.2 BUILDING CODE

The New Zealand Building Code sets out the performance criteria to be met for all new building work. The Building Code does not prescribe how work should be done but states how completed building work and its parts must perform. The Building Code covers aspects such as stability, protection from fire, access, moisture, safety of users, services and facilities, and energy efficiency.

Buildings need to comply with all clauses of the Building Code — however clause B1 (structure) of the Building Code is often the primary driver of the geotechnical and structural design aspect of a building. Amongst other things, B1 states that 'buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium or collapsing during construction or alteration and throughout their lives'. They should also have 'a low probability of causing loss of amenity…'

Of the two sets of loading criteria (ie SLS and ULS) meeting the serviceability requirements of clause B1 on liquefiable soils can prove the more challenging. The deformation performance and its prediction are subjective issues lacking the ability to precisely calculate the effects, particularly when an SLS event could trigger liquefaction of the soils below the foundation that may or may not lead to building deformation. Secondly it is easier to calculate that a building is unlikely to collapse with modest foundation deformation.

A critical feature in meeting serviceability requirements is to demonstrate that the intended use of the building will be maintained or can be restored within a short time at reasonable cost. For instance, a factory floor that has minor cracking from the effects of liquefaction in an SLS earthquake event, but the building is otherwise safe and functional could be deemed to meet the serviceability standard. However, a four-storey building that rotates on its foundations just sufficient to render the internal lifts inoperable will likely require closure of the upper two floors until repairs can be effected, which may take months to achieve. This latter situation could be deemed to not meet the Code for Serviceability as the upper stories have lost a key means of access that will take a long time and significant expense to reinstate.

The Building Act provides a number of pathways that designers may follow to achieve compliance with the Building Code.

- Acceptable solutions provide a prescriptive means of meeting the Building Code. If followed by the designer, the designer must be granted a building consent as they are deemed to comply with the Building Code. This is the simplest path.
- Verification methods provide a prescriptive design method, which if followed by the designer will produce a design that is also deemed to comply with the Building Code. This path does require more scrutiny than designs that follow

an Acceptable Solution to check that correct assumptions and within the verification method are used and that any calculations used in the design have been done correctly.

- Alternative solutions whereby designers demonstrate to the satisfaction of the Building Consent Authority that a design solution, not covered directly with an acceptable solution or verification method, does achieve the performance requirements of the Building Code. Demonstration may include fundamental engineering design and expert review, history of use, or testing of the design or product. If it can be demonstrated to the Building Consent Authority that the performance criteria are achieved, the Building Consent Authority must also grant a building consent.

Section 49 of the Building Act emphasises that before a building consent can be issued the application must provide the assessing officer with confidence (on 'reasonable grounds') that, if built as specified, the building is likely to comply with the Building Code. 'Reasonable grounds' is not defined in the Act but it is usually accepted by Building Consent Authorities as meaning less than a full technical review of the application. But sufficient documentation must be provided in the consent application as to create a reasonably held expectation by the consenting officer that the Building Code requirements will be met. The onus is on the applicant to ensure an adequate level of work has been done to attain the reasonable grounds benchmark.

B.2 The status and relevance of the MBIE Guidelines for residential houses in Canterbury

Following the initial earthquake of the Canterbury Earthquake Sequence in 2010-11, the former Department of Building and Housing (DBH), now the Ministry of Building Innovation and Employment (MBIE) recognised that the existing design standards and Building Code did not provide adequate guidance on how to comply with the Building Code when reinstating houses damaged by the effects of liquefaction.

Consequently, a guideline with regular revisions was developed, setting out how to assess the degree of future liquefaction and providing suggested foundation options that would suit particular liquefaction conditions.

Development of the MBIE Guidance for house foundation replacement in Canterbury, under s.175 of the Building Act 2004, explicitly recognised that the existing Acceptable Solutions and Verification Methods did not cover foundations on liquefiable soils. Therefore, many of the foundation solutions provided in the MBIE Guidance, have been developed by MBIE as Alternative Solutions (Section 8.2.1 of the MBIE Guidance 2012).

The MBIE Guidance was developed for specific application to residential properties in the Canterbury area and was not intended to be used in other parts of New Zealand. Therefore, while the guidance will serve as a useful reference for site investigation elsewhere in New Zealand, practitioners, owners and consenting authorities need to be aware of the possible limitations particularly if commercial projects are being considered or where the geological and/or seismic settings are substantially different.

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